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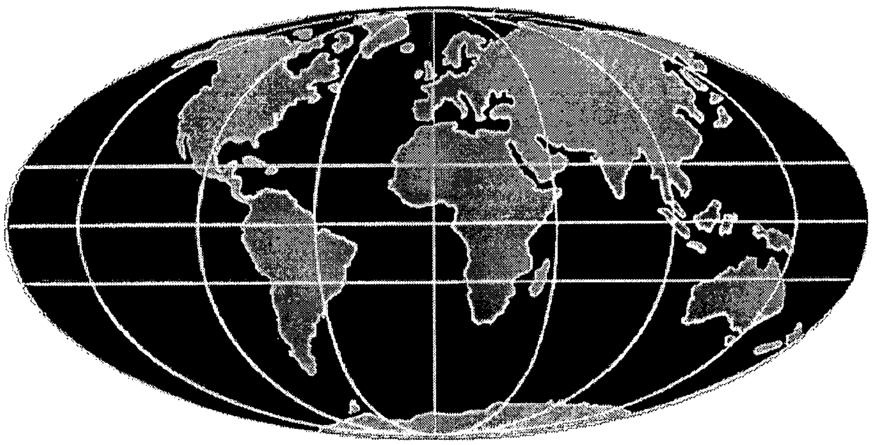
TITLE: Designing asphalt mixes to last

by: A B Sterling and K A Zamhari



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DESIGNING ASPHALT MIXES TO LAST

by
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DESIGNING ASPHALT MIXES TO LAST

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DESIGNING ASPHALT MIXES THAT LAST

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ABSTRACT

There is a worldwide tendency for traffic to get heavier. This has been accompanied by an ever increasing number of road failures caused by plastic ruts and corrugations. Those who have sought to avoid this problem, by making leaner, stiffer mixes, have often found their asphalt failing, very prematurely, by cracking.

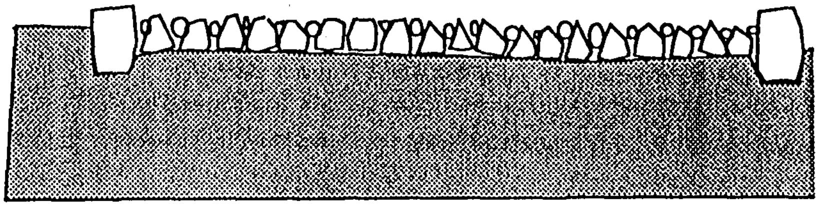
This paper discusses the mechanisms of both cracking and plastic flow failures. It offers a simple mix design procedure and a set of mix design criteria that can help avoid plastic flow without becoming susceptible to premature cracking.

DESIGNING ASPHALT MIXES THAT LAST

1. A BRIEF HISTORY OF ROADS IN INDONESIA

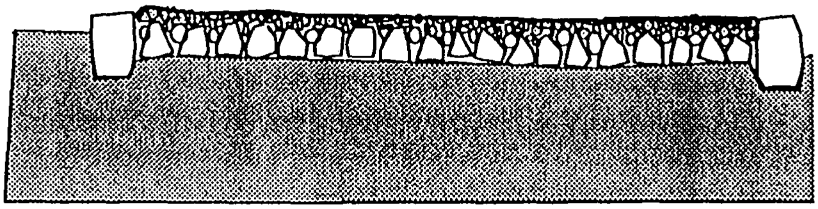
Routes from village to village always start as earth tracks. On steep hills, especially those with clay soils, it is necessary to cut steps and this is the first level of "engineering." As soon as modes of transport develop from the use of legs (whether two or four) to the use of wheels it becomes necessary to provide some form of surfacing. In Java the most common has been the "Telford" type pavement.

Figure 1 Telford Type Surfacing



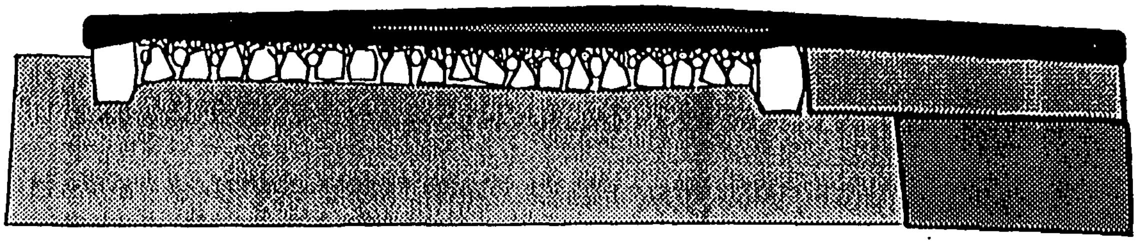
This needs no machines. A road bed is excavated by hand and large "kerb" blocks are set into the ground at the edges. Between these the road is made of hand placed, hand broken stones. These are preferably slightly wedge shaped, with the wider face at the bottom. Key stones are then used to lock the larger pieces together. You can still see these roads being built on hillsides throughout Java. They provide cheap and effective (but slow and uncomfortable) all weather access to remote communities. When traffic levels grow, the uneven Telford surfaces become unacceptable. In Java, the next stage, of development was Penetration Macadam. The Telford was covered with smaller hand broken stone that were sealed with a coat of hot bitumen. As the name "Penetration Macadam" indicates the bitumen did penetrate to some extent and this held the surface together.

Figure 2 Penetration Macadam Type Surfacing



Penetrasi surfacings are still very common on Indonesian roads that carry low traffic levels. They can be maintained by periodic surface seals using bitumen emulsion, blinded with sand or grit. However, hand made roads are not very smooth. Fast driving is usually uncomfortable, sometimes unsafe. As traffic levels increased it became an economic necessity to re-surface main roads with machine laid asphalt. In many cases this work was accompanied by widening of the old road.

Figure 3 AC Laid over Penetration Macadam Type Surfacing



The first material used for this purpose was Asphaltic Concrete. This is a stiff material that is rather sensitive to variations in mix proportions. It was not very suitable for use over the Telford/Penetrasi constructions, some of which were rather flexible. The result was that many of the Asphaltic Concrete surfaces failed, by cracking, very prematurely.

The current range of Specifications used in Indonesia (DGH 1986 & 1992) were developed to solve the problem of premature cracking. They achieved that aim, by substantially increasing the bitumen content of asphalt mixes, but we now have a new problem. **The majority of new road surfacings still fail prematurely, this time by plastic rutting.**

The purpose of this paper is to explain what is going wrong.
It will answer questions such as:

- How much of the problem is the fault of the current Specifications?
- What can we do to correct them?
- How much of the problem comes from people ignoring the Specifications?

To answer these questions we will look at:

- The earlier cracking failures in a little more detail.
- The causes of the plastic rutting that we now suffer from.
- What we can do to avoid *premature* failure by both these mechanisms.

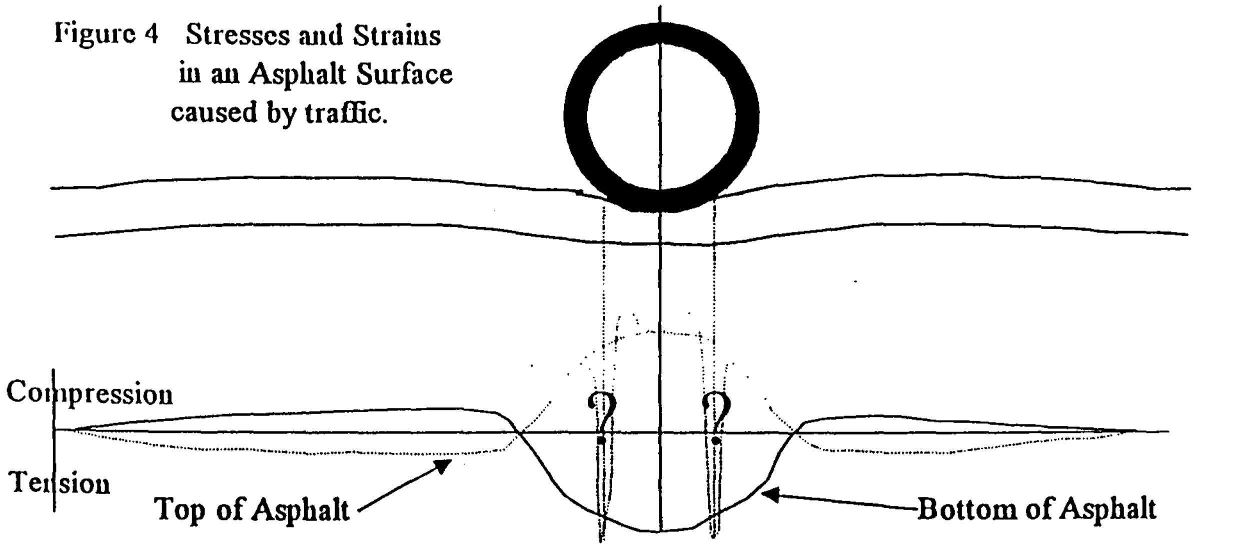
2. WHY DID THE ASPHALTIC CONCRETE FAIL ?

The simple answer is that the mix was too stiff for the high levels of strain that it experienced. However, that simple answer hides many mis-conceptions and we need to look at these if we are not to be in danger of repeating our mistakes.

2.1 Causes of cracks in road pavements

Cracks are caused by tensile stresses or strains. These can result from Traffic or the Environment.

Figure 4 Stresses and Strains in an Asphalt Surface caused by traffic.



Traffic: Standard structural theory says that the largest tensile strains occur at the bottom of the asphalt, directly under the vehicle wheel. Smaller, but significant, tensile strains also occur at the top of the asphalt, before and after the wheel. However, there are also strains all around the contact area between the tyre and the road, caused by localised deformation of the surface, and immediately below the wheel, caused by traction, braking and steering forces. All these strains occur predominantly at the upper surface of the asphalt.

Environment: Changes in temperature, from day to night and from hot to cold seasons, produce tensile strains in the asphalt, especially at the upper surface. This mechanism is probably less important in Indonesia than in countries farther from the tropics.

2.2 Where cracks start

At the surface of the road the bitumen looses its lighter oils, by evaporation, and is progressively oxidised. These changes lead to it becoming hard and brittle. The penetration can drop to between 10 and 20 at the top while it may be 50 or above in the body of the layer.

Classical pavement design theory assumed that cracks would start at the bottom of the asphalt because that is where the tensile strains, caused by flexure of the pavement, are largest. ***However, cores taken from cracked roads have shown that most cracks start at the top of the asphalt. This observation has been verified in many parts of the world, including UK, Africa, Arabia and Indonesia.*** Apparently, the embrittlement makes the tensile strains at the top of the layer more damaging than those at the bottom.

2.3 Design to prevent premature failure by cracking

Any bituminous mix, even pure bitumen, will fail by cracking if large enough tensile strains are applied to it, often enough. There is usually an approximately linear relationship between the logarithm of tensile strain amplitude and that of the number of strain repetitions the material can stand before cracking.

Well-designed mixes will have a higher strain tolerance than bad ones but all will eventually fail. Even a good mix may fail prematurely if the road is too weak and the strains too high for the number of vehicles that must be carried. Hence, prevention of *premature* cracking involves both:

- mix design and
- pavement design.

Mix Design Parameters that are likely to correlate well with good resistance to cracking include:

- Binder Properties, such as Penetration or Viscosity and maybe Penetration Index.
- Effective Binder Content (EBC).
- Voids in Mineral Aggregate and % Voids Filled with Bitumen (VMA/VFB).
- Binder Film Thickness (BFT).

In mixes with a continuous stone matrix, only partially filled with sand asphalt, the atmosphere has access to the individual, coated particles of aggregate. In such mixes is possible that Binder Film Thickness (BFT) will be useful to ensure there is "enough" bitumen to make the mix durable. Mixes that fall into this class include coarse DBM mixes as well as SMA open graded asphalts. The grading curves for all these mixes lie well below the Fuller Curve.

In mixes that have a continuous sand asphalt matrix, with discontinuous stone particles, it is the sand asphalt that is exposed to the atmosphere. Because this is a continuum, the concept of individual particles, each coated by a finite thickness of binder, is not relevant. For such mixes, which include continuous (AC) and gap graded (HRS) mixes, the percentage Voids Filled with Bitumen (VFB) is likely to be a better criterion for durability than BFT.

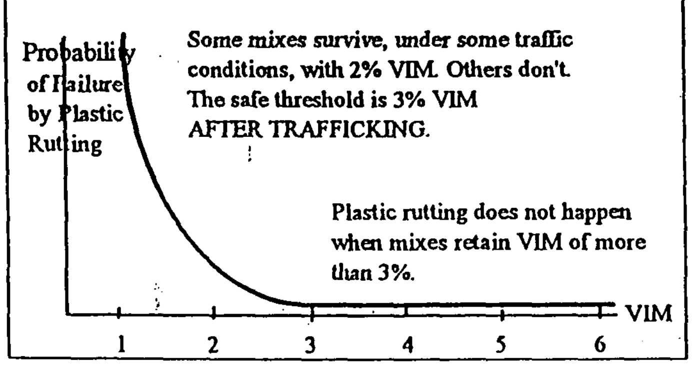
To be successful an asphalt mix must have "enough" bitumen to bind the aggregate particles and "enough" air voids (VIM) to avoid failure by plastic flow. In an asphalt that has low Voids in the Mineral Aggregate (VMA), these two requirements are contradictory. If you add enough bitumen for durability there will not be sufficient VIM; if you leave sufficient VIM there will not be enough bitumen to make a durable mix. Therefore, it is essential to achieve a high VMA that allows room for both "enough" bitumen and "enough" VIM.

3. WHY DO WE HAVE PLASTIC FLOW IN OUR ROAD PAVEMENTS

3.1 The cause of plastic flow in asphalt

Studies in several countries have shown that when the VIM drop below 3% asphalt concrete mixes are very likely to fail by plastic flow. Data from roads observed in Indonesia confirms that the probability of failure by plastic deformation increases greatly as VIM drop beyond 3%. (TARP 1993) See Figures 7, 8 and 9 for evidence from the Middle East and from Malaysia. (TRL 1997) There is some evidence to suggest that gap graded mixes may have a slightly lower failure threshold (2% VIM) than continuously graded mixes (3% VIM).

Figure 5 Plastic Rutting and Refusal VIM

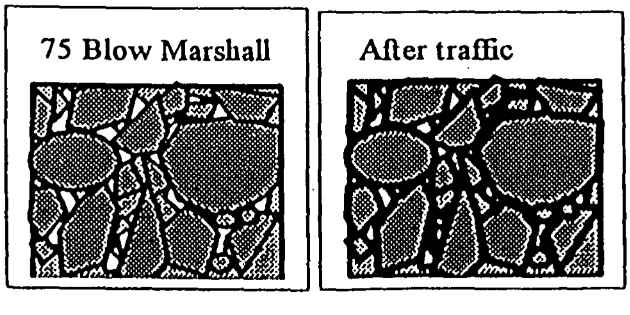


We can offer an explanation for the above. It seems reasonable but we need to remember that it is only hypothetical. To resist plastic flow, asphalt mixes depend on both internal friction, between the aggregate particles, and binder viscosity. If, at any point in the mix, the local VIM approaches zero, the bitumen begins to separate one aggregate particle from another. In other words, it ceases to be a binder and becomes a lubricant. Once this happens the mix will fail by plastic flow. Continuously graded mixes are highly non-homogeneous. By the time the average VIM has dropped to 3% there may be a significant number of locations in the mix that have near zero voids. Gap graded mixes, especially those with lower stone contents, would be expected to be less inhomogeneous and this may account for their reaching average VIM as low as 2% before flow becomes significant.

There is some evidence to suggest that in "free flow" conditions the VIM threshold is closer to 2% than 3% but this is not yet certain. Indeed, the American Asphalt Institute, in the 1994 issue of their mix design manual (MS-2 1994), recommend the final void content after trafficking should be 4%.

3.2 Mix design to prevent plastic flow

Figure 6 Loss of VIM Due to Compaction by Traffic



Marshall mix design procedures have always required the VIM to be above 3%. The trouble is, the density achieved by 35, 50 or even 75 blows on each face of the sample is less than that which occurs in the wheel paths of roads carrying severe traffic loads. The final in situ density may be 3% or even 4% higher than the 75 blow Marshall density.

This means that 3% or 4% of air voids can be lost during the further compaction caused by traffic. If the original design was for 4% or 5%, the residual VIM can have dropped to between 0% and 2%, in which case the road will be failing by plastic flow. To prevent this happening, the Asphalt Institute (MS-2 1994) now recommend that mixes are laid and compacted to air voids of 8%. This allows for 3% - 5% of expected consolidation during trafficking. The target for the "final" VIM is 4%.

To be sure that in-situ VIM never drop below 3% we need an additional test procedure, in which samples are compacted to a refusal condition. That is, until they "refuse" to become any more dense. ***We can then set an upper limit to the design binder content, corresponding to 3% VIM at the refusal condition.***

There are two methods by which one can perform "refusal density" teats.

The method we recommend uses a vibrating hammer to compact samples in a CBR mould. It follows the procedures described in the Percentage Refusal Density (PRD) test in BS 598 (BSI 1989), with one major improvement. The hammer is surcharged by hanging 30 kg onto it rather than using body weight.

The other method is an extension to the Marshall test (AASHTO T245), using about 400 blows per face instead of 75.

It is essential that the density achieved in these "refusal density" tests really does correspond to that reached in the wheel paths under severe traffic.

The vibrating hammer method is quicker than the extended Marshall procedure, is less likely to break the aggregate particles and it is very easy to do. The extended Marshall procedure is not difficult but it is unpleasantly noisy and does break down more aggregate than the vibrating hammer method.

<u>VIM as a Determinant for the Performance of Asphalt Concrete in the Middle East</u>

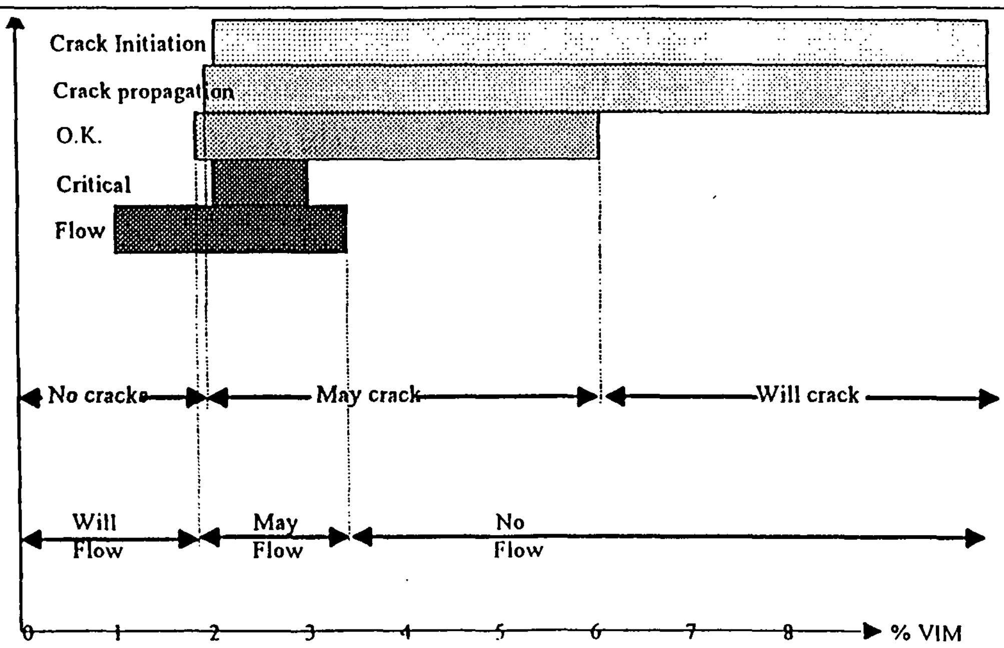


Figure 7 Performance of Asphalt Concrete on "Middle East Road One"

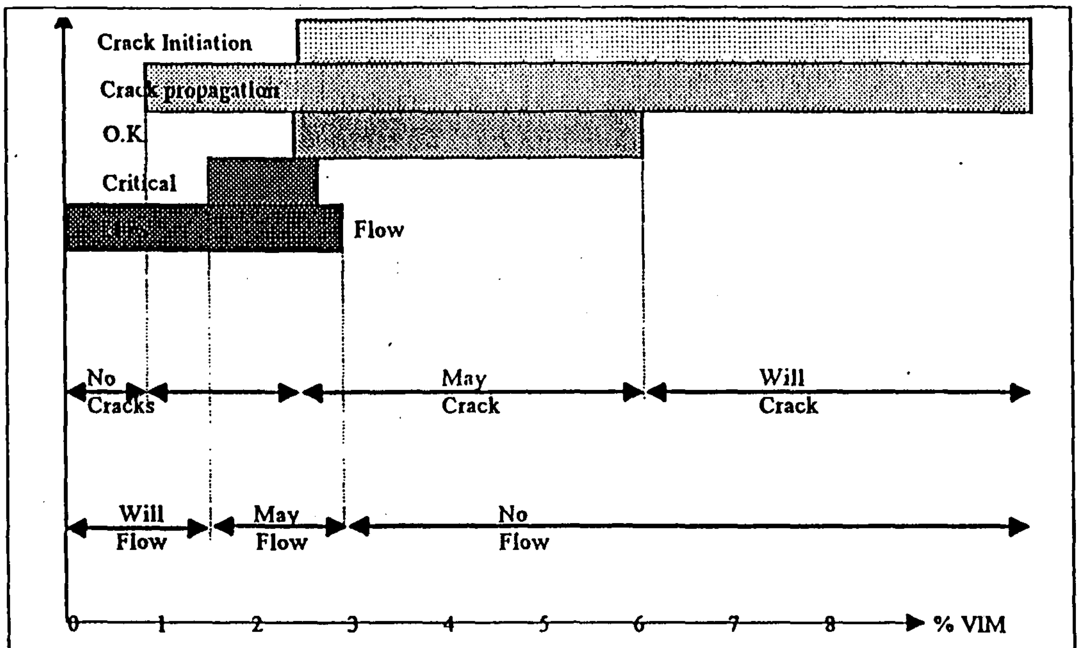


Figure 8 Performance of Asphalt Concrete on "Middle East Road Two"

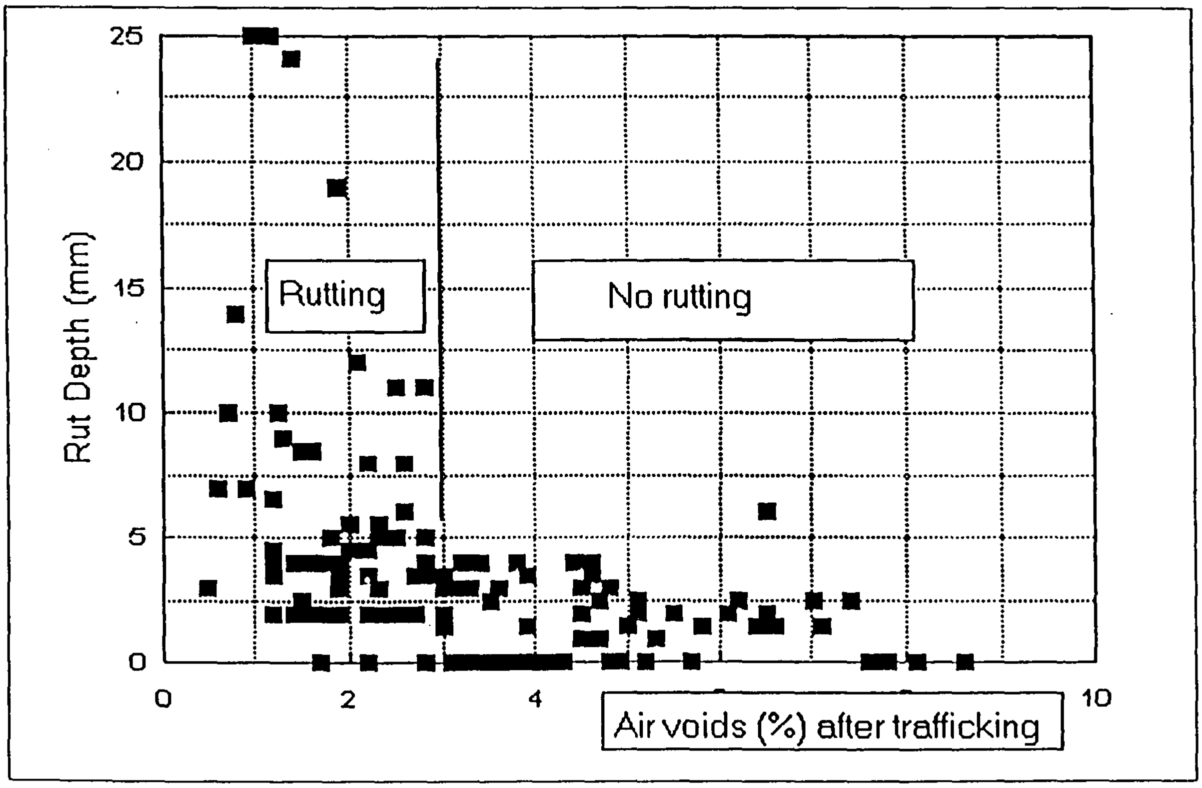


Figure 9 VIM as Determinent of Freedom from Rutting in Malaysia

4. IS IT POSSIBLE TO DESIGN AN ASPHALT MIX THAT HAS ACCEPTABLE RESISTANCE TO BOTH CRACKING AND PLASTIC RUTTING ?

The short answer to this question is, "Yes". We need to look very carefully at the requirements of such a mix.

1. We are considering heavy traffic so the mix will be HRS, SMA or AC and the Marshall design will use 75 blows per face.
2. The Contractor will lay the asphalt at 98% of Marshall density.
3. In the wheel paths, the final density will be 102% to 103% of Marshall density.
4. To avoid plastic rutting we will need to retain at least 3% VIM after secondary compaction by traffic.
5. To avoid premature cracking we will need at least 65% VFB at Marshall density.

It follows that:

A. Since VFB $\geq$ 65% $= 100\dfrac{(Pbe)\times(Gmb)}{(VMA)\times(Gb)}$

the **minimum** acceptable effective bitumen content (Pbe) is $\dfrac{65\times(VMA)\times(Gb)}{100(Gmb)}$

B. Since Refusal VIM $\geq$ 3% = (VMA - Vol. of Bitumen - Loss of VIM under traffic)

the **maximum** acceptable effective bitumen content (Pbe) is (VMA - dVIM - 3) x $\dfrac{Gb}{Gmb}$

The design binder content must be between the minimum and maximum values, calculated from these formulae. The charts in Figure 10 show the relationship between VMA and the possible ranges of design binder contents. They consider two values of dVim, the loss of VIM under traffic. These are 2% and 3%. As you can see, for the normal range of VMA achieved in an AC mix, 13% - 15%, there is no binder content that will satisfy both A. and B. above.

We need at least 16% VMA (preferably 17%) before we can have a design that can avoid premature failure, under severe loading conditions.

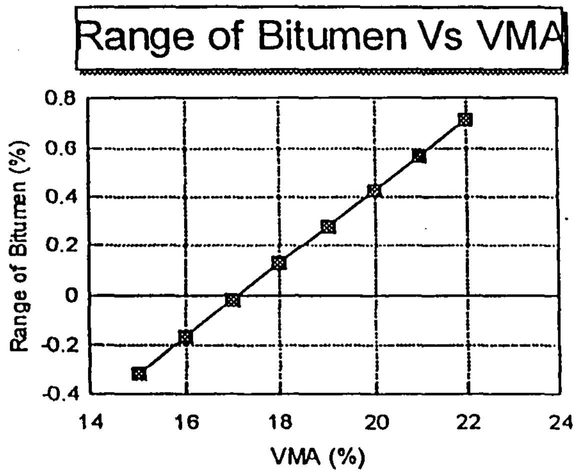
To have a safe margin that will allow for typical variations in mix proportions, we want a VMA of 20% or more. This is not achievable with an asphalt concrete type of mix. It can be achieved with a gap graded mix or with a coarse stone matrix mix such as an SMA grading.

Figure 10 **How Much VMA Do We Need**

Data Set 1

VMA 75	Wbe min	Wbe max	Bit Range
15	4.1	3.8	-0.3
16	4.4	4.2	-0.2
17	4.6	4.6	-0.0
18	4.9	5.0	0.1
19	5.2	5.5	0.3
20	5.5	5.9	0.4
21	5.7	6.3	0.6
22	6.0	6.7	0.7

	Design	Trafficked	
VFB min	0.65	0.80	
dVIM	3		
VIM ref	3		
Gb	1.03	Gmb	2.45



Wbe min =VFBmin*VMA*Gb/Gmb

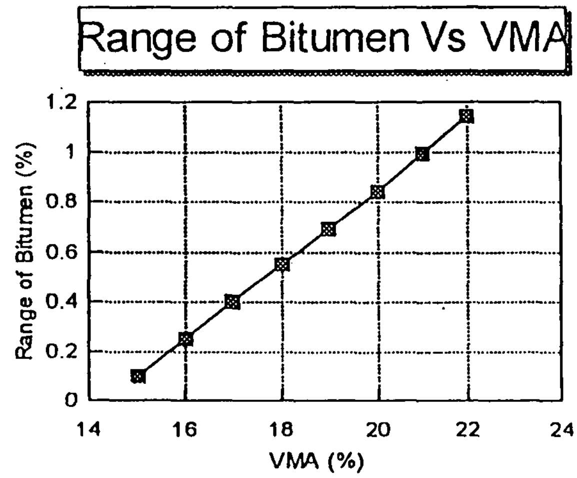
Wbe max =(VMA - dVIM-VIMr min) *Gb/Gmb

Range =Wbe max - Wbe min

Data Set 2

VMA 75	Wbe min	Wbe max	Bit Range
15	4.1	4.2	0.1
16	4.4	4.6	0.3
17	4.6	5.0	0.4
18	4.9	5.5	0.5
19	5.2	5.9	0.7
20	5.5	6.3	0.8
21	5.7	6.7	1.0
22	6.0	7.1	1.1

	Design	Trafficked	
VFB min	0.65	0.78	
dVIM	2		
VIM ref	3		
Gb	1.03	Gmb	2.45



Wbe min =VFBmin*VMA*Gb/Gmb

Wbe max =(VMA - dVIM-VIMr min) *Gb/Gmb

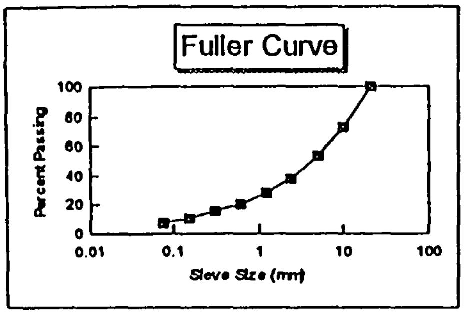
Range =Wbe max - Wbe min

To achieve a practicable range into which we can fit a design binder content with a realistic tolerances, say ± 0.5 %, we need a VMA of 20% to 22%. This is achievable with a gap graded mix, which is what HRS was always meant to be or with a coarse stone matrix mix.

With a continuous grading above the Fuller curve, the best we can do is about 16% VMA.

So, we need to look more closely at grading curves. In particular, we need to see what a Gap Graded Mix really is.

5. UNDERSTANDING GRADING CURVES ESPECIALLY GAP GRADING



Lets start with the Fuller curve. It is named after the man who devised it and it represents the aggregate grading that will have the lowest possible VMA. We can represent it by:

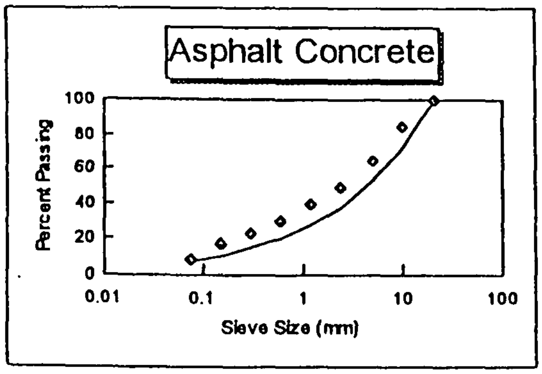
$$\text{Percent Passing} = 100\left(\frac{d}{D}\right)^{0.45}$$

If you are making concrete you want to minimise the volume of cement paste, both to save money and to reduce shrinkage, and the Fuller curve is the ideal grading. In asphalt, where we have a binder that can also act as an excellent lubricant, we must have enough air voids to prevent the mix going plastic. We have to stay as far away from the Fuller curve as we realistically can.

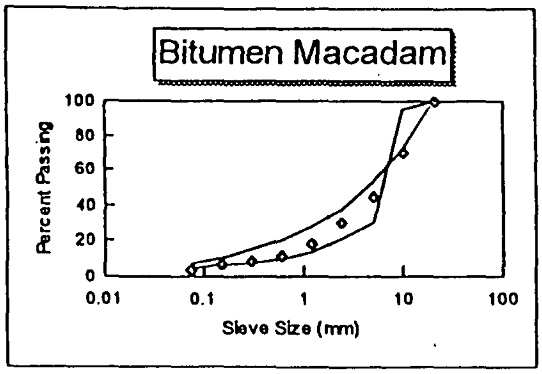
Aggregate that follows the Fuller curve has just enough fines to fill all the spaces left by the coarse aggregate. If the grading is above the Fuller curve it has more fines than the spaces between the coarse particles can hold. It follows that the sand asphalt matrix will be the continuous medium and the stone will be enclosed in this. Let's call this a "sand matrix" material. Below the Fuller curve there are not enough fines to fill all the spaces between the coarse particles. Therefore the coarse aggregate forms a skeleton that is the continuous medium. The fines fill some, but not all, of the spaces between the stones. We can call this a "stone matrix" material.

There are three families of asphalt mixes. They work like this:

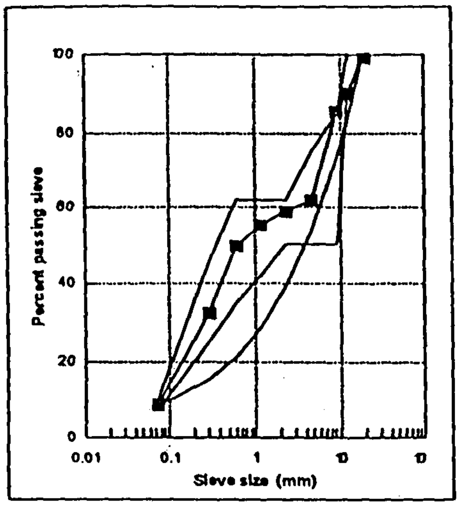
1. You can be above (finer than) the Fuller curve and can use a continuous grading (AC).
2. You can be above (finer than) the Fuller curve and can use a gap grading (HRS).
3. You can be well below the Fuller curve, with gradings that do not provide enough fine material to fill all the gaps between the coarse aggregate particles.



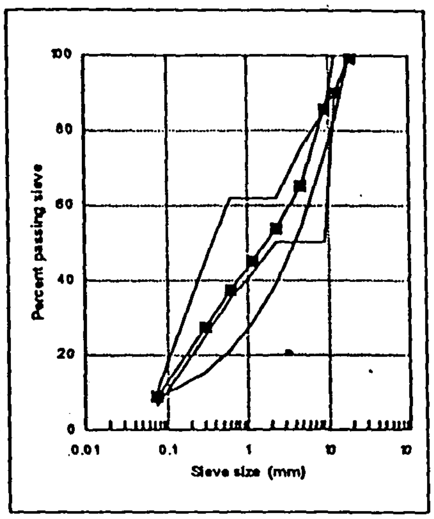
With an AC, the farther you move the grading above the Fuller curve the more VMA you will get, for a given particle shape. If you use more angular fines you will get a higher the VMA but the mix will be "harsher". Moving farther above the Fuller curve means more fines and less stone. That is not ideal for carrying heavy traffic. Also, no matter how little stone you use, you can never get the VMA as high as you really want. This makes AC a relatively sensitive mix. Unintended changes in mix proportions are more likely to cause problems than in other mixes.



Dense Bitumen Macadam is coarser than the Fuller curve. Having fewer fines than spaces for them, guarantees stone to stone contact, unless you flood the mix with too much bitumen. You can get a higher VMA with this sort of mix by reducing the fines even more. If you go too far the mix becomes liable to segregation; to prevent that you have to reduce the maximum size of the stone. Take this process far enough and you no longer have a dense mix at all; you have SMA (as shown in the lowest curve).

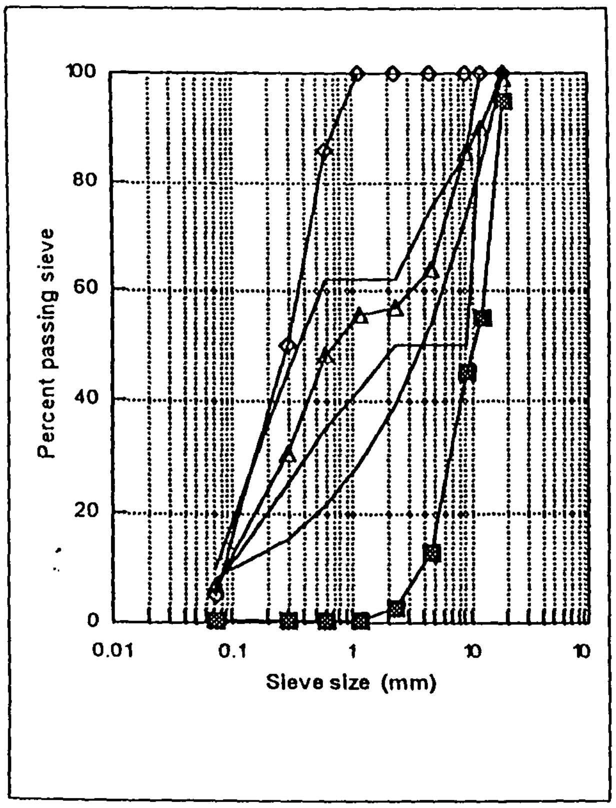


Finally we come to the "gap graded' mixes. As you can see, they lie well above the Fuller curve. ***The essential thing is the "Gap". Between the 600 micron and 2.36 mm sieves there is, and must be, a reduced amount of material.*** This is what gives these mixes the extra VMA. The figure on the left is gap graded; that on the right is not. It doesn't matter that the one on the right also fits inside a dog-legged envelope; it isn't gap graded and it will not have the VMA it needs.



HRS has been blamed for many failures when the culprit was a grading curve like the one on the right. The grading on the right will work if you call it AC and add, maybe, 6% bitumen. If you call it HRS and put in 7.5% bitumen you just have caused another expensive failure.

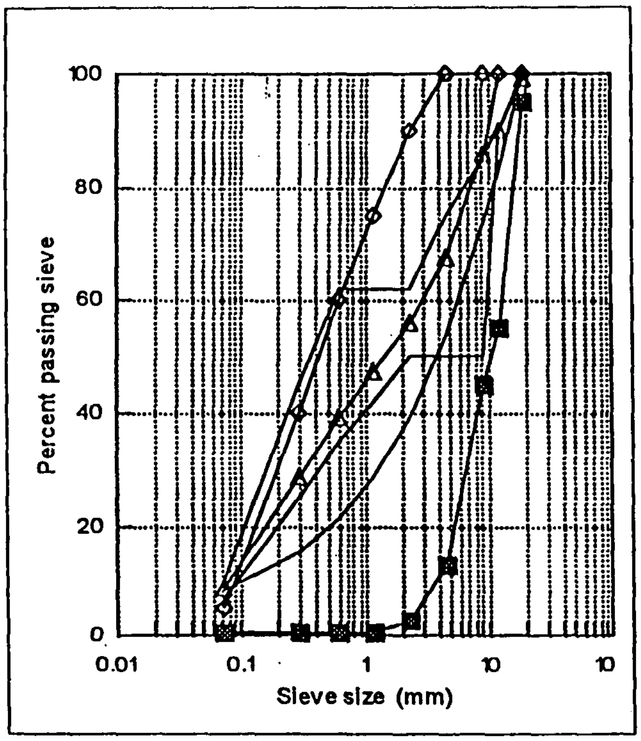
In fact, 7.5% bitumen may be too much even for the mix on the left. That is why compaction to refusal is essential. We have to stop guessing what the correct bitumen content is and start measuring it. ***If you limit your bitumen content so that there are 3% VIM at refusal*** (making sure that you have done the refusal compaction properly) ***your mixes will not fail by plastic rutting.*** If you have enough VMA so that you can also get 65% VFB, at the 75 blow Marshall density, your mix will also be safe from premature cracking.



6. HOW TO MAKE A GAP GRADED MIX

The first thing to understand is that you cannot make a gap graded mix unless you have the right sand to blend with your crushed fines. <u>Just any sand will not do.</u>

It may help to see how gap graded mixes started. The Thames Valley in UK, that runs eastwards from London to the sea, is full of sand deposits. The uppermost line represents a fairly typical grading. Note how little material there is between 600 microns and 2.36 mm, less than 15%. The lowest line represents a crushed stone or a Thames Valley gravel. If you combine the two gradings in approximately equal proportions you will have a gap graded mix. You couldn't get a continuous mix if you tried because there is a gap between the top end of the sand and the bottom end of the stone. These mixes are specified in BS594. (1992)



Now look at the grading of this sand, that is typical of many Javanese river deposits. 40% of its content is coarser than 600 microns. It is impossible to find a combination of this sand and this coarse aggregate that will give a gap grading because of the overlap between the sand and the stone.

Java has sand deposits along the north coast that were once beaches. In a previous geological age, the sea washed out the silt. Then, as plate tectonic action slowly lifted the island of Java and caused the sea to recede, the rain had a million years (or two or three) to wash out the salt. Some of these deposits are suitably graded for making HRS. So Indonesia does have the sand it needs, just where it most needs it, along the Northern Corridor route.

In UK we carry almost all our heaviest traffic on gap graded mixes. Indonesia has a section of road between Cirebon and Losari that has been doing the same thing for the past eight years. There are no cracks and the mix is just about to lose its voids and go plastic. However, it has survived for longer that all the AC mixes around it. If we design similar mixes, using compaction to refusal to make sure they never go plastic, they should still be there in ten or twenty years time.

7. SO WHAT DO WE DO?

7.1 Gap Graded Mixes

This is probably the best answer, if you have a suitable sand available, because:

1. gap graded mixes are less sensitive than continuously graded mixes.
2. They are easier to compact.
3. They are more resistant to oxidation/embrittlement/cracking.
4. If you ensure 3% VIM at refusal, they are very resistant to plastic flow.

Blend 20% to 40% of fine sand with 60% to 80% of coarse and fine graded crushed stone. A Mix Design Spreadsheet is a useful tool to help you get the blend right. Such a spreadsheet is not essential; highway engineers were making good asphalt mixes long before there were computers. However, computers are now available; so, why not make mix design easier and more reliable?

Remember the 3 key issues:

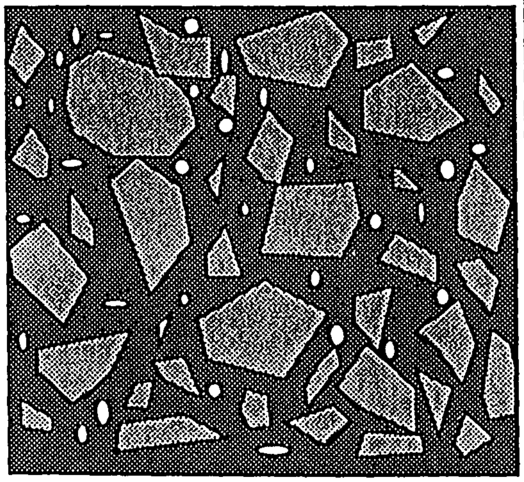
A> It is not HRS unless it is Gap Graded
B> For heavy traffic, you must have 3% VIM at refusal density.
C> For durability, you must have 65% VFB at Marshall density.

7.2 Continuously Graded Mixes

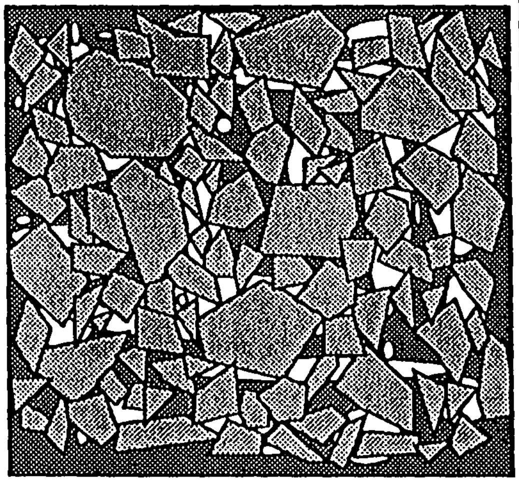
If you do not have the right sand to make a gap graded mix, design one that is continuously graded ***as far from the Fuller curve as you can reasonably get.***

Again, use of a Mix Design Spreadsheet will make life quicker and will probably increase the probability of getting the first trial mix to work.

1. Decide whether you want to make a "stone matrix" material, below the Fuller curve, or a "sand matrix" material, above the Fuller curve.
2. Provided you can get 16% to 17% VMA, either of these will work.
3. Above all else, if you are designing for heavy traffic, make sure you have 3% VIM at refusal density and 65% VFB at 75 blow Marshall density.



Above the Fuller Curve . . sand matrix



Below the Fuller Curve . . stone matrix

7.3 Split Mastic Asphalt

It would probably be more helpful if we thought of SMA as meaning Stone Mastic Asphalt. As we said in Section 5, above, SMA is an extreme stone matrix material that can have 30% , or even less, passing the 4.75 mm sieve. Using so little fines would tend to allow segregation of the larger sizes of coarse aggregate. To prevent this, the maximum stone size is limited to between 8 and 12.5 mm.

The sand content is so low that the mix is no longer dense graded. This means that air has access to the surface of most aggregate particles. Oxidation and embrittlement of the binder, followed by cracking, are obviously potential problems. What saves the mix from rapid failure is that the binder films are very thick.

In the current specifications, cellulose fibres are added to reduce the tendency for the binder to drain from the mix. An alternative version of the specification, Butonite Mastic Asphalt, does not use any cellulose fibres. Rubberised bitumen would probably work even better.

These specifications ought to include VIM at refusal density to ensure the mixes can carry heavy traffic without fear of plastic rutting. We don't yet know what VIM they need at refusal density. Until we know better, 3% is a sensible working figure.

8. HOW DO WE REPAIR ROADS THAT HAVE FAILED BY PLASTIC RUTTING?

DO NOT OVERLAY !!!!!

This is going to be difficult but it is essential. If you lay sound asphalt on top of plastic material it will fail very quickly.

SO WHAT DO WE DO ?

1. Leave it as long as you reasonably can.
2. Do any local repairs that are necessary to keep traffic flowing.
3. If ruts become large enough to interfere with traffic, yet the road remains structurally sound, plane off the ruts.
4. ***When you have to resurface, plane right back to sound material and then rebuild from there.***

WE ARE GOING TO NEED

1. More planing machines
2. To learn how to recycle successfully and economically:
   (a) in-situ,
   (b) using plant based hot mixes (needs modified plant),
   (c) using cold mixes.

RECYCLING SHOULD BE A PROFITABLE, GROWTH INDUSTRY FOR THE NEXT TEN YEARS IN INDONESIA . . .
and anywhere else that has a lot of plastic rutting or a lot of cracking.

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