UTILISING RECYCLED MATERIAL STABILISED WITH BITUMEN TO REHABILITATE A MAJOR HIGHWAY WITHIN STRINGENT TIME CONSTRAINTS

Dave Collings
UCD Technology / Loudon International

Kim Jenkins
Stellenbosch University

Everton Souza
EcoRodovias (Brazil)

Abstract— The Ayrton Senna Highway feeding into São Paulo City from the north-east ranks as one of the most heavily-trafficked links in Brazil's road network. This highway was originally constructed in the late 1970s and now carries more than 200 000 vehicles per day (± 15% heavy vehicles). A concession agreement with the State Highway Authority was signed in 2009 transferring maintenance responsibilities to EcoRodovias, a consortium of large construction companies.

EcoRodovias soon learned that meeting their concession obligations required continuous interventions, mostly carried out during a 6-hour working window at night. Operations that milled and replaced 100mm of asphalt often had to be repeated within six months as a consequence of problems deeper in the pavement structure. Clearly, a more appropriate solution was required, one that was affordable and could be completed within 6 hours.

This paper describes the methodology followed to find such a solution. Technology developed in South Africa was adopted to recycle material from the existing pavement, stabilised with bitumen to provide a balanced pavement that could withstand early trafficking. By the end of 2014, more than 100km of truck lanes had been rehabilitated using this technology and ongoing monitoring of sections treated in 2011 confirmed performance predictions.

Keywords—cold recycling; foamed bitumen; in plant mixing; heavy duty pavement; pavement rehabilitation

I. INTRODUCTION

The Ayrton Senna Highway is one of the most heavily trafficked links in Brazil's road network, heading eastwards from the city of São Paulo. It provides access from the city to Guarulhos International Airport, the industrial region of Paraiba Valley, the popular tourist attraction of Campos do Jordão and eventually merges with the Presidente Dutra Highway that continues eastwards to Rio de Janeiro. It is a primary State Highway (SP-070), starting at the São Paulo city limit as an 8-lane divided facility with one carriageway on each side of the Tietê River, heading north-east for some 7km to the Guarulhos Interchange. The carriageways then merge, separated by a wide grassed median. At the São Miguel Interchange, a further 3km east, the number of lanes reduces to 6 and, after another 12km, down to 4 lanes at the Itaquaquecetuba Interchange. The Ayrton Senna Highway then continues east-wards for a further 27km to the BR 116 Inter-change where the name changes to the Carvalho Pinto Highway. The general alignment of the western 25km section is shown in Figure 1.

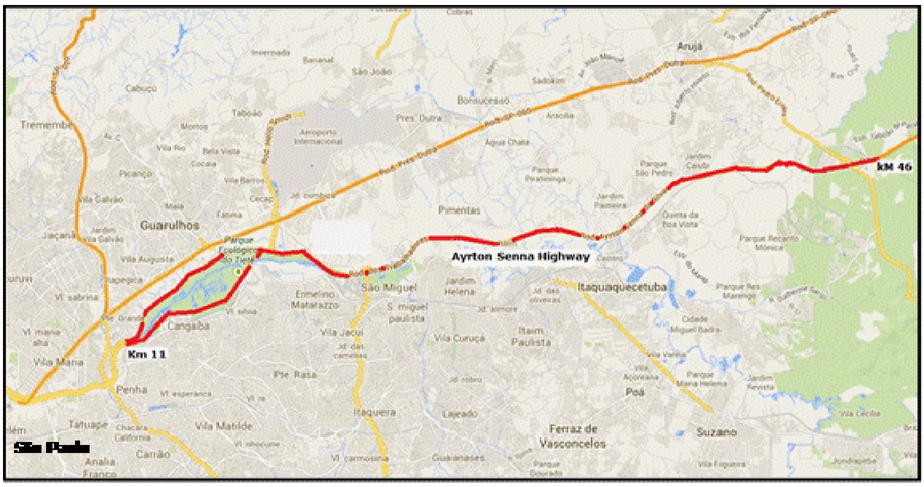


Fig 1. Location of the western 25km of the Ayrton Senna Highway

This paper is concerned with the rehabilitation of 600m of the slow lane on the westbound carriageway between the Guarulhos Interchange and the city of São Paulo (the 8-lane section). An innovative "non-traditional" construction method was applied. The design and construction work was carried out as a "Trial Section" during the second half of 2011 and, as a consequence of the success achieved, this rehabilitation method was adopted for a further 100km of truck lanes that were completed by the end of 2014.

A. Background

The Ayrton Senna Highway was originally constructed in the 1970s to relieve traffic congestion on the adjacent Presidente Dutra Highway (the primary link between the cities of São Paulo and Rio de Janeiro). The region has a monsoon-influenced "humid subtropical climate" according to the Köppen classification[11]. Sunshine is plentiful and the mean annual rainfall is 1454mm. Rain can be expected every month of the year with heavy thundershowers being common in summer. The average daily temperature ranges from a high of 30°C in February reducing to 20°C in the winter months. Temperatures below 10°C are seldom experienced.

In June 2009 the State Road Authorities entered into a concession agreement with Group EcoRodovias for the "Ayrton Senna / Cavalho Pinto" corridor, a total distance of 134.9km. A new company "EcoPistas" was formed to implement the concession agreement with two large construction firms participating (CR Almeida from Brazil and Impregilo from Italy). Two of the primary requirements of the concession agreement were ride quality and surface integrity. These proved to be a major concern as the cost of maintenance spiralled ever upwards as construction crews wrestled to hold the pavement together. Figure 2 shows an example of the poor surface condition affecting a portion of the slow lane towards the Sao Paulo end of the highway in July 2011, two years after the concession agreement came into force.

Fig 2. Condition of slow lane (July 2011)

Milling out and replacing 100mm of asphalt material provided only temporary relief before the same pattern of distress reappeared, suggesting that the cause lay deeper in the pavement. This presented a major challenge because any construction work on this section of highway had to be carried out at night within a limited 6-hour working window. A partial closure (maximum of 2 lanes) was permitted only after 23:00 and all work completed by 05:00 when the full width of carriageway had to be opened to traffic. Failure to meet these requirements attracted severe penalties since the closure of a single lane during daylight hours caused a major disruption with traffic backing up for several kilometres, threatening to choke the city of São Paulo and the busy International Airport.

B. The Pavement Structure

Figure 3 shows the original pavement design for the western section of highway between the Guarulhos Interchange and the São Paulo city limit that was constructed on top of 2m high levees (embankments), one carriageway on each side of the Tietê River. The specification for the material used to construct these embankments required a minimum CBR value of 8%. The design called for 800mm of cover on top of the embankment, comprised of 100mm of asphalt (combined base and surfacing layers), 400mm of graded crushed stone (of which the upper 200mm was to be stabilised with cement) and 300mm of selected gravel with a CBR in excess of 15%. The structural capacity rating for this pavement was estimated as ± 50 million equivalent standard (80kN) axle loads (ESALs).

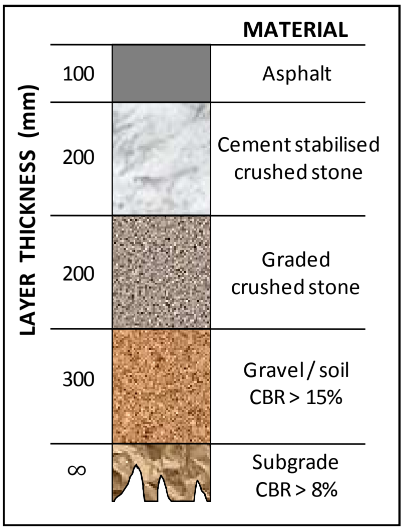


Fig 3. Original pavement design

The block-crack pattern and white powdery material pumped out through the cracks suggested that failure of the cement stabilised layer underlying the asphalt was the likely cause of distress. It was reported that an application rate of cement of some 6% (by mass) had been used to stabilise the 200mm thick layer of crushed stone. Such a high dosage of cement would have created a relatively strong and rigid layer that would have cracked and broken down into a series of blocks under the repeated loading of heavy traffic.

II. REHABILITATION OPTIONS

The key factors limiting rehabilitation options was the restricted time available for construction work coupled with the requirement to open the completed work to heavy truck traffic immediately after construction. Although the allowed closure period was 6 hours (between 23:00 and 05:00), the actual time available for construction work was only 5 hours due to the time required to effect the closure and then to clean up and remove traffic accommodation barriers and delineators once the work was complete.

It was clear that the cement stabilised layer was the cause of distress and therefore had to be removed and replaced. Such an operation would expose the underlying crushed stone layer, allowing any defects to be identified and addressed. However, the effective 5-hour working window posed a serious challenge for:

- removing the upper 300mm portion of the existing pavement (in one lane only);
- determining the adequacy of the underlying support and, where defects were identified, effecting repairs;
- replacing (backfilling) the excavation using a material with sufficient early strength; and
- matching levels on both sides of the lane with a suitable wearing surface for immediate trafficking.

In search of a solution, EcoPistas approached Fremix who, at that time, was successfully using reclaimed asphalt (RA) material treated in plant with foamed bitumen to rehabilitate / upgrade urban avenues in São Paulo. The main attraction of this construction process was that foamed bitumen stabilised material was a "cold material" that gained sufficient strength to withstand traffic loading as soon as it had been compacted. Additional perceived advantages included utilising material recovered from the existing pavement (thereby reducing both the cost and environmental impact by recycling) as well as the ability to hold the treated material in stockpile for extended periods, thereby separating the milling and backfilling operations.

Although an attractive option, foamed bitumen stabilisation was relatively new to the Brazilian construction industry. In addition, some premature failures had occurred on previous projects where this technology had been incorrectly applied using in situ recycling machines. However, with more than two years of experience working with RA material treated in plant (as opposed to in situ recycling), Fremix were of the opinion that this technology was appropriate. To gain EcoPistas' confidence and ensure that the design and construction work was undertaken correctly, Fremix requested assistance from Loudon International (a South African firm specialising in foamed bitumen stabilisation technology). In response, EcoPistas agreed to consider carrying out a 600m Trial Section on the slow lane of the most-heavily trafficked section (the westbound carriageway between the Guarulhos Interchange and the São Paulo city limits, one of the most distressed sections of the highway).

III. DESIGN CONSIDERATIONS

A. Structural Capacity Requirement

Brazil has a legal axle load limit of 8.2 tons. EcoPistas estimated an annual average daily traffic (AADT) figure of approximately 200 000 vehicles per day for this specific section of highway, 15% of which was comprised of trucks and other heavy vehicles, implying that some 15 000 heavy vehicles travelled in each direction every day. Assuming that the average loading from heavy vehicles is 3.0 equivalent standard (80kN) axle loads (ESALs), each carriageway carried some 45 000 ESALs per day, or about 15 million ESALs per year. However, trucks are permitted to travel only in the outer two lanes on this highway and, based on the assumption that two thirds of the trucks keep to the slow lane (especially the slower moving nine-axle interlinks), the annual loading carried by the slow lane was estimated to be in the order of 10 million ESALs. Thus, a 10-year service life called for a structural capacity in excess of 10^8 ESALs.

B. Falling Weight Deflectormeter (FWD) measurements

During May 2010, Messrs Dynatest (Brazil) had carried out a FWD survey of the entire length of the highway. The measurements for maximum deflection recorded at 40m intervals in the outer wheel path of the slow lane over the designated Trial Section are shown in Figure 4. Plots of the deflection bowls for the maximum and minimum deflections recorded over this section, together with the mean and 85[th] percentile are shown in Figure 5.

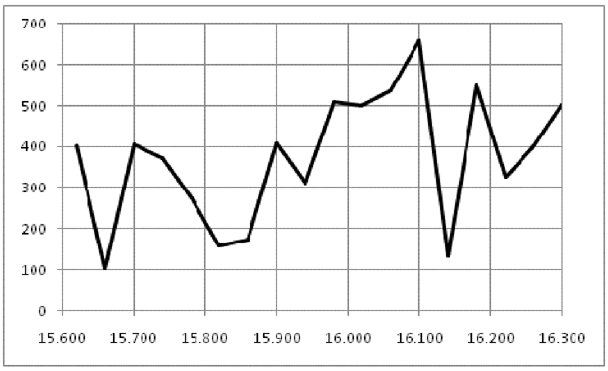


Fig 4. FWD measurements for maximum deflection

Figure 4 highlights the non-uniform response of this section of highway to an applied load.

C. Detailed investigations

During August and September 2011, the following investigations were undertaken on the designated Trial Section:

- excavation of 4 test pits by carefully removing each layer, one at a time, to a depth of 600mm;
- LWD deflection measurements on the surface of each layer exposed in the test pits;
- laboratory testing of samples taken from each layer encountered in each test pit; and
- extraction of cores to determine variations in the thickness of asphalt. (This exercise proved challenging due to poor specimen recovery.)

All field work was carried out by Messrs JBA Engenharia e Consultoria Ltda, working at night, under lights, during the allowable 6-hour closure period.

1) Test pits and tests conducted whilst excavating the test pits

Four test pits were excavated in the outer wheel path at the following locations, selected in accordance with measured deflections:

- TP 1. km 16+105 (isolated low deflection)
- TP 2. km 16+210 (high deflection zone)
- TP 3. km 15+945 (average deflection)
- TP 4. km 15+795 (low deflection zone)

The thickness of each layer encountered was measured and samples taken for laboratory testing. In addition, since there was no Dynamic Cone Penetrometer (DCP) available, an attempt was made to measure the resilient modulus at the surface of each exposed layer using a Light Weight Deflectometer (LWD).

Table 1 summarises the information obtained from the four test pits. (Note: Cells marked “N/M” indicate that the value was not measured; “N/A” indicates that the value was measured but is not available.)

The modulus values obtained from the LWD device made little sense, especially in light of the respective deflection measurements (the highest modulus measured on top of the exposed cement stabilised layer coincided with the highest deflection measurement (TP 2)). Several explanations for this phenomenon were postulated, the most reasonable being the inappropriateness of using the device inside a relatively small excavation. These results were therefore ignored.

2)Core recovery

As shown in Table 2, six of the ten cores were extracted over the 600m length of the Trial Section failed to recover the lower asphalt layer and only partial recovery in a further two. The two cores that did achieve full recovery indicated an asphalt thickness in excess of 100mm (127mm at km 15+800 R and 122mm at km 15+875 L). This was confirmed by the measurements taken at the four test pits where the average asphalt thickness was 126mm.

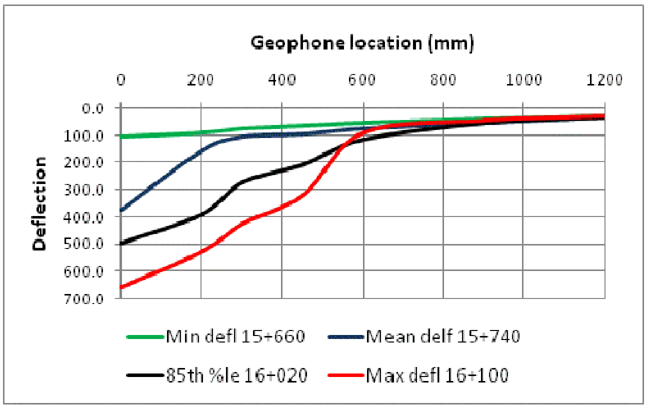


Fig. 5. Selected deflection bowls

Table 1. Summary of Test Pit information (layer thickness measurements (mm) and LWD moduli (MPa))													
Test Pit		B Beam Deflect^n	HMA surfacing		HMA Binder		Cement Stabilised s/base		Crushed stone subbase		Selected subgrade		Subgrade
No	km	mm	Thick	E-mod	Thick	E-mod	Thick	E-mod	Thick	E-mod	Thick	E-mod	E-mod
TP 1	16+105	0.2	45	118	80	49	200	60	100	100	185	15	31
TP 2	16+210	0.9	65	18	70	74	200	101	150	41	100	25	N/M
TP 3	15+945	0.4	50	82	80	N/M	200	63	150	40	100	40	50
TP 4	15+795	0.3	45	N/M	70	N/M	200	N/A	150	118	140	40	20

Table 2. Measurements (mm) of cores extracted from the asphalt layers									
15+650	15+650	15+725	15+800	15+875	15+950	16+100	16+175	16+250	16+250
Left	Right	Left	Right	Left	Right	Right	Left	Left	Right
51	52	52	48	49	50	30	58	70	70
0	0	0	79	73	0	33	0	29	0

3)Laboratory test results

Primarily for classification purposes, standard laboratory tests were carried out on the unbound materials sampled from the test pits. Results for each test pit are captured in Table 3.

Table 3. Summary of laboratory test results for layers of unbound material

Layer	Property		TP 1. km 16+102	TP 2. km 16+210	TP 3. km 15+945	TP 4. km 15+795
Lower subbase (crushed stone)	CBR @ 100% density*		120	114	117	118
	Swell (%)		0	0	0	0
	MDD** (kg/m^3)		2173	2160	2187	2166
	OMC** (%)		9.1	7.9	7.6	7.3
	Grading (% passing)	25.4mm	82	92	72	90
		13.2mm	64	67	52	71
		2mm	27	19	21	27
		0.075mm	7.4	3.5	5.4	5.1
Selected subgrade (silty sand)	CBR @ 93% density*		22	23	21	20
	Swell (%)		0.27	0.36	0.35	0.26
	MDD** (kg/m^3)		1985	1940	1893	1993
	OMC** (%)		9.1	7.5	9.5	9.2
	Field MC (%)		10.8	10.8	12.4	8.15
Subgrade (clayey sand)	CBR @ 93% density*		-	13	12	14
	Swell (%)		-	0.4	0.42	0.39
	MDD** (kg/m^3)		-	1512	1517	1602
	OMC** (%)		-	24.5	24.6	20.9
	Field MC (%)		-	26.2	28.0	-

* Density of specimen tested (modified AASHTO T-180 compaction effort)
** Moisture / density relationship (MDD: maximum dry density; OMC: optimum moisture content)

Since the proposed rehabilitation process would result in the unbound materials remaining in place, the following observations were deemed relevant:

- with the exception of TP 4 (intact pavement with low deflection), in situ moisture contents (Field MC) for the selected subgrade layer and subgrade were all in excess of the OMC;
- the grading of the crushed stone material in TP 2 (severely cracked with high deflection) reflected a loss of fines (< 0.075mm fraction), presumably as a result of pumping;
- the relatively low CBR swell values determined for the subgrade material indicated that the clay content was low, suggesting that the material was a silty sand rather than a clayey sand

4)Summary of investigations, surveys and tests

Data collected from all the investigations, surveys and tests was used to compile the "Representative Pavement" shown in Figure 6.

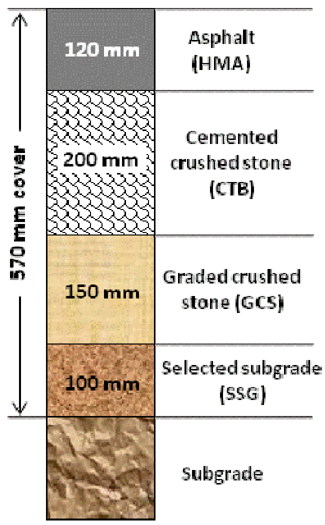


Fig 6. Representative pavement

The main difference between the actual structure and that previously assumed (refer Figure 4) was a shortfall of 240mm in the combined thickness of all pavement layers above the subgrade (total cover). The thickness of both the graded crushed stone (GCS) and the selected subgrade (SSG) layers was half that previously assumed (250mm compared with the assumed as-built thickness of 500mm). It would appear that the SSG layer was constructed as a 100mm thick "capping layer" on top of the silty sandy subgrade material in order to provide access for construction traffic. Such a reduction in cover thickness (30%) impacts significantly on the carrying capacity of a pavement.

However, laboratory test results indicated that the quality of material used to construct the various pavement layers was in general accordance with the original specification.
Tests on disturbed samples of GCS, SSG and subgrade material showed the material could be classified in accordance with the South African system [9] as:

- GCS layer: G3 (shear properties: C = 38kPa; φ = 46°);
- SSG layer: G7 (15 < CBR < 25 at 93% of mod AASHTO density); and
- Subgrade: G8 (10 < CBR < 15 at field density)

Although these classifications were adopted in the following pavement evaluation exercise, they may be considered conservative for the SSG and subgrade materials. In the absence of DCP data, it is difficult to assess the in situ condition of these materials, especially density. More than 40 years of heavy trafficking under varying moisture regimes would undoubtedly have resulted in an increase in density (consolidation) in all unbound materials in the pavement. The results of CBR tests carried out on disturbed samples are therefore likely be conservative.

D. Pavement Design / Evaluation

Based on the "Representative Pavement" shown in Figure 6, the total thickness of material to be removed would be 320mm (120mm asphalt and 200mm CTB material). Backfill would then be undertaken by constructing a new 270mm thick base using the foamed bitumen treated RA (constructed in 2 x 135mm thick layers), followed by 50mm of asphalt wearing course.

Pavement analyses were undertaken using the Linear Elastic Layer module in the Rubicon Toolbox suite of programmes. The structure shown on the right side of Figure 7 was modelled, using several iterations to determine the effect on structural capacity by varying "T", the thickness of the BSM layer. Unbound granular layers in the lower portion of the pavement were evaluated in accordance with the South African Mechanistic Pavement Design Method (SAPDM) [10]. The Limiting Deviator Shear Stress Ratio principle proposed by Jenkins [6] was applied to the BSM layer. Being relatively thin and flexible (and therefore not prone to fatigue cracking), the asphalt surfacing layer was included in the model (with an assumed modulus of 3000MPa for polymer modified binder), but not evaluated as a structural member of the pavement. With the mean annual rainfall figure for São Paulo approaching 1500mm, this region of Brazil classifies as "wet" [8]. The relevant material properties for wet conditions were therefore adopted for modelling.

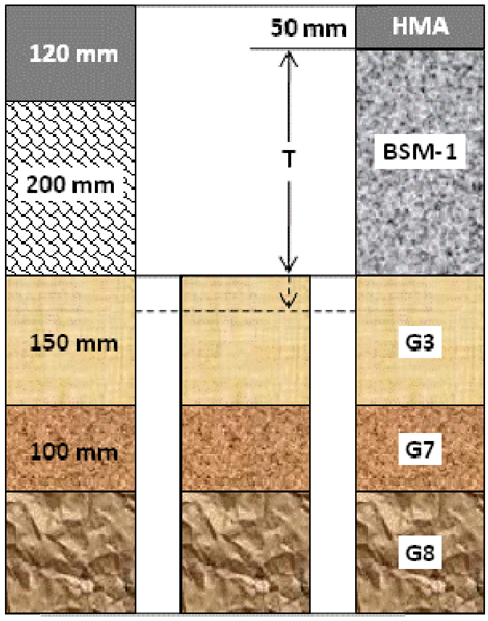


Fig 7. Rehabilitation concept

The stiffness (resilient modulus, M_R) for each layer was estimated in accordance with the modular ratio principle described in TG2 (2009) [1], commencing with the subgrade and progressing upwards, layer by layer, thereby ensuring pavement balance. Moduli limits recommended for granular materials in TRH 4 [8] and SAPDM [10] were applied.

However, the modulus limit proposed in TG2 for layers of BSM-1 material was not applied since it relates specifically to the empirical Pavement Number design method. Such non-continuously bound materials are stress dependent and can therefore develop stiffness values in excess of 1000MPa. For the modelling exercise that considered the new BSM base as a single thick layer (as opposed to multiple sub-layers), stress dependency was ignored and a modular ratio of 3 assumed. The input parameters and results of the modelling exercise for a 260mm thick BSM layer are summarised in Table 4.

Table 4. Input parameters and evaluation criteria for mechanistic analysis (Rubicon Toolbox)								
Layer	Thickness (mm)	Modulus (MPa)	Poisson's ratio	Cohesion (kPa)	Friction angle (°)	Evaluation criterion	Value	Capacity 10^6 ESALs
HMA	50	3000	0.4	-	-	Not evaluated		-
BSM-1	270	660	0.35	250	40	Shear stress ratio	0.262	>100
GCS (G3)	150	220	0.35	26	42	Shear safety factor	1.92	>100
SSG (G7)	100	120	0.35	22	35	Shear safety factor	2.49	>100
S'grade (G8)	∞	70	0.35	-	-	Vertical strain (µm)	307	>100

This exercise showed that a 270mm thick layer of BSM-1 quality material would achieve a structural capacity in excess of 100 million ESALs (i.e. an estimated service life of 10 years).

Increasing the thickness of the BSM layer to 300mm (which reduced the thickness of the underlying G3 crushed stone layer to 110mm) reduces the deviator stress ratio to 2.44 and the vertical subgrade strain to 294µm. In recognising that the upper portion of the crushed stone layer (beneath the cement stabilised layer) was probably damaged by pumping and/or disturbed by movement (rocking) of the overlying CTB slabs, the decision was taken to remove 350mm of existing pavement (that would effectively increase the thickness of the BSM layer to 300mm).

The following design recommendations were made:

1. Mill off and remove all asphalt, CTB and crushed stone material to a depth of 350mm below existing surface elevations. In addition to ensuring the removal of all troublesome CTB material, milling to such a depth would take off the upper ± 30mm of the crushed stone layer.
2. To ensure adequate structural support, the surface exposed by milling would have to be checked for poor material and/or instability. Where detected, the offending material would then have to be removed and backfilled with suitable material.
3. Representative samples of RA taken from existing stockpiles adjacent to the highway would be tested. If suitable for treatment with foamed bitumen, comprehensive mix designs (with and without blend material) would then be carried out to determine the ideal mix (RA, blend material, bitumen and active filler) as well as the relevant shear properties (cohesion and angle of internal friction).
4. The BSM would then be mixed in a suitable off-site mixing plant and placed in temporary (covered) stockpile. Strict quality controls would be applied to all input materials. In addition, the moisture content of the treated material would have to be carefully monitored to ensure that it remained between 70% and 75% of OMC prior to transporting to site (to assist with compaction and avoid any heaving under traffic).
5. The first BSM layer, 200mm thick, would then be constructed on the crushed stone layer and compacted to maximum density using a heavy padfoot roller. Since they create a "slick finish" that would prevent the two layers adhering to each other, pneumatic tyred rollers would be kept off the first layer.
6. Construction of the second BSM layer, 130mm thick, would then follow immediately, shaped to 20mm below existing surface levels, compacted to maximum density and finished off with pneumatic tyred rollers.
7. A temporary 20mm thick gap-grade asphalt surfacing layer would then be applied. Such a surfacing would provide for at least 3 months of trafficking, allowing the moisture content on the BSM to reduce to equilibrium levels.
8. After about 3 months, the uppermost 50mm of the pavement (temporary surfacing and BSM) would be milled off and replaced with a robust asphalt surfacing mix.

In accordance with the above, it was estimated that a 200m section could be constructed within the 5-hour working window.

IV. STABILISATION MIX DESIGNS

All testing was carried out in JBA's laboratory facilities in São Paulo. Sampling the stockpiles of RA together with all preliminary tests were completed before the arrival of a technician from BSM Laboratories (Pty) Ltd in South Africa who spent two weeks in the laboratory assisting with the foamed bitumen stabilisation designs.

Standard laboratory procedures were followed [(1)] to determine the need for active filler (with a preference shown for cement) and indicate the optimum amount of bitumen addition required to achieve adequate stabilisation.

Two mixes were tested, one using 100% RA material, the other a blend of RA and crusher dust. Before testing, all RA material was passed through an impact crusher to break down oversized lumps. Table 5 summarised the results of ITS tests carried out to indicate the optimum addition of foamed bitumen.

Table 5. Results of tests to determine the effect of different application rates of foamed bitumen								
Material:	**100% processed RA material**				**Blend of 85% RA and 15% crusher dust**			
Bitumen addition (%)	1.75	2.0	2.25	2.5	1.75	2.0	2.25	2.5
ITS_{DRY} (kPa)	378	387	396	400	463	432	535	592
ITS_{WET} (kPa)	315	349	347	369	356	346	449	517
TSR (%)	83	90	88	92	77	80	84	87
Notes. 1. All application rate expressed as a percentage by mass of the material being treated								
2. All mixes included 1% cement as active filler								

All mixes exceeded the minimum requirements for BSM-1 classification. In addition, the Tensile Strength Retained (TSR) value achieved for all mixes was in excess of 75%, indicating a low level of moisture sensitivity (a by-product of using RA material where all the particles were previously coated with bitumen).

Consequently, based on these results, the following decisions were taken:

- The 100% processed RA mix would be more than adequate for the construction the lower 200mm thick BSM layer where anticipated stress levels were relatively low. Conservative application rates of 2.0% foamed bitumen and 1% cement were recommended.
- The 85% RA / 15% crusher dust blend was recommended for the upper 130mm thick layer where stress levels near the surface would be significant higher. Blending with crusher dust would ensure that a non-continuously bound material would be achieved and the benefits of stress dependency could be fully realised. Similar to the 100% processed RA mix, conservative application rates of 2.2% foamed bitumen and 1% cement were recommended.

The confidence factor [1] achieved by conducting the above tests was less than 50% which was deemed insufficient for a project of such importance. To increase confidence, additional tests for both materials (the 100% processed RA and the blend with crusher dust) were carried out on mixes with the recommended application rates. Large specimens (150mm φ x 95mm high) were manufactured for additional ITS tests and 150mm φ x 300mm high specimens for triaxial testing. All these specimens were cured at their equilibrium moisture content. [12]

The ITS test results for the 95mm high specimens are shown in Table 6.

Table 6. ITS test results at equilibrium moisture content		
Material:	**100% RA**	**85% RA / 15% dust**
Bitumen addition (%)	2.0%	2.2%
ITS_{EQUIL} (kPa)	337	368
ITS_{SOAK} (kPa)	312	335

Triaxial tests were to be carried out on the 300mm high specimens using the research facilities at the University of São Paulo. Unfortunately the equipment was not available when the tests were scheduled to be carried out. As a result, the shear properties were not determined and therefore had to be estimated, based on the ITS test results and experience from working with similar materials in South Africa where triaxial test results were available. These estimated properties are shown in Table 7.

Table 7. Estimated shear properties based on ITS test results		
Material:	**100% processed RA**	**Blend with 15% dust**
Cohesion (kPa)	300	280
Friction angle (%)	38	40

V. THE CONSTRUCTION PROCESS

Shortly after the design recommendations [7] were submitted, EcoPistas took the decision to proceed with the Trial Section. Detailed planning followed and construction was scheduled to commence on the first night by tackling the 150m section closest to the São Paulo end of the designated trial section where deflection measurements were low. If all went well, additional 150m sections would be completed each night, leaving the last section to be tackled over a weekend when work was permitted during daylight hours. (Problems with underlying instability were anticipated on this last section where the highest deflection measurements had been recorded.) Figure 8 summarises the 7-step construction process that was adopted.

Work commenced at the RA stockpile site (located adjacent to the highway, some 2km from the Trial Section) during the morning of Wednesday 16th November 2011 by mixing the RA that had previously been processed through an impact crusher. Material both the 100% RA mix and the blend of RA / crusher dust were treated with the required application rates of foamed bitumen, active filler and water in a Wirtgen KMA 220 plant established at the RA stockpile site. Sufficient material for the nightshift was mixed for both layers and placed in stockpile, covered by tarpaulins. At 23:00, the outer two lanes of the westbound carriageway were closed off over a 300m length spanning the 150m to be rehabilitated and work commenced following the sequence shown in Figure 8.

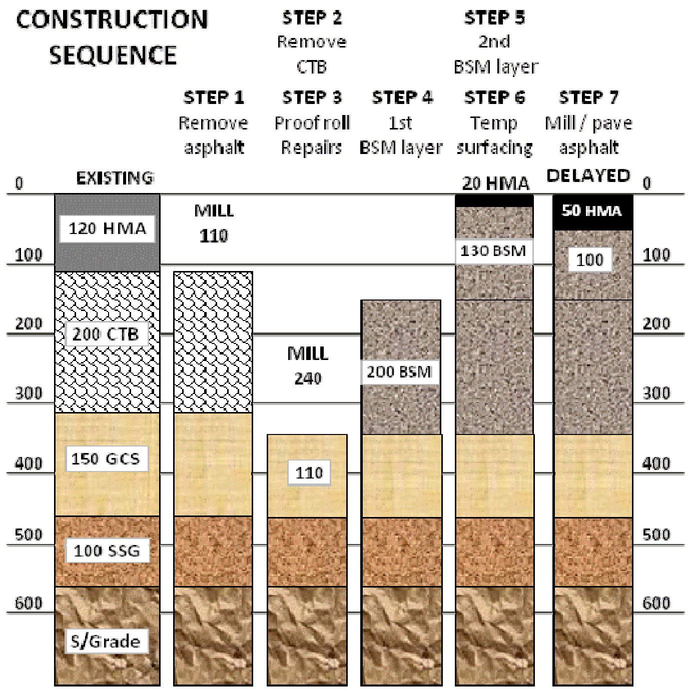


Fig 8. The 7 step sequence of construction processes

Two milling machines were used to remove the asphalt and the underlying CTB (2m machine working in a westerly direction and smaller 1m machine working in the opposite direction). As soon as all material had been removed, a 27 ton pneumatic-tyred roller was brought onto the exposed surface of crushed stone and proof rolling commenced. No heaving was observed. Material for the lower BSM layer was then imported and paving commenced. As soon as 50m had been paved, a 20 ton vibrating padfoot roller started the compaction process.

The second BSM layer with the RA / crusher dust blend was then imported, paved and compacted, followed immediately by the application of the thin asphalt surfacing layer. The finished work was then opened to traffic.

Further 150m sections were completed the following two nights, leaving the last 150m to be tackled on Saturday when daylight work was permitted. Accordingly, work commenced at 06:00 on Saturday and, soon after proof rolling had commenced, heaving was detected over a 40m length across the full lane width.

An excavator was immediately brought in and saturated material removed to a depth of approximately 500mm. The trencher followed the excavator and the outlet drain installed within an hour. The geofabric liner was then placed and the hole backfilled with coarse crushed stone (see Figure 9). The two BSM layers were then constructed followed by the thin asphalt surfacing and the section was opened to traffic before nightfall when the full width of carriageway had to be opened to traffic.

Fig 9. Undercut backfilled with coarse crushed stone

A. *Quality Controls*

The following controls were established and closely monitored:

1) *Mixing Plant*

a) Checklist. Prior to mixing, the comprehensive checklist in Appendix D of TG2 (2009) [1] was followed. Since the Wirtgen KMA 220 plant was relatively new (< 1000 hours) and well maintained, no problems were encountered.

b) Temperatures. The temperature of all input materials (bitumen, water, cement, processed RA and crusher dust) was measured on a daily basis before starting to mix.

c) Processed RA and crusher dust. Samples were taken from the input stockpiles on a daily basis and transported to JBA's Central laboratory for testing (sieve analysis). Whilst collecting samples, the gap setting on the Kleemann impact crusher was checked (and adjusted when necessary)

d) Mixing process. Whilst mixing the following was continuously monitored:
   - flow of all input materials (especially replenishment of feed bins);
   - moisture content of the material on the delivery belt;
   - material throughput rate
e) Stockpiles of mixed material. On days when material was mixed, three samples were taken from each stockpile (100% RA and 15% crusher dust blend), taken to the central laboratory and tested for:
   - moisture content
   - moisture / density relationship
   - ITS_{DRY} and ITS_{WET}

2) BSM Layer Construction

a) BSM quality. Three samples of material were taken from the paver hopper for both the lower and upper BSM layers. As with the samples from stockpile, these were taken to the central laboratory and tested for:
   - moisture content
   - moisture / density relationship
   - ITS_{DRY} and ITS_{WET}

   ITS test results for all samples taken from stockpile and from the paver showed that BSM-1 classification was comfortably achieved throughout.

b) Density of the completed layer. Density was measured using the sand replacement method (nuclear gauges and not permitted in Brazil). Since this is a time consuming procedure, only 3 locations were tested on each layer for every 150m section.

   The mean of the densities measured over the full 600m length of the Trial Section test results were:
   - lower 200mm thick BSM layer: 100.8% of mod AASHTO density
   - upper 130mm thick BSM layer:102.2% of mod AASHTO density.

B. Evaluation Period

Following completion of the 600m long Trial Section on 19th November 2011, the pavement was closely monitored for any signs of deterioration. In May 2012, six months after completing the Trial Section, there was no sign of any problem. As a result, Fremix received the order to continue using this process to rehabilitate the slow lane westwards from where the Trial Section ended to the São Paulo city limit (±4km).

The rehabilitation design called for the upper 50mm of asphalt / BSM material to milled off and replaced with a competent asphalt wearing course mix some two to three months after construction, by which time the moisture content of the BSM material would have reduced to equilibrium levels. However, since the 20mm thick temporary asphalt surfacing was behaving similar to one twice as thick, the decision was taken to leave it in place and to monitor its performance. Such monitoring continued until August 2013 when the decision was taken to adopt the design requirement and install the proper wearing course.

Accordingly, the upper 50mm of pavement was milled off and replaced. It was noted that this decision was prompted by the need to established the “as designed” pavement structure for on-going assessment, not as a consequence of any failure. The additional work being carried out by Fremix continued to receive only the 20mm thick temporary asphalt surfacing and monitoring of these sections is ongoing.

Benkelman Beam deflections were measured in August 2013 immediately before milling and again after the upper 50mm had been removed (before the final asphalt wearing course was paved). Minor differences between the two measurements were recorded. However, of significance was the overall reduction in the level of deflection that had occurred with time, with the majority of measurements falling between 0.2mm and 0.3mm. These measurements together with those taken before and after the trial section was constructed are shown in Figure 10.

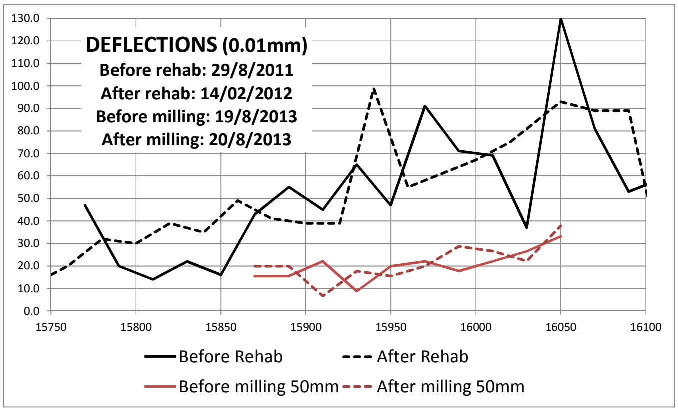
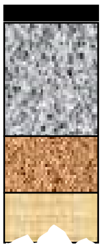


Fig 10. Benkelman Beam deflections

In addition, the permanent deformation (rut depth) affecting the wheel paths was measured prior to milling off the upper 50mm. The average recorded was less than 5mm over the full 600m length of the Trial Section.

Table 8. FWD Analysis: From km 15+650 to km 16+250						
SLOW LANE Applied Load:		8 ton axle load, 541 to 607 kPa applied pressure				
Pavement Layers		Thickness (mm)	Poisson's ratio	**Derived resilient modulus (M_R) (MPa)**		
				Average	80th %ile (high)	80th %ile (low)
	Asphalt surfacing	50	0.4	3000	3000	3000
	BSM base	100	0.35	**1633**	**1954**	**1312**
	BSM upper subbase	200	0.35	**1192**	**1527**	**857**
	Natural lower subbase	250	0.35	346	459	233
	Subgrade support	Inf	0.35	275	309	240

VI. PERFORMANCE EVALUATION

During October 2014, Dynatest was commissioned to carry out a comprehensive FWD survey of the slow lane with measurements recorded at 20m intervals in the outer wheel path. Table 8 summarises the results of back analyses of all data showing the relevant resilient modulus (M_R) values for each layer in the pavement.

To gain further insight into the behaviour of the BSM layers, additional analyses were carried out on the average deflection bowl. Since such analyses limit the number of layers in a pavement structure to 5, the asphalt surfacing was ignored and the lower 200mm thick BSM layer split into 2 sub-layers, each 100mm thick. (BSMs are not continuously bound and, similar to granular materials, can be sub-layered.) The printout of this analysis is shown in Figure 11 and a graph of the derived stiffness (M_R) values in Figure 12.

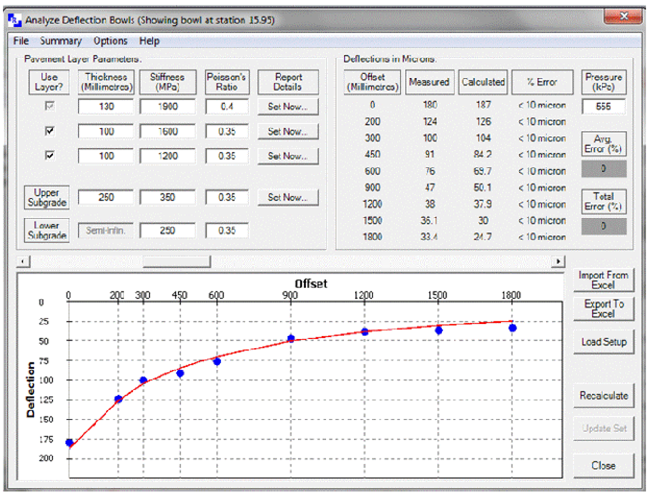


Fig 11. Back calculation for the average deflection bowl using Rubicon Toolbox

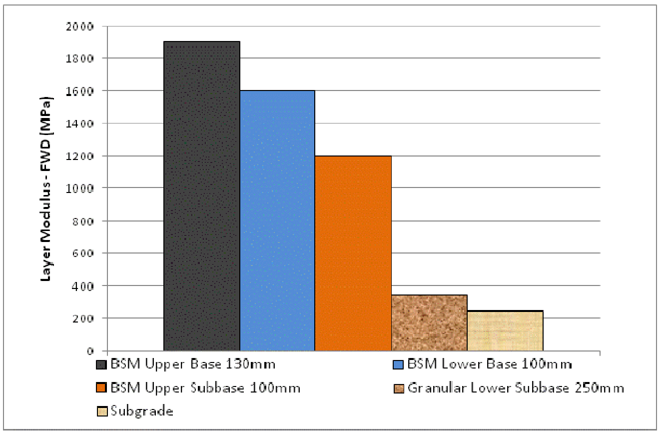


Fig 12. Stiffness (M_R) values derived from sub-layering

This analysis highlights the ability of granular and non-continuously bound materials to find balance within a pavement structure. A modular ratio between the lower (unbound) granular subbase layer and the overlying non-continuously bound) BSM upper subbase sub-layer is in the order of 3.4, which is relatively high.

However, the ratio between successive BSM layers in the upper pavement structure is shown to be below 2. Further sub-layering will see these ratios self-adjusting to a more uniform balance and is considered to be the primary reason for the exceptional performance of this pavement.

VII. CONCLUSIONS

Estimates indicate that over 30 million EASLs have been carried to date on the 600m long Trial Section constructed in November 2011. The 20mm thick temporary asphalt surfacing was replaced in August 2013, thereby eliminating the small amount of permanent deformation that had developed in the wheel paths due to consolidation. This section will be closely monitored in the future as part of a long-term pavement performance (LTPP) programme. In the meantime, work on rehabilitating the truck lanes of this highway is ongoing with more than 100 lane-kilometres completed by the end of 2014. The 20mm thick temporary asphalt surfacing has been retained on these sections and will only be replaced with a permanent 50mm thick asphalt surfacing layer when the need arises.

The success of this project and the excellent result achieved was a product of attention being paid to detail during every step of the process, including pavement investigations, design procedures and, especially, the construction work. The short 6-hour working window was the biggest challenge faced, necessitating that the contractor plan and execute the various operations with military precision. Performance to date shows that the end-product envisaged at the design stage was achieved in the field.

The results of analyses carried out on a comprehensive FWD survey carried out in October 2014 (33 months after construction) show that the rehabilitated pavement has achieved balance between the various layers that constitute the pavement structure. This augurs well for the future. Provided the magnitude of applied loads is controlled, such a balanced pavement will provide a long service life. In addition, the 300mm thick BSM base layer incorporates predominantly RA material that, after treating with foamed bitumen and active filler, is largely insensitive to moisture ingress (as already shown by the lack of moisture-related damage to those sections that have endured more than 3 years with only a 20mm thick gap-graded asphalt surfacing).

The decision taken by EcoPistas to continue using this process reflects the economic benefits that accrue from addressing the cause of the problem rather than continuing to treat the symptoms by milling and replacing the asphalt layers every three-to-six months.

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