Insitu Foamed Bitumen Stabilisation - The City of Canning Experience

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Abstract

This paper reviews the rehabilitation of three roads in the City of Canning and discusses the aspects of the selection of the insitu foamed bitumen stabilisation as the rehabilitation method and testing performed on the pavement sections. The design parameters and fatigue deterioration model used to predict the performance of the pavement are reviewed. The cost savings and reduction in construction time are highlighted with the other alternatives considered.

Introduction

The City of Canning has over a number of years been keen to foster the development of innovative solutions for the rehabilitation of failed pavement sections, and the maintenance of the remainder of its road network. One such method is insitu foamed bitumen stabilisation, which was introduced to the City as an alternative rehabilitation method by Pavement Technology Ltd. In the 1998/99 financial year, seven road sections were scheduled for rehabilitation, (not including roads scheduled for resurfacing). Of these, three were rehabilitated by the conventional granular overlay method, one by modified full depth asphalt construction and three by insitu foamed bitumen stabilisation.

Previously, the City has opted for either:

- Structural asphalt overlays
- Granular overlays
- Full granular reconstruction
- Insitu stabilisation using cement or slag/lime binder
- Modified full depth asphalt construction
- Plant-mix emulsion stabilisation

Over the past five years, the insitu foamed bitumen stabilisation method has begun a renaissance after falling from favour twenty years ago, following large price rises in the cost of bitumen. Recent work on the east coast of Australia, Europe and South Africa has seen a revival of the technique due to constraints on various rehabilitation methods and the development of more sophisticated equipment for insitu recycling applications.

It is as important road to rehabilitation as in any other area of engineering activity to review the available options, assess the implications of each option, and to progress in the full knowledge of the assessment.

However in the adoption of any new pavement method, this should not be considered a linear progression. The method should be reviewed in the light of available knowledge, assessed for suitability and performance, and the method progressed to a more reliable design process, which in itself is reviewed for soundness, assessed for suitability and progressed to greater refinement.
This paper reports on the three insitu foamed bitumen stabilisation projects undertaken in the City in February 1999. The pavements were designed by the City of Canning, and constructed by Pavement Technology Pty Ltd.

Selection of Treatment Option

In selecting an appropriate rehabilitation treatment, the City has traditionally used criteria of:

- Magnitude of pavement distress
- Boundary height restrictions imposed by existing developments
- Subgrade strength and existing pavement thickness
- Traffic volumes and type
- Cost of treatment
- Deficiency in existing pavement materials

However the growing public and commercial pressure have resulted in the need for additional criteria to be considered such as:

- Cost and effect on business trade loss due to traffic disruption and detours
- Cost to public and commercial traffic due to delays caused by works
- Environmental considerations
- Safety of construction workers and public during works

The City now considers all of these criteria when assessing various treatment options.

The adoption of insitu foamed bitumen stabilisation was based on the following factors in High Road, Nicholson Road and John Street:

- The method allowed for the retention of existing kerbs and minimised crossover alterations
- The existing pavements were suitable for the process
- Disruption to business operations were minimised in High Rd and John Street
- Construction time was significantly reduced, limiting long traffic delays and the risks of injury to pedestrians confused by contraflow conditions.
- The pavement would be sound when open to traffic at night with no loose stones and minimal dust.
- The process was the least cost option, both initially and whole of life.

The selection of a rehabilitation method should proceed in a systematic manner. The most important consideration is to gain an understanding of the performance of the current pavement, the properties of the existing pavement and the traffic, both current and predicted, that the pavement will be subjected to.

As a minimum level of investigation therefore, it would be recommended that pavement dippings be undertaken at a regular spacing, with penetrometer surveys undertaken on the subgrade and a level survey on the channel on kerbed roads, particularly in flat areas where ponding may be a problem.

A more detailed investigation would identify the various pavement materials used in the original construction, and undertake testing to ascertain deficiencies in the materials.

A final level of testing may be a deflection survey, to determine the performance of the existing pavement. One of the significant benefits of this method is that it is non destructive, causes minimal disruption and provides the engineer with a good indication of subgrade strength and variability, and can indicate defective pavement materials.
Once the subgrade modulus and design traffic loading have been determined, various pavement options can be determined to provide the required protection to the subgrade for the life of the pavement.

Case Studies

General

The City of Canning had several important road rehabilitation projects listed on the 1998/9 budget. These included High Road between Willeri Drive and Meadowbrook Drive and Nicholson Road between Metcalf Road and the Canning River. During the investigations for the proposed resurfacing of John Street, it became apparent that the section between Railway Parade and Charles Street required rehabilitation rather than resurfacing.

After an evaluation of each of these projects, the insitu foamed bitumen stabilisation option was selected as the most appropriate rehabilitation method. The reasons for the selection of the treatment options are outlined in the following sections.

High Road

High Road is a dual carriageway, and both carriageways on the section from Willeri Drive to Meadowbrook Drive required rehabilitation. The pavement was significantly deformed, with ruts up to 60mm deep, and the longitudinal deformation providing an uncomfortable ride, especially for trucks. In addition the pavement was significantly cracked, with both fatigue and block cracking evident.

The road passes the Riverton Forum Shopping Centre, with a service station, several fast food outlets, a hotel and a TAB outlet having the major or sole access from High Road (see Figure 1).

Figure 1 ***View of High Road from Cerberus Ave. (Estimated 30yr traffic of 2.6 x 10^6 ESAs)***

Pavement cores showed that the pavement was constructed of a limestone subbase, crushed granite roadbase base course and asphalt wearing course. The thickness of each layer was variable as shown in Table 1. A deflection survey was undertaken using the Falling Weight Deflectometer, and the design subgrade CBR was determined by back-modulus calculation at 10%. Due to soil conditions in Perth being a well drained sand, this CBR value is considered more realistic than a soaked CBR value.

Using the Austroads Pavement Design Guide with a subgrade CBR of 10%, rehabilitation using a granular overlay would require a minimum layer of 120 mm of high quality base course was required to provide for a 30 year design traffic of 2.6×10^6 ESAs. This pavement rehabilitation approach would require the removal of existing kerbing and asphalt, placing the new base course, a primer seal and new asphalt wearing course.

Due to the severe disruption to the business operations adjacent to the road, and the risks to pedestrians crossing from the adjacent residential areas, the alternatives considered to minimise construction time were full depth asphalt and insitu foamed bitumen stabilisation. Both of these options would allow the existing kerbs to be maintained.

The estimated cost and construction times for the three options considered were as follows:

Option	Construction Time	Cost
Granular reconstruction of base course	62 days	$618,000
Insitu foamed bitumen stabilisation	6 days	$508,600
Full Depth Asphalt	8 days	$589,800

The insitu foamed bitumen stabilisation option, which was then adopted as the most beneficial method, due to its lower capital cost and short construction time

Nicholson Road

Nicholson Road is a heavily trafficked road performing the tasks of the proposed Roe Hwy at present (see Figure 2). The rehabilitated carriageway carries 23,500 vehicles/day, with approximately 11% commercial vehicles. Approximately mid way in the section to be treated, Spencer Road intersects Nicholson road at a signalised intersection.

Figure 2 ***View of Nicholson Street from Canning River Bridge (Estimated 30yr design traffic 1.9 x 10^7 ESAs).***

No business premises were effected by the works, but due to high traffic volumes, very long delays occur when lanes are closed at this location.

A section of this pavement had been reconstructed approximately three years ago using a Ferricrete granular overlay. This section had already failed, with significant areas of fatigue cracking. The remaining sections of the pavement were severely rutted, and shoving of thick asphalt repairs was widespread.

Table 1 Existing pavement details for High Road.

Chainage (m)	Surfacing (mm)	Base Layer (mm)	Subbase Layer (mm)	Subbase Layer (mm)	TOTAL (mm)
East Bound Carriageway					
20	30 AC	30 Roadbase	280 Gravel		340
120	25 AC	65 Roadbase	195 Limestone		285
220	15 AC	65 Roadbase	230 Limestone		310
320	25 AC	75 Roadbase	220 Limestone	160 Gravel	480
420	30 AC	65 Roadbase	215 Limestone		310
520	25 AC	70 Roadbase	105 Limestone		200
620	25 AC	85 Roadbase	155 Limestone	225 Gravel	490
720	15 AC	25 Roadbase	270 Limestone		450
820	30 AC	60 Roadbase	180 Limestone	140 Roadbase/Limestone	410
920	15 AC	85 Roadbase	230 Limestone		330
1020	35 AC	85 Roadbase	370 Limestone		490
West Bound Carriageway					
530	30 AC	40 Roadbase	180 Limestone		250
630	30 AC	50 Roadbase	200 Limestone		280
730	25 AC	55 Roadbase	270 Limestone		350
830	25 AC	65 Roadbase	200 Limestone		290
930	25 AC	95 Roadbase	280 Limestone		400
1030	35 AC	55 Roadbase	240 Limestone		330
1130	20 AC	70 Roadbase	235 Limestone		325
1230	25 AC	70 Roadbase	175 Limestone		270
1330	25 AC	75 Roadbase	200 Limestone	150 Gravel	450
1430	25 AC	55 Roadbase	220 Limestone		300

Falling Weight Deflectometer testing, backed by laboratory testing, dynamic cone penetrometer and friction cone penetrometer testing provided a design subgrade CBR of 8%. The 30 year design traffic was determined at 1.9×10^7 ESAs.

The pavement cores showed that with the exception of the previously reconstructed section, the pavement was extremely variable, as shown in the records of pavement cores in Table 2. Due to the variability of the pavement cores, three options were examined, a granular reconstruction of the base course, full depth asphalt and insitu foamed bitumen stabilisation.
The estimated cost and construction times for the options were as follows:

Option	Construction Time	Cost
Granular reconstruction of base course	10 days	$134,300
Full depth asphalt	4 days	$131,000
Insitu foamed bitumen stabilisation	3 days	$116,500

The advantages of the full depth asphalt and insitu foamed bitumen stabilisation are that these options are not subject to the problems of damage to the base whilst waiting for dry back.

Table 2 Existing pavement details for Nicholson Road.

Chainage (m)	Surfacing (mm)	Base Layer (mm)	Subbase Layer (mm)	Subbase Layer (mm)	TOTAL (mm)
82	70 AC	120 Limestone	100 Stabilised	210+ Limestone	500+
108	75 AC	25 Roadbase	230 Limestone		330
128	25 AC	80 Roadbase	70 Stabilised	280+ Limestone	455+
152	110 AC	210 Limestone			320
178	20 AC	100 Roadbase	240 Limestone		360
198	80 AC	300 Limestone			380
218	50 AC	35 Roadbase	30 Old Mix	335 Limestone	450
267	40 AC	150 Ferricrete	50 Roadbase	165 Limestone	405
270	35 AC	150 Ferricrete	310 Limestone		495
310	40 AC	150 Ferricrete	145 Roadbase	320+ Limestone	655+

John Street

John Street is an important industrial road, and the section between Sevenoaks and Charles Streets is subjected to heavy truck traffic, including road trains serving a major stock feed manufacturer (see Figure 3). It carries 3,330 vehicles/day with 10% commercial vehicles, and has several business premises whose only access is from John Street.

Pavement cores indicated a minimum existing pavement thickness of 185 mm (see Table 3), and an estimate of subgrade CBR was 6% from back-modulus calculations from FWD testing. For a design traffic of 5.1×10^6 ESAs, a granular pavement of 400mm was required. This could only be achieved with a complete reconstruction of the road, as a granular overlay with a raising of levels would not allow drainage from adjoining properties to be maintained.

The alternative was to use the insitu foamed bitumen stabilisation technique, and incorporate some of the coarse single sized sand subgrade into the pavement.

Figure 3 *View of John Street from Charles Street (Estimated 30yr design traffic 5.1 x 10^6 ESAs).*

The estimated cost and construction times for the options were as follows:

Option	Construction Time	Cost
Granular reconstruction of base course	19 days	\$117,100
Insitu foamed bitumen stabilisation	2 days	\$103,700

Table 3 Existing pavement details for John Street.

Chainage (m)	Surfacing (mm)	Base Layer (mm)
30	15 AC	255 Gravel
40	35 AC	250 Gravel
40	20 AC	165 Gravel
75	25 AC	230 Gravel
75	20 AC	195 Gravel
80	30 AC	220 Gravel
110	20 AC	170 Gravel
110	30 AC	220 Gravel
130	10 AC	210 Gravel

Design Method

The design of an Insitu Foamed Bitumen Stabilised pavement requires the following inputs:

- Subgrade modulus (or CBR)
- Flexural modulus for stabilised layer
- Flexural modulus of asphalt surface
- Fatigue performance criteria for stabilised layer
- Fatigue performance criteria for asphalt surface
- Subgrade performance criteria
- Design traffic loading and heavy vehicle annual growth
- Traffic constant
- Design Reliability Factor

These parameters are input into the CIRCLY program, which calculates the stresses and strains in each pavement layer and the compressive strain on the subgrade when the pavement is subjected to the design standard axle for an assumed pavement thickness. From the computer results the level of strain and the performance fatigue criteria one can estimate the allowable number of load repetitions, which should be greater than the actual traffic loading factored by the selected design reliability factor.

There are some important points to note in the selection of the appropriate inputs:

The modulus required for the CIRCLY program is the flexural modulus, not the commonly reported resilient or indirect tensile modulus.

For bituminous materials, the modulus is not a single value, but varies with traffic speed and pavement temperature. The modulus values used therefore need to be specified at a specific speed (rise time) and temperature. The Weighted Mean Annual Pavement Temperature (WMAPT) for Perth (30^0C) was used for these designs.

The fatigue performance criteria used for the insitu foamed bitumen stabilised layer is assumed to be that for asphalt. (Maccarone, Holleran and Leonard, 1993). This was considered doubtful as the material has not the same degree of control as asphalt, and as such some testing was considered necessary to confirm the validity of this assumption in future works.

The subgrade performance criteria used is the Austroads equation, which has again been developed over considerable time and much research.

The traffic constant is the same as that used for asphalt, that is 1.1.

The design reliability factor is a statistically based factor that improves the reliability of the final pavement. A factor of 4.5 was adopted in line with Vic Roads Technical Bulletin No 37.

In all cases, as the asphalt wearing course is very thin in relation to the pavement thickness, and as the tensile strains in the bound layers are at a maximum at the bottom of the layer, the design is invariably governed by the fatigue life of the stabilised layer.

A conservative presumptive flexural modulus of 2,400 MPa for the stabilised layer was adopted after testing of an actual pavement sample by the Mobil laboratory in Melbourne. The testing indicated an indirect tensile modulus of 7,730 MPa dry and 3,080 MPa wet at 4% bitumen content and 2% hydrated lime content.

Experience in previous test results obtained by Pavement Technology Ltd. in other locations indicated that the 4% bitumen, 2% hydrated lime combination was almost a standard result. Due to the high cost of testing and in view of the extreme variability of the existing pavement materials it was decided to adopt this design across all pavements, and assess the design by testing after constructing the pavement.

Using CIRCLY to calculate the strain levels in each layer, and hence the allowable number of repetitions at that strain level, the following pavement configurations were adopted:

High Road: 30 mm Asphalt : 225 mm Insitu foamed bitumen stabilised
Nicholson Road: 40 mm Asphalt : 320 mm Insitu foamed bitumen stabilised
John Street: 30 mm Asphalt: 275 mm Insitu foamed bitumen stabilised

There was considerable variation in the materials making up the stabilised mix for all three sites, as shown in Tables 4, 5 and 6 for High Road, Nicholson Road and John Street respectively.

Table 4 High Road - Percentage by mass of existing pavement materials to be recycled.

Chainage	Asphalt	Roadbase	Limestone	Gravel	Sand
East Bound Carriageway					
20	13.6%	13.6%		72.8%	
120	11.1%	28.9%	60.0%		
220	6.7%	28.9%	64.4%		
320	11.1%	33.3%	55.6%		
420	13.3%	28.9%	57.8%		
520	11.1%	31.1%	46.7%		11.1%
620	11.1%	37.8%	51.1%		
720	6.7%	11.1%	82.2%		
820	13.3%	26.7%	60.0%		
920	6.7%	37.8%	55.6%		
1020	15.6%	37.8%	46.7%		
West Bound Carriageway					
530	13.3%	17.8%	68.9%		
630	13.3%	22.2%	64.4%		
730	11.1%	24.4%	64.4%		
830	11.1%	28.9%	60.0%		
930	11.1%	42.2%	46.7%		
1030	15.6%	24.4%	60.0%		
1130	8.9%	31.1%	60.0%		
1230	11.1%	31.1%	57.8%		
1330	11.1%	33.3%	55.6%		
1430	11.1%	24.4%	64.4%		

Table 5 Nicholson Road - Percentage by mass of existing pavement materials to be recycled.

Chainage	Asphalt	Roadbase	Ferricrete	Limestone	Gravel
82	21.9%			46.9%	31.2%
108	23.4%	7.8%		68.8%	
128	7.8%	25.0%		45.4%	21.8%
152	34.4%			65.6%	
178	6.2%	31.2%		62.6%	
198	25.0%			75.0%	
218	15.6%	10.9%		64.2%	9.3%
267	12.5%	15.6%	46.9%	25.0%	
270	10.9%		46.9%	42.2%	
310	12.5%	40.6%	46.9%		

Table 6 John Street - Percentage by mass of existing pavement materials to be recycled.

Chainage	Asphalt	Gravel	Sand
30	5.5%	92.7%	1.8%
40	12.7%	87.3%	
40	7.3%	60.0%	32.7%
75	9.1%	83.6%	7.3%
75	7.3%	70.9%	21.8%
80	10.9%	80.0%	9.1%
110	7.3%	61.8%	30.9%
110	10.9%	80.0%	9.1%
130	3.6%	76.4%	20.0%

Construction Process

The construction process involved two stages,

Stage 1: Pulverisation of the existing pavement, to approximately 90% of total design depth with the incorporation of the quicklime and water to slake the lime and for compaction (see Figure 4). After the mixing of lime and water, the pavement is trimmed to shape using a grader and lightly compacted. At this stage, some shape correction can be achieved.

Figure 4 ***The Wirtgen 2500 reclaimer/stabiliser was used for stages 1 and 2.***

Stage 2: The next stage involves the bitumen stabilisation process where the hot bitumen with additives to produce a stable foam is pumped from the tanker to the mixing chamber of the stabiliser. Water is injected into the bitumen at the spray nozzles which causes the bitumen to foam to a minimum of 10 times its original volume. This foamed bitumen is blended with the pavement material and binds the fine particles.

Following the stabiliser, the material was compacted using a vibrating 11t padfoot roller, trimmed to shape with a grader, and further compacted using a combination of a vibrating 10t smooth drum roller and 30t rubber tyre roller.

Using this technique, 2,500 m² of pavement material was insitu stabilised in an 8-hour shift.

At the end of each day, the road was opened to traffic, and there was no evidence of any initial deformation or ravelling of the surface. The stability of the surface was graphically illustrated by the Nicholson Road and Spencer Road intersection, where after four days of traffic, the stabilised pavement was in excellent condition, with minimal loose stones, ravelling or deformation (see Figure 5). At one location, skid marks were clearly evident where a truck had made an emergency stop. (see Figure 6)

Figure 5 ***A view of the surface before sealing at the intersection of Nicholson and Spencer Roads***.

Figure 6 – ***Skid marks on stabilised surface of Nicholson Rd***

Insitu and Laboratory Testing

Testing during construction

The testing during construction consisted of collecting samples for Modified Maximum Dry Density (MDD) testing and Particle Size Distribution. As it was not clear how the samples would be effected by the bitumen, samples for grading were collected after addition of lime and immediately prior to the addition of the bitumen. MDD determinations were carried out immediately after the addition of bitumen, and prior to compaction of the stabilised layer.

A nuclear density meter was used to determine density, and the results of testing showed that the compaction requirements of a characteristic value of 95% (modified) were easily achieved. The mean relative densities achieved were 101.7%, averaged over the 200mm test depth, over all projects, with a standard deviation of 2.7%. Testing was undertaken to AS1289.5.8.1 (1995) Methods of Testing Soils for Engineering Purposes – Soil & Density Tests – Determination of Field Density and Field Moisture Content Using a Nuclear Surface Moisture Density Gauge – Direct Transmission Method.

The particle size distribution was compared to recommended envelopes. In all cases the grading was outside of the recommended envelopes (see Figure 7). The John Street samples were deficient in fines, but in High Road and Nicholson Road, the samples were deficient in both fines and sand to small aggregate sizes. Testing was undertaken to AS1289.3.6.1 (1995) Dry test method.

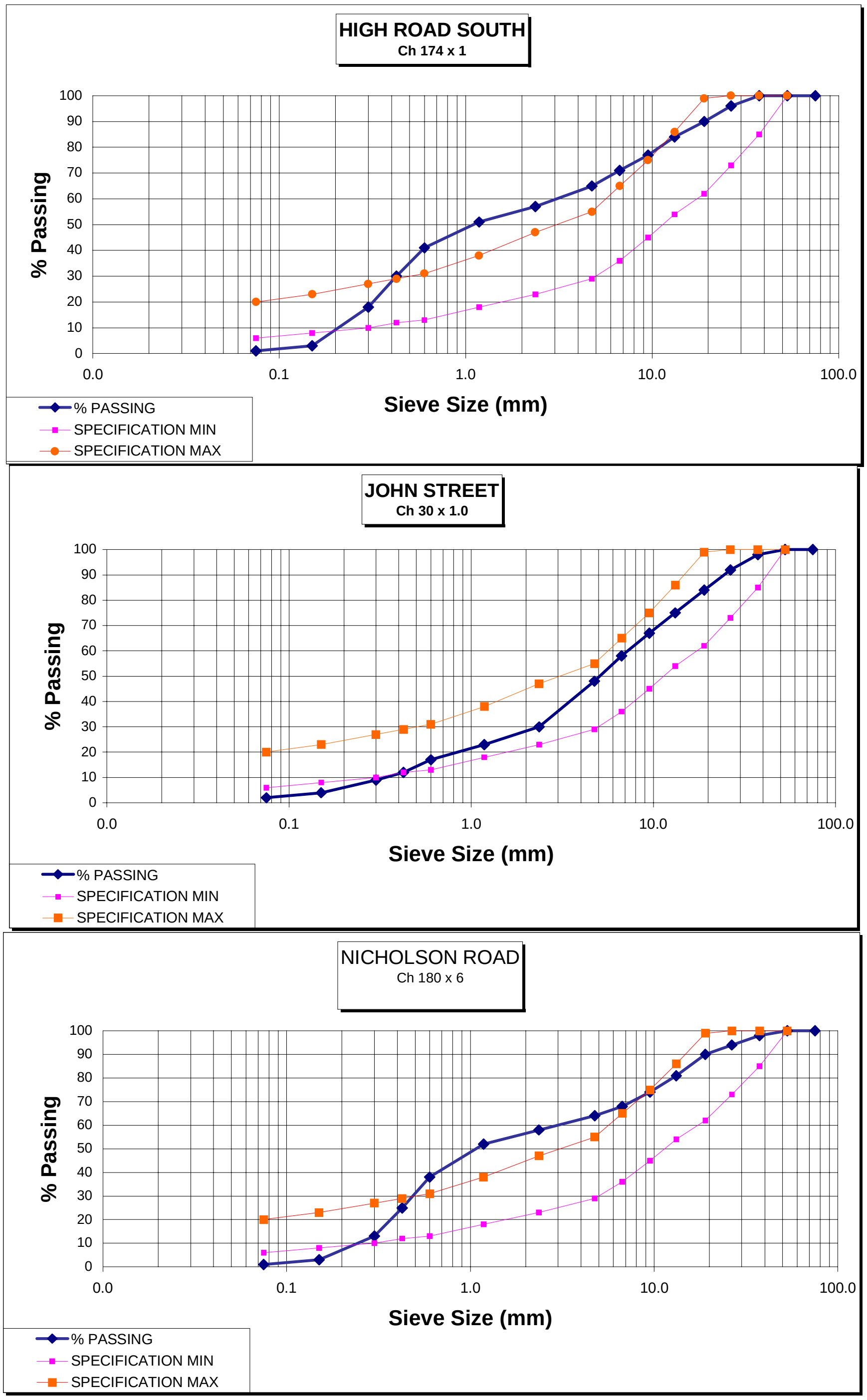

Figure 7 ***Typical grading curves for the pulverised pavement materials after lime stabilisation***

Testing on Completed Pavement

Testing on the completed pavement comprised of :

- Falling Weight Deflectometer testing,
- indirect tensile modulus testing on pavement cores,
- density testing on pavement cores,
- bitumen extraction,

In addition, slabs were cut for fatigue, modulus (both flexural and indirect tensile) and unconfined compressive strength testing at the ARRB Transport Research laboratory in Melbourne.

Deflection Testing

The eastbound lanes of High Rd were tested with the Falling Weight Deflectometer at various time intervals after construction. The results indicated that initially, the pavement performed as a conventional granular pavement, but by two months had stiffened considerably, such that vertical deflection and curvature values were similar to those obtained on full depth asphalt pavements constructed elsewhere in the City of Canning.

The rehabilitated sections of High Rd, Nicholson Rd and John Street were tested after 5 months, and all showed low deflection and curvature values as shown in Table 7.

Table 7 Falling Weight Deflectometer test results

Pavement Section	Date Constructed	Date Tested	Mean Pavement Temp (^{0}C)	Maximum Deflection (mm)		Curvature (mm)	
				Mean	Std Dev	Mean	Std Dev
High Rd East Bound	18/01/99	22/01/99	24.2	0.38	0.12	0.30	0.05
High Rd East Bound	18-21/01/99	25/03/99	21.9	0.02	0.01	0.19	0.05
High Rd East Bound	18-21/01/99	09/06/99	20.5	0.02	0.01	0.01	0.01
High Rd West Bound	22-23/01/99	09/06/99	13.0	0.06	0.04	0.04	0.02
Nicholson Rd	24/01/99	09/06/99	20.3	0.03	0.04	0.01	0.02
John St	25/01/99	09/06/99	23.0	0.12	0.04	0.05	0.02

Modulus, density and strength testing

Testing was undertaken on cores retrieved from the pavement after five months of service. Testing was undertaken to determine the:

- Insitu density to MRWA 733.1 Bulk density and Voids Content of Asphalt test method,
- Unconfined compressive strength to ASLABS Method on 100mm diameter x 65mm sample,
- Voids to MRWA 733.1 Bulk density and Voids Content of Asphalt test method,
- Bitumen content to MRWA 730.1 Bitumen Content and Particle Size Distribution of Asphalt and stabilised Materials – Centrifuge Method and
- Indirect tensile modulus to ASLABS Method on 100mm diameter x 65mm sample.

The testing undertaken on the cores for indirect tensile modulus was undertaken using the MATTA machine by ASLAB Pty Ltd and ARRB Transport Research. Testing by ASLAB was undertaken at 25^0C and 30^0C, and at a range of rise times, to simulate various traffic speeds. Testing by ARRB Transport Research was undertaken at 25^0C only.

In several locations it was possible to test cores at different levels in the pavement in order to determine if there was a compaction profile, and the effects that this profile has on the performance of the pavement. The testing showed that there is a density profile and that density reduces with depth. However, it was not always possible to recover the full depth of the stabilised material by coring. Table 8 shows the recorded densities of the pavement levels tested, and the relative density of the lower layers compared to the top layer.

Table 8 Density measurement results from testing cores at various pavement depths.

Pavement Section	Chainage	Core Depth	Density (t/m^3)				
			0-100mm	100-200mm		>200mm	
			Value	Value	% Top	Value	% Top
High Road West Bound	174	250	2.10	2.03	96.6		
	473	170	2.10				
High Road East Bound	200	180	2.10				
	587	100	2.07				
	787	180	2.07	1.97	95.2		
	816	220	2.02	1.92	95.1		
	1083	270	2.12	2.08	98.1	1.98	93.4
Nicholson Road	180	330	2.03	1.94	95.6	1.92	94.6
	277	330	2.13	2.05	96.2	2.01	94.4
	310	220	2.09	1.95	93.3		
	340	320	2.08	2.05	98.6	2.04	98.1
John Street	40	150	2.18				
	113	120	2.24				

The mean compaction of the 100 to 200 mm layer is 96.1% (modified) of the top 0 to 100 mm layer. The mean compaction of the >200 mm layer is 95.1% (modified) of the top 0 to 100 mm layer. Whilst the reduced compaction values of the lower layers does not appear significant, as will be shown later, the effect on modulus is considerable.

Due to the operation of the MATTA machine, variations in the rise times occur, and in order to compare results, a polynomial regression was calculated for each data set. The results are reported in Tables 9 and 10 are adjusted to specific rise times for both 25^0C and 30^0C for comparison purposes.

Where cores were able to be recovered at 100 to 200 mm and 200 to 300 mm depths, the indirect tensile modulus value at these levels in addition to the 0 to 100 mm modulus value was recorded and the percentage value of the lower layers compared to the upper layers has been calculated.

It is apparent from tables 9 and 10, that the small reduction in compaction with depth has a large effect on the recorded indirect tensile modulus of the layer.

Some of the cores were tested for bitumen content and Unconfined Compressive Strength (UCS) after 5 months in operation. The results of these tests are shown in Table 11. It can be seen there is a considerable variation in the UCS and bitumen contents of the samples. The variation in bitumen content may be largely explained by the effect of large aggregate pieces in the small sample volume obtained using a coring machine, as well as the effect of incorporating differing proportions of existing asphalt into the mix.

As expected, the UCS does reduce with the level of compaction, as indicated by the reduced values obtained when cores at different levels in the same location were tested.

Table 9 Indirect tensile modulus values at 25^{0}C at 5 months age.

Pavement Section	Chainage	Rise Time	Indirect Tensile Modulus (MPa)				
			0-100mm	100-200mm		200-300mm	
			Value	Value	% Top	Value	% Top
High Road West Bound	174	75	10291	6871	66.8		
		37	11692	8539	73.0		
		25	12236	9310	76.1		
		19	12523	9736	77.7		
	473	75	8434				
		37	10634				
		25	11645				
		19	12203				
High Road East Bound	200	75	13309				
		37	14865				
		25	15477				
		19	15801				
	587	75	6818				
		37	8107				
		25	8724				
		19	9067				
	787	75	8414	4270	50.8		
		37	9459	4868	51.5		
		25	9800	5165	52.7		
		19	9969	5332	53.5		
	816	75	4442	3195	71.9		
		37	5285	3810	72.1		
		25	5652	4099	72.5		
		19	5851	4259	72.8		
	1083	75	7836	5331	68.0	3628	46.3
		37	9203	6426	69.8	4136	44.9
		25	9820	6947	70.7	4351	44.3
		19	10158	7237	71.2	4468	44.0
Nicholson Road	180	75	8999	5894	65.5	3911	43.5
		37	12560	8043	64.0	5204	41.4
		25	14229	9073	63.8	5864	41.2
		19	15154	9647	63.7	6238	41.2
	277	75	9667	9212	95.3	2797	28.9
		37	10307	10312	100.0	3297	32.0
		25	10536	10749	102.0	3494	33.2
		19	10653	10981	103.1	3598	33.8
	310	75	6510	4842	74.4		
		37	7915	5646	71.3		
		25	8571	6019	70.2		
		19	8934	6226	69.7		
	340	75	6745	6031	89.4	4725	70.1
		37	8144	7150	87.8	5634	69.2
		25	8790	7660	87.1	6031	68.6
		19	9147	7941	86.8	6247	68.3
John Street	40	75	3067				
		37	3820				
		25	4184				
		19	4387				
	113	75	3690				
		37	4678				
		25	5140				
		19	5395				

Table 10 Indirect tensile modulus values at 30^{0}C at 5 months age.

Pavement Section	Chainage	Rise Time (ms)	Indirect Tensile Modulus (MPa)				
			0-100mm	100-200mm		200-300mm	
			Value	Value	% Top	Value	% Top
High Road West Bound	174	75	9769	6179	63.3		
		37	10585	7606	71.9		
		25	10826	8281	76.5		
		19	10941	8657	79.1		
	473	75	8558				
		37	10358				
		25	11164				
		19	11605				
High Road East Bound	200	75	9776				
		37	11573				
		25	12353				
		19	12777				
	587	75	6488				
		37	7675				
		25	8240				
		19	8555				
	787	75	7235	4450	61.5		
		37	8549	5000	58.5		
		25	9154	5276	57.6		
		19	9488	5431	57.2		
	816	75	3676	3104	84.5		
		37	4485	3629	80.9		
		25	4848	3862	79.7		
		19	5047	3989	79.0		
	1083	75	7544	4873	64.6	3578	47.4
		37	9008	5854	65.0	4010	44.5
		25	9658	6331	65.6	4200	43.5
		19	10014	6598	65.9	4304	43.0
Nicholson Road	180	75	10020	5420	54.1	3518	35.1
		37	12090	6545	54.1	4543	37.6
		25	12928	7087	54.8	5068	39.2
		19	13375	7389	55.2	5365	40.1
	277	75	9113	8401	92.2	2095	23.0
		37	10424	9562	91.7	2461	23.6
		25	11021	10069	91.4	2589	23.5
		19	11349	10345	91.2	2654	23.4
	310	75	5355	4182	78.1		
		37	6277	4820	76.8		
		25	6707	5114	76.2		
		19	6945	5275	76.0		
	340	75	5893	5505	93.4	4286	72.7
		37	6823	6622	97.0	5043	73.9
		25	7228	7150	98.9	5383	74.5
		19	7448	7443	99.9	5569	74.8
John Street	40	75	2936				
		37	3261				
		25	3407				
		19	3486				
	113	75	2802				
		37	3179				
		25	3324				
		19	3401				

Table 11 Unconfined Compressive Strength and Bitumen Content at 5 months age.

Road	Chainage	Depth (mm)	Density (t/m^3)	UCS (MPa)	Bitumen Content (% by mass)
Nicholson Road	180	0-100	2.034	7.4	
		100-200	1.939		3.1
		200-300	1.915	6.3	
	277	0-100	2.13		4.4
	310	0-100	2.09		6.2
		100-200	1.951	5.5	
	340	0-100	2.082	7.8	5.4
		100-200	2.048	7.1	
		200-300	2.044	5.8	
High Road East Bound	200	0-100	2.095		3.9
	587	0-100	2.056		3.9
	787	0-100	2.067	6.4	4.8
		100-200	1.969	5.7	
	816	0-100	2.015	4.4	6.0
		100-200	1.928	4.1	
	1083	0-100	2.115		5.7
		100-200	2.079		6.2
		200-300	1.975		5.4
High Road West Bound	174	0-100	2.097	8.3	4.6
		100-200	2.028	7.9	
	473	0-100	2.097	8.5	4.5
John Street	40	0-100	2.184		2.4
	113		2.239	3.5	5.8

Fatigue testing

Slabs were cut from a single location in each road treated and sent to the ARRB Transport Research laboratory in Melbourne for fatigue testing. Four slabs of 0.5 m square were cut in each location, and from three of these three beams were cut to 400 mm x 64 mm x 50 mm, the size required for the fatigue tests. The beams are cut from the centre of the stabilised layer depth. The remaining slab was used for indirect tensile modulus testing.

The fatigue testing was undertaken at 20^0C, using draft test currently under development as an Australian Standard. The test value is obtained by determining the initial flexural modulus and applying a cyclic load to achieve given strain level until the flexural modulus reduced to half its initial value. This value is taken as the fatigue life of that sample at the particular strain level. The results of fatigue tests are shown in Table 12.

Unfortunately it was discovered during the preparation of the beams that the sample from Nicholson Road had been cut in the site of a longitudinal joint, and there was a definite weakness at this point. This would indicate that joints could be considerably weaker than the remainder of the pavement.

In order to calculate the predicted life using the asphalt fatigue relationship, it is necessary to determine the actual binder content. The binder content from core extractions is not accurate, as it includes the bitumen contained in the original variable thickness asphalt surface. Thus to estimate the actual binder content, the mean actual spread rate by mass of bitumen was compared to the mean field density values obtained in density testing. The calculated values are as follows:

High Road -	3.2% by mass	7.8% by volume
Nicholson Road -	3.4% by mass	6.9% by volume
John Street -	3.2% by mass	6.9% by volume

The final bitumen contents were less than the specified 4% in all cases.

The fatigue relationship adopted at design was the Austroads asphalt fatigue relationship, and the predicted cycles to failure determined by this model, the actual measured test cycles to failure and the ratio of the predicted number of cycles to the measured number of cycles have been included in Table 12

Table 12 Fatigue testing and comparison of measured and predicted values.
(As reported by ARRB Transport Research)

Road	Sample	Air Voids (%)	Initial Flexural Modulus (MPa)	Actual Mean Strain (με)	Measured Cycles to Failure	Predicted Cycles to Failure	Ratio Predicted: Measured
High Road	1a	11.8	6,402	126.1	703,000	1,786,000	2.5
	1b	13.0	6,151	126.3	1,220,000	1,904,000	1.6
	1c	12.2	6,424	126.7	1,220,000	1,734,000	1.4
	2a	12.2	7,765	189.6	2,213,000	164,000	0.07
	2b	12.4	8,159	190.1	3,612,000	148,000	0.04
	2c	13.0	8,251	189.8	4,103,000	146,000	0.04
	3a	10.1	4,900	251.1	4,700	92,300	20
	3b	9.2	5,882	251.3	6,800	66,200	10
	3c	8.5	4,632	251.3	8,000	101,800	13
Nicholson Road	1a	12.4	2,465	124.7	3,010**	6,240,000	2,073
	1b	11.3	1,319	124.9	4,490**	19,077,000	4,249
	1c	11.6	4,286	124.9	7,600**	2,287,000	301
	2a	10.8	3,567	124.9	63,400**	3,183,000	50
	2b	11.6	3,295	125.1	174,900**	3,642,000	21
	2c	10.2	4,722	125.0	1,188,500**	1,913,000	1.6
	3a	12.4	2,365	252.1	3,700**	199,100	54
	3b	12.1	4,473	251.6	10,000**	63,900	6.4
	3c	12.0	3,461	251.5	369,000**	101,500	0.3
John Street	1a	14.5	2,029	126.6	663,8000	8,213,000	1.2
	1b	14.7	2,100	126.8	12,000,000*	7,659,000	0.6
	1c	14.2	3,312	125.3	14,200,000*	3,580,000	0.3
	2a	14.2	2,735	251.8	510,000	154,200	0.3
	2b	14.8	1,544	251.4	2,414,000	435,000	0.2
	2c	13.8	1,183	251.4	3,492,000	702,000	0.2
	3a	16.0	681***	400.2	1,390	185,700	134
	3b	15.4	1,655	400.2	5,500	37,500	6.9
	3c	15.3	1,804	400.2	25,600	32,100	1.3

* These values have been estimated due to the test being terminated before failure
** These values are effected by the locating of the test slabs on a longitudinal joint
*** This value differs significantly from the remainder of the values for this material and should be eliminated from analysis.

The results of fatigue tests indicate that there is considerable variation between the fatigue life predicted by the model, and the measured values.

In addition to fatigue testing, the ARRB Transport Research laboratory also undertook tests on cores for unconfined compressive strength, indirect tensile modulus and compressive modulus. At the time of testing, samples were approximately 5 months old. Cores tested were 100mm diameter taken from the top of the pavement. The mid 50mm was monitored during the UCS testing and the cores were cut to approximately 60mm for ITS testing. These results are summarised in Tables 13 and 14.

Table 13 Compressive modulus & unconfined compressive strength test results.

Road	Sample	Air Voids (%)	Compressive Modulus (MPa)	UCS (MPa)
High Road	1	11.9	6270	4.36
	2	12.6	3310	4.43
	3	12.5	8800	4.55
	4	13.2	5290	3.87
Nicholson Road	1	14.2	6290	4.53
	2	12.3	2320	4.54
	3	11.2	3150	4.53

Table 14 Indirect tensile modulus laboratory results.

Road	Sample	Air Voids (%)	Total Strain (με)	Rise Time (ms)	Modulus (MPa)
High Road	1	12.6	32.0	46.6	10140
	2	13.9	25.6	46.8	12540
	3	11.8	20.7	46.0	15030
John Street	1	16.2	51.3	44.0	2380
	2	15.8	51.7	43.0	1830
	3	15.7	50.6	44.2	1220
	4	15.6	48.3	44.4	1320
Nicholson Road	1	10.1	51.0	38.6	4490
	2	13.6	44.4	39.6	6240
	3	10.2	51.4	39.6	4710

Note: UCS tested at 25^0C to AS1141.51 (1996) on Unsoaked samples
ITM tested at 25^0C to AS2891.13.1 (1995)
Air Voids tested to AS2891.7.1, AS2891.8, AS2891.9.2 (1993a, 1993b,1993c)

Discussion

In the design process, it was assumed that the flexural modulus of the stabilised layer would be 2400 MPa, and that the fatigue life of the stabilised material would be equivalent to asphalt. Whilst there was some confidence in the assumed modulus value based on the initial Mobil laboratory test results, the assumption that the asphalt fatigue equation could be used with confidence was considered questionable. Asphalt is a manufactured product made to close grading tolerances, and it was considered that the fatigue life of an insitu stabilised material could be quite variable due to grading differences.

In addition, the most often reported modulus value for a stabilised material is the indirect tensile modulus as determined in the MATTA device, rather than the flexural modulus required in the mechanistic design process.

The pavement materials stabilised fell into two categories, namely:

- High Road and Nicholson Road were both composed of a mixture of limestone, asphalt and a crushed aggregate base course, either granite road base or Ferricrete, a crushed laterite product.
- John Street was composed of a mixture of asphalt with a rounded, fine graded natural gravel, and there was a noticeable difference in the modulus of the two materials.

In the analysis of High Road, and Nicholson Road results, there were no statistically outlying values.

As the requirement is for a minimum flexural modulus, an analysis of the flexural modulus for the combined testing of Nicholson Road and High Road given in Table 12 gives minimum modulus of 4,134 MPa at 20^0C using a 1-tailed 95% confidence limit for the upper 100mm of the stabilised pavement layer.

As there is some doubt about the Nicholson Road values due to the longitudinal joint discontinuity, an analysis of the High Road results only gives a minimum flexural modulus of 5,782 MPa at 20^0C using a 1-tailed 95% confidence limit for a mixture of limestone and crushed granular material for the upper 100mm of the stabilised pavement layer.

In the analysis of John Street, statistical analysis showed the presence of upper and lower outlying values. Analysis has been performed both including and excluding outliers. The 95% confidence value for minimum flexural modulus was determined as 1,463 MPa, all values considered, and 1,588 MPa (outliers excluded) at 20^0C.

It is also important to note that the beams were cut from the slabs near the top of the slab, and as such relate to the 0 mm to 100 mm depth in the pavement. It is apparent that the material shape may have a significant effect on the flexural modulus of the pavement.

However, as noted in Table 9 there is a density and modulus profile with depth in all of the test cores, and in Perth, the design for the stabilised pavement, of which the asphalt surface is an integral part, is based on a Weighted Mean Annual Pavement Temperature of 30^0C. Therefore, it is required to adjust the flexural modulus determined above from 20^0C to 30^0C.

In order to determine the likely effect on the pavement performance, and in the absence of actual values for flexural modulus at 30^0C and other levels in the pavement, it is proposed that the flexural modulus be proportioned by factors determined by the indirect tensile modulus testing undertaken in the MATTA device.

From analysis of Tables 9 and 10, the following statements are made:

The indirect tensile modulus values in Table 9 results in a 95% confidence value of 9,195 MPa for Nicholson Road and High Road and 3,845MPa for John Street at 25^0C.

The indirect tensile modulus values in Table 10 results in a 95% confidence value of 8,320 MPa for Nicholson Road and High Road and 3,084 MPa for John Street at 30^0C.

The mean modulus of the 100 to 200mm layer is 74% of the top 0 to –100 mm layer.

The mean modulus of the > 200 mm layer is 46% of the top 0 to –100 mm layer.

The flexural modulus was determined at a temperature of 20^0C. A linear regression was undertaken on the indirect tensile modulus values recorded at 20^0C and 30^0C to project a modulus at 20^0C. Thus the modulus at 30^0C is estimated to be 83% of the 20^0C value for High and Nicholson Roads, and 67% for John Street, and the assumption is made that the ratio for flexural modulus will be similar.

Based on the above observations and analysis of the test results the proposed design flexural material modulus for analysis in CIRCLY is listed in Table 15. The values in Table 15 should be compared to the value adopted (2400 MPa) during the design phase.

Table 15 Proposed flexural modulus of pavement sections.

Pavement Temperature	20^0C			30^0C		
Depth	0-100	100-200	200-300	0-100	100-200	200-300
	Flexural Modulus (MPa)					
Nicholson and High Roads	5,800	4,200	2,700	4,800	3,600	2,200
John Street	1,600	1,200	700	1,000	800	500

A most significant finding is that the measured fatigue life of the test blocks exhibit great variation, and due to the sighting of the Nicholson Road test blocks on a longitudinal joint, these values are considered suspect. Thus with the wide scatter and limited number of results, no definitive relationship could be obtained.

However, it is apparent when viewing the data in Table 12, that with exclusion of the Nicholson Road results, the measured fatigue life varies from 0.05 to 25 times that predicted by the asphalt model, and these results indicate that the asphalt model is not a reliable indication of performance.

The results of the investigation also indicate that to use a single modulus value for total pavement thickness does not reliably predict the pavement performance. It is also apparent that material characteristics play an important role in the modulus values obtained. In order to ascertain the effect that these finding have on the predicted design life, the flexural modulus values from Table 15 at 30^0C were input into CIRCLY for High Road.

Based on a worst case scenario of 0.05 times the asphalt fatigue life, and dividing the pavement into three sub-layers of flexural moduli assumed in Table 15, the primary pavement life is determined by fatigue failure of the bitumen stabilised layer at 1.1×10^6 ESAs. This compares to the design 30 year predicted traffic of 2.6×10^6 ESAs. However following fatigue of the stabilised layer, the pavement has a secondary life performing as an unbound granular pavement. Whilst a modulus for the fatigued material is unknown, it is considered that the modulus will be greater than an unbound granular material.

Assuming a modulus of 1,000MPa for the fatigued stabilised layer, the secondary failure mode by compressive failure of the subgrade following failure of the bitumen stabilised layer of a further 5.8×10^6 ESAs. Thus the total predicted life of High Rd is 6.9×10^6 ESAs which is greater than the design traffic. To provide for the full design life in the primary failure mechanism, the stabilised thickness would need to be increased from 225mm to 275mm.

A similar analysis for Nicholson Road gives a first stage pavement life of 6.0×10^6 ESAs, and a secondary life after fatigue of the stabilised layer of 9.2×10^7 ESAs compared to a design 30 year predicted traffic of 1.9×10^7 ESAs. To provide for the full design life in the primary failure mechanism, the stabilised thickness would need to be increased from 320mm to 400mm.

In the case of John Street, inserting the modulus values from Table 15 into CIRCLY, and assuming a pavement fatigue life of 0.18 times that of asphalt, being the lowest ratio determined in John Street, gives a first stage pavement life of 2.8×10^5 ESAs, compared to a design 30 year prediction of traffic of 5.1×10^6 ESAs.

Due to the rounded particle shape, an assumed value of 500MPa is used for the fatigued stabilised layer. This gives the secondary failure mode by compressive failure of the subgrade of 2.0×10^6 ESAs. Thus the predicted life of John Street is 2.3×10^6 ESAs, significantly less than the design period. This pavement would be predicted to show signs of deformation at 18 years.

An analysis of Table 7 shows that by 4 months, the stabilised pavement shows a high stiffness, with very low deflection values and curvature values. This would indicate the potential for a considerable increase in the asphalt fatigue life. The deflections obtained are consistent with those obtained on full depth asphalt pavements constructed elsewhere in the City of Canning.

An additional benefit of foamed bitumen stabilisation is that the pavement is expected to be far less moisture susceptible than conventional granular base material.

It is apparent from FWD testing that there is a strength gain with time, and as such consideration of a standard curing time for testing of laboratory samples should be given in the test method.

Finally, the total cost savings for the three projects amounted to an estimated 16% of the granular rehabilitation method. More importantly, the savings in construction time and associated inconvenience to business operations and the motoring public is estimated at 88%, based on the savings in construction time.

Conclusion

It is concluded that the insitu foamed bitumen stabilisation technique is a cost-effective tool for the rehabilitation of road pavements. The technique offers a considerable saving in construction time, thus reducing inconvenience and financial losses to adjacent business operations and to the road users, as well as the reduction in risk of accidents to road users and pedestrians.

The cost reductions result from the ability to improve the performance of a granular pavement without increasing the pavement thickness, thus allowing the maintenance of existing kerbs, crossovers and other fixtures.

The testing undertaken indicates that there is considerable variation in fatigue performance within the material, and that the recommended design model of using the asphalt fatigue equation may not valid. Further research to validate this conclusion and to determine the existence of a unique equation is required.

There was a considerable difference in flexural modulus values obtained between pavements produced by stabilising rounded natural gravels as opposed to pavements produced by stabilising pavements composed of crushed rock materials. Whilst it would be unwise on the basis of one sample site to generalise, this indicates the importance of testing pavements composed of different materials.

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