The re-use of existing Bituminous Stabilised Materials for the rehabilitation of National Route 7 - Case Study

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***Synopsis* – Western Cape Government's (WCG) Department of Infrastructure recognised the need to rehabilitate the existing N7 (TR11/1) in Cape Town between the Bosmansdam Interchange (km 2.00) and the Melkbosstrand Interchange (km 18.02). WCG tasked the project team with developing a cost-effective, sustainable solution that would minimize disruptions to the traffic and local community during the rehabilitation of the N7. In response, an innovative approach to reprocess the existing bitumen stabilised material (BSM), which behaved somewhere between a published BSM and cement stabilised material, as well incorporating the milled off asphalt for use in the base layer was proposed. The proposal effectively meant re-processing the base material for a third time thereby providing a "third-life-cycle rehabilitation".**

This paper will focus on the materials investigation undertaken on the project that led to the design proposal, design process followed at the time, pavement life cycle evaluation and relook at the design post publication of the TG 2, 2020 BSM transfer function.

Keywords—recyling; project life cycle; bitumen stabilised materials; BSM; reclaimed asphalt; RA; asset preservation; third-life-cycle rehabilitation

1. INTRODUCTION

The Western Cape Government's (WCG) Department of Infrastructure recognised the need to rehabilitate the existing N7 (TR11/1) in Cape Town between the Bosmansdam Interchange (km 2.00) and the Melkbosstrand Interchange (km 18.02) as part of their broader goal to ultimately upgrade the N7 to freeway standards. The existing N7 is a dual-lane, dual carriageway road and is situated in the jurisdiction of the Paarl District Roads Engineer (DRE) within the Cape Metro area.

The project was subdivided into three distinct work packages. Package 1 consisted of the periodic maintenance in the form of milling and replacing the existing asphalt surface from the Bosmansdam Interchange (km 2.00) to the Refinery Interchange (km 5.40). The second package included the vertical realignment of the N7 between the Refinery Interchange (km 5.40) and the Potsdam Interchange (km 8.40) to accommodate two new underpass structures which would provide future access under the N7 due to the planned eastward expansion of the Du Noon area. The third package consisted of the rehabilitation of the N7 from the Potsdam Interchange (km 8.40) to the Melkbos Interchange (km 18.02).

This paper will focus on the third package, which includes the rehabilitation of the existing slow lane pavement base and surfacing layers on both carriageways.

2. EXISTING TRAFFIC

Based on the 2016 traffic count data, this portion of the N7 carries between 10 000 and 13 250 vehicles per day per carriageway with an approximate growth rate of 4.6 % per annum. The heavy vehicles on this section of road are approximately 13.6 % of the total vehicles, ranging from 872 to 1698 heavy vehicles per day. The growth rate of the heavy vehicles is approximately 2.2 % per annum. The traffic on each carriageway was fairly evenly distributed with a heavy vehicle lane distribution of approximately 89.0 % in the slow lane and 11.0 % in the fast lane.

An existing weighbridge facility is located north of the Vissershok Landfill (km 13.00) thus limiting potential overloading through load control enforcement on the northbound carriageway only. Heavy vehicle traffic in the southbound direction is not able to gain access to the weighbridge facility due to current geometric constraints on the roadway. Based on the traffic count data, historic traffic data, projected gross domestic product forecasts and future expansion around the N7, the cumulative design traffic targets in million equivalent standard (80kN) axles (MESA) were determined and are presented in Table 1.

Table 1: Cumulative design traffic forecast

Description		**Design traffic for 5-year design period (MESA)**	**Design traffic for 10-year design period (MESA)**	**Design traffic for 20-year design period (MESA)**
1a	Low range E80 assumption	From ± 5.11	From ± 11.18	From ± 37.38
1b	High range E80 assumption	To ± 10.23	To ± 27.39	To ± 58.59
2	**Design Value (Recommended)**	**10**	**20**	**45**

3. EXISTING PAVEMENT AND REHABILITATION HISTORY

The N7 from the Bosmansdam Interchange (km 2.00) to the Refinery Interchange (km 5.40) was originally constructed in 1963 with the remainder of the section from the Refinery Interchange to the Melkbos Interchange (km 18.02) being constructed in 1982. The original pavement consists of a G3 base layer varying between 150 mm and 200 mm in thickness, constructed on a 150 mm thick G5 quality subbase and 250 mm thick G8 quality selected subgrade. The in-situ subgrade consists of a G9 quality material.

Subsequent to the original construction, numerous periodic maintenance interventions have been undertaken on various sections of the road. In 2002, the WCG undertook the rehabilitation of the existing slow lane from the Refinery Interchange to the Vissershok Landfill (km 11.20) on the northbound carriageway, and from the Melkbos Interchange to the Refinery Interchange on the southbound carriageway. An in-situ foam bitumen stabilised material (BSM) was used for the base layer. The in-situ material was treated to a depth of 250 mm with a new 35 mm asphalt layer placed over the BSM and an ultra-thin friction course (UTFC) surfacing placed as the final wearing course.

Subsequent to this in 2007, the slow lane of the northbound carriageway from the Vissershok Landfill–to the Melkbos Interchange was rehabilitated using an in-situ emulsion BSM with a new 35 mm asphalt layer and an UTFC surfacing final wearing course.

It should be noted that this portion of the N7 formed part of a BSM research trial section with the pavement being tested with the Council for Scientific and Industrial Research's (CSIR) Heavy Vehicle Simulator (HVS). The mix design for this BSM was reported to be 1.0 % cement and 2.3 % foamed bitumen [1].

HVS testing commenced on the N7 in September 2002 and was undertaken in two phases. The first phase involved testing with high wheel loads between 80 kN and 100 kN while the second phase of testing was undertaken with a standard wheel load of 80 kN [2]. At the conclusion of the first phase of testing, compression of 3mm was measured in the UTFC however it was still considered stable. Additionally, no crushing was visible with the multi-depth deflectometer indicating that the permanent deformation took place mainly in the subbase [2]. It was reported that a total of 6 mm of rutting was measured following 10.0 MESA loading [3].

4. VISUAL INSPECTION

As part of the inception phase of the project, a detailed visual assessment was carried out on the N7 in 2016. The fast lanes were considered to be a good condition with minimal defects observed however, the slow lane exhibited severe distress and was considered to be in a very poor condition. Notably, the portion of road on the northbound carriageway from the Visserhok Landfill to the Melkbos Interchange was observed as being in a better condition to that of the adjoining southbound carriageway and remaining northbound carriageway sections. The visual evaluation highlighted that the southbound carriageway exhibited significantly more distress than the northbound carriageway and was considered to be in a much poorer structural condition. This was attributed to two main factors, the first being the rehabilitation of the northbound carriageway from the Visserhok Landfill to Melkbos Interchange was carried out much later, in 2007, with this section of the pavement being approximately 5 years younger. The second factor was the location of the weighbridge within this section of road. With active overload control being undertaken regularly by the authorities at the weighbridge, the northbound pavement loading was assumed to be lower with less overloading likely on this carriageway compared to the southbound carriageway.

The structural design capacity of the BSM constructed in 2002 was estimated to be 9.0 MESA [1]. An evaluation of the actual traffic loading on this portion of road indicated that the pavement had carried approximately 17.8 MESA by 2016 which is significantly higher than the original anticipated design loading. The portion of road rehabilitated in 2007 carried approximately 9.5 MESA and hence had significantly less visible defects.

The main defects observed were severe transverse cracking, rutting with associated crocodile cracking and pumping, and block cracking which are not typical distress attributes associated with a BSM base layer. Fig. 1 illustrates these typical defects.

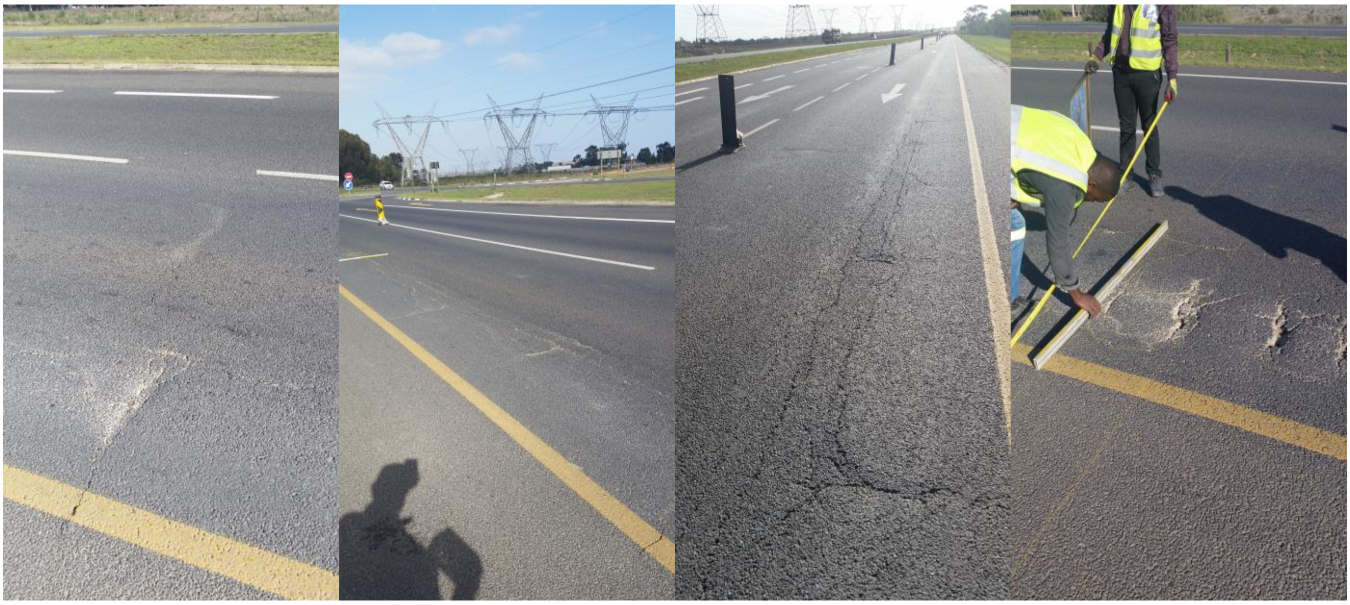

Fig. 1. Typical defects observed on the N7

Long-term pavement performance monitoring on the N7 conducted by the CSIR indicated the presence of block cracks, longitudinal cracks as well as “other cracks” which could not be well defined with a substantial increase in crack lengths recorded over a 6-month period from May 2015 to November 2015 [4]. Although the road had significantly exceeded the structural design capacity of the pavement structure with severe defects present, the type of defects observed were not typical when compared to published BSM distress and failure mechanisms. The type of distress observed during the detailed visual assessment thus led to an in-depth materials investigation to establish the cause of the failures observed and the overall poor condition of the road.

5. PAVEMENT INVESTIGATIONS

Due to the type of pavement distress observed during the visual evaluation, a detailed testing plan was developed to investigate the failure mechanisms. The investigations included the excavation of no less than 20 test pits, 8 test trenches, the extraction of numerous cores and dynamic cone penetrometer (DCP) tests on both carriageways. In addition, monotonic and dynamic triaxial testing was conducted by Stellenbosch University on the cores extracted from the existing pavement as part of a study being undertaken by a post graduate master's student. The data obtained from the testing would be utilised in the development of the revised BSM transfer function to be published in an upcoming revision of the TG2 Bitumen Stabilised Materials manual [5]. Test positions were identified based on the falling weight deflection (FWD) data combined with the visual assessment data. Test areas included areas which were in a sound condition, areas in a poor condition and areas displaying specific defects such as transverse cracking, rutting and block cracking.

Early in the materials investigation and testing phase, it became evident that the existing base layer was not exhibiting typical BSM behaviour. Transverse cracks were observed as reflecting through the existing BSM base layer which exhibited high cement contents based on the phenolphthalein field test, as depicted in Fig. 2.

Fig. 2. Materials testing on the N7

Trenches were excavated across the slow lane to assess the uniformity of the pavement layer depth and mixing consistency. Phenolphthalein sprayed across the face of the excavation revealed pockets of active cement in the base layer in certain areas. This indicated a significant amount of active cement still present in the BSM layer. This could be attributed to either excess cement (above 1.0 %) in the BSM layer or poor distribution of the cement within the BSM layer during construction. A typical example of this is depicted in Fig. 3.

Fig. 3. Typical example of active cement pockets within the BSM layer on the N7

The as-built materials records were interrogated to determine if additional cement, in excess of the design requirements of 1%, was in fact placed within the BSM layer. The as-built data records revealed a consistent cement content of 1.0 % and a foamed bitumen content of 2.3 % for the BSM foam sections. For the BSM emulsion sections a 1.0 % cement content and a 3.0 % bitumen emulsion content were recorded. The rehabilitation of the road was undertaken by means of in-situ recycling and it was concluded that the BSM may not have been mixed sufficiently at the time of construction resulting in pockets of active cement observed within the BSM layer. It was further concluded that the BSM layer was behaving somewhere between a flexible BSM and a stiffer cement treated material. This was evident in the variability in the defects observed on the roadway which included deformation (which is typical of a BSM material), to block cracking commonly associated with cement treated base materials.

Further to the standard testing undertaken as part of the field investigations, a Master of Engineering student at the Stellenbosch University carried out a study on the N7 BSM base materials [5]. This study included evaluating the historic FWD data and then extracting cores at specific locations to determine the shear properties of the material through monotonic triaxial testing and the residual resilient modulus values through dynamic triaxial testing.

The outcome of the monotonic triaxial testing indicated that the BSM foam sections had a variation in cohesion of between 171.1 kPa and 528.7 kPa and a variation in the friction angle between 33.5 degrees and 49.9 degrees. The BSM emulsion sections of road had a variation in cohesion between 293.2 kPa and 423.6 kPa and a variation in the friction angle between 38.3 degrees and 56.4 degrees [5]. In addition, samples within the same area also exhibited a high variation in test result data. A variation in failure mechanism observed during testing was also recorded which included micro cracking at the bottom of the layer indicating a typical cement stabilized material failure and deformation of the specimens indicating a typical BSM failure [5].

Based on the dynamic triaxial test results, the samples yielded a high stiffness ranging from 1400 MPa to 2300 MPa which fall well outside of the typical stiffness ranges for BSM's (typically between 600 MPa and 1000 MPa) [6]. Based on the findings of the study, it was concluded that the existing BSM base layer on the N7 did not fully behave like a typical BSM base [5]. This study confirmed the observations, assumptions and conclusions made during the materials investigation phase of the project.

6. Design Proposals

The findings of the materials investigation concluded that the existing BSM base layer had active pockets of cement present as a result of poor mixing during construction. This resulted in an uncharacteristic behaviour of the BSM base layer which was not as anticipated.

The project team was tasked with developing a cost-effective, sustainable solution that would minimize disruptions to the traffic and local community on the N7.

During the design phase of the project, WCG were undertaking a study for the proposed upgrading of the N7 to freeway standards. The outcome of the study included upgrades to numerous interchanges on the route and well as options to realign a portion of the N7 between Refinery Interchange and Potsdam Interchange to allow for two underpass bridge structures for future access and development needs for the City of Cape Town to the east of the N7. This particular project fell within these project limits. As the existing N7 surface had numerous defects and was in a poor condition, the routine road maintenance team were unable to effectively maintain the road. The employer requested an investigation into various maintenance and rehabilitation options taking these future upgrades into consideration. The investigation would involve looking into holding actions (short term) and light rehabilitation (medium term) options to allow time for the future upgrade options on the N7 to be finalised. Full rehabilitation options were also included should the future N7 upgrade design be concluded timeously or works packages separated. It was agreed with the employer, that an analysis period of 20 years using the Present Worth of Cost (PWOC) method would be used to assess the various options as identified. Eight life cycle costing options were generated based on three initial intervention types, namely Options A, short term initial interventions with an estimated design life of less than 5 years, Options B, medium term initial interventions with an estimated design life not exceeding 10 years and Options C, long term initial interventions with a 20-year design life.

A summary of the eight options identified indicating the various 1st interventions with the proposed 2nd and 3rd interventions are summarised in Table 2.

Table 2: Life cycle costing interventions

Option	1st Intervention Option	Year (Life)	2nd Intervention Option	Year (Life)	3rd Intervention Option	Year (Life)
A1	Mill and replace asphalt surfacing	0 (3)	Rework in-situ base layer (C4) with an asphalt base course overlay, final asphalt surfacing and UTFC (as per Option B1 1st Intervention)	3 (10)	Mill and replace asphalt surfacing and UTFC	13 (8) (7-year cost pro-rata)
A2	Mill and replace asphalt surfacing	0 (3)	Rework in-situ base layer (C4) with an EME overlay, final asphalt surfacing and UTFC (as per Option C1 1st Intervention)	3 (12)	Mill and replace asphalt surfacing and UTFC	15 (8) (5-year cost pro-rata)
A3	Mill and replace asphalt surfacing	0 (3)	Off-site recycling (BSM) in slow lane with sacrificial asphalt surfacing, final asphalt surfacing and UTFC (as per Option C3 1st Intervention)	3 (12)	Mill and replace asphalt surfacing and UTFC	15 (8) (5-year cost pro-rata)
B1	Rework in-situ base layer (C4) with an asphalt base course overlay, final asphalt surfacing and UTFC	0 (10)	20 % Patching and mill and replace asphalt surfacing and UTFC	10 (8)	20 % Patching and mill and replace asphalt surfacing and UTFC	18 (8) (2-year cost pro-rata)
B2	Rework in-situ base layer (C4) with a crushed stone base overlay, final asphalt surfacing and UTFC	0 (8)	20 % Patching and mill and replace asphalt surfacing and UTFC	8 (8)	20 % Patching and mill and replace asphalt surfacing and UTFC	16 (8) (4-year cost pro-rata)
C1	Rework in-situ base layer (C4) with an EME overlay, final asphalt surfacing and UTFC	0 (12)	Mill and replace asphalt surfacing and UTFC	12 (8)	None	
C2	Rework in-situ base layer (C4) with a BSM overlay, final asphalt surfacing and UTFC	0 (12)	Mill and replace asphalt surfacing and UTFC	12 (8)	None	
C3	Off-site recycling (BSM) in slow lane with sacrificial asphalt surfacing, final asphalt surfacing and UTFC	0 (12)	Mill and replace asphalt surfacing and UTFC	12 (8)	None	

All options investigated were costed including the roadworks and ancillary works as identified. Table 3 lists the major costing items that were used to determine the estimated project construction costs for each option.

Table 3: Unit rate assumptions

Item	Description	Unit	Rate
13.01 (c)	Time-related obligations	month	R 555,000.00
16.03 (c)	Overhaul on material	m^3km	R 10.50
32.03 (a)	Single stage crushing to maximum fraction size of 20mm	m^3km	R 110.00
35.08 (c)	Bituminous stablizing agent (70/100 penetration grade bitumen)	t	R 5,500,00
35.02 (a)	Cement (CEM II B 32.5 N)	t	R 2,200.00
35.08 (c)	Road Lime	t	R 1,900.00
36.01 (c)	Crushed-stone base (G2)	m^3	R 550.00
38.02 (c)	Milling out bituminous and bituminous stabilised Material <100mm	m^3	R 800.00
38.02 (c)	Milling out bituminous and bituminous stabilised Material >100mm	m^3	R 600.00
42.01 (a)	Asphalt Base (50/70 penetration grade bitumen)	t	R 950.00
42.01 (a)	EME Base	t	R 1,800.00
42.02 (a)	Asphalt surfacing (50/70 penetration grade bitumen)	t	R 950.00
42.02 (a)	Asphalt surfacing (A-E2)	t	R 1,000.00
42.02 (a)	Asphalt surfacing (A-R1 / A-R2)	t	R 1,200.00
90.02	Ultra-Thin Friction Course	m^2	R 140.00
F10.01	BSM 1	m^3	R 950.00

A summary of the estimated project costs for each option per square metre is presented in Table 4.

Table 4: Estimated project construction cost per square metre

Option	Descriptions	Cost (per m^2)
A1, A2, A3	Mill and replace asphalt surfacing	R 313.24
B1	Rework in-situ base layer (C4) with an asphalt base course overlay, final asphalt surfacing and UTFC	R 735.29
B2	Rework in-situ base layer (C4) with a crushed stone base overlay, final asphalt surfacing and UTFC	R 652.94
C1	Rework in-situ base layer (C4) with an EME overlay, final asphalt surfacing and UTFC	R 876.47

Option	Descriptions	Cost (per m^2)
C2	Rework in-situ base layer (C4) with a BSM overlay, final asphalt surfacing and UTFC	R 1147.06
C3	Off-site recycling (BSM) in slow lane with sacrificial asphalt surfacing, final asphalt surfacing and UTFC	R 764.71
	Mill and replace asphalt surfacing and UTFC	R 441.18
	20 % Patching and mill and replace asphalt surfacing and UTFC	R 500.00

The eight options investigated including the maintenance interventions presented in Table 2, together with the costing calculated per intervention in Table 4 were used to generate the life cycle PWOC. In order to ensure a 20-year design horizon for comparison, the 3rd intervention costs were pro-rated. A summary of the analysis is tabulated in Table 5.

Table 5: Life cycle present worth of cost (PWOC)

Option	**Capital Cost (R/m^2)**	**Life (Yrs)**	**PWOC at Discount Rate (R/m^2)**			**Maintenance Cost (R/m^2)**	**Life (Yrs)**	**PWOC at Discount Rate (R/m^2)**			**Maintenance Cost (R/m^2)**	**Life (Yrs)**	**PWOC at Discount Rate (R/m^2)**			**PWOC (R/m^2)**			**Total Life (Yrs)**
			6 %	**8 %**	**10 %**			**6 %**	**8 %**	**10 %**			**6 %**	**8 %**	**10 %**	**6 %**	**8 %**	**10 %**	
A1	313.24	3	263.00	248.66	235.34	735.29	10	410.58	340.58	283.49	386.03	5	256.73	225.24	198.09	930.31	814.48	716.92	**20**
A2	313.24	3	263.00	248.66	235.34	876.47	12	435.58	348.06	279.27	275.74	5	206.05	187.66	171.21	904.62	784.38	685.82	**20**
A3	313.24	3	263.00	248.66	235.34	764.71	12	380.04	303.68	243.66	275.74	7	206.05	187.66	171.21	849.08	739.99	650.21	**20**
B1	735.29	10	410.58	340.58	283.49	500.00	8	313.71	270.13	233.25	125.00	2	111.25	107.17	103.31	835.54	717.89	620.05	**20**
B2	652.94	8	409.66	352.76	304.60	500.00	8	313.71	270.13	233.25	250.00	4	198.02	183.76	170.75	921.39	806.66	708.61	**20**
C1	876.47	12	435.58	348.06	279.27	441.18	8	276.80	238.35	205.81	0.00	0	0.00	0.00	0.00	**712.38**	**586.41**	**485.08**	**20**
C2	1 147.06	12	570.05	455.51	365.49	441.18	8	276.80	238.35	205.81	0.00	0	0.00	0.00	0.00	**846.85**	**693.87**	**571.30**	**20**
C3	764.71	12	380.04	303.68	243.66	441.18	8	276.80	238.35	205.81	0.00	0	0.00	0.00	0.00	**656.83**	**542.03**	**449.47**	**20**

Based on the PWOC calculations, Options C1, C2 and C3 (the long-term 1st intervention options) were considered the most cost effective over a 20-year design horizon. This was mainly due to the poor condition of the existing pavement and reduced structural capacity expected for the various short- and medium-term proposals which would still require a light to heavy rehabilitation in the future. The most favourable option was considered to be Option C3, which included off-site recycling (BSM) in the slow lane with a sacrificial asphalt surfacing, then final asphalt surfacing and a UTFC friction course.

Various other technical factors were also considered in combination with the PWOC calculation. The three most viable option are discussed in a more detail. Option C1 and C2 involved the in-situ reconstruction of the existing BSM base layer to form a cement stabilised subbase for EME or crushed stone base overlay. The existing bitumen content and pockets of active cement within the existing BSM base increased the risk of producing a uniform cement stabilised material and possible shrinkage cracking could be expected. The construction of a cement stabilised layer would also increase the duration of temporary road closure required for curing of the layer thus impacting on the road user and traffic. The longer duration of road closure would also result in increased traffic on the fast lanes thus creating more damage to this portion of the pavement. These two options would also require a substantial increase in final road level which would have an impact on the structures along the route. At the time, design and construction of EME bases were largely unknown and could potentially carry a high risk with a road such as the N7.

Option C3 would allow for opening to traffic earlier than Options C1 and C2 through the use of a BSM. Due to the high traffic volumes on the N7, this option would require an additional 20 mm of BSM base which was overlaid with a temporary 20 mm asphalt surfacing. This temporary surface would provide protection to the underlying BSM base and allow for early trafficking of the pavement. This also would alleviate the risk of damage by ravelling of the BSM and the potential for loose aggregate to cause damage to other vehicles should it have been left exposed whist under traffic. The temporary surfacing and the upper 20 mm of the BSM base were later milled off to ensure that a good riding quality could be achieved and provide more control to the final surface levels.

The main concern for this option was that reprocessing of existing BSM base layers are largely unknown and requires further research and investigation. In addition, construction of BSMs within the winter periods in the Western Cape can be problematic in achieving a quality mix without stringers and binder balling as a result of low day time temperatures and persistent rain. This option would however allow for off-site processing of the material which would allow for greater quality control and improved mix quality together with the reuse RA produced from the construction activity thus promoting sustainability.

Following the PWOC analysis and technical feasibility, Option C3 was agreed with the employer for implementation and a separate work package was created where the rehabilitation of the N7 could be undertaken independently of the realignment of the N7. This led to the creation of work Package 3 between the Potsdam Interchange and Melkbos Interchange.

The pavement design is depicted in Fig. 4.

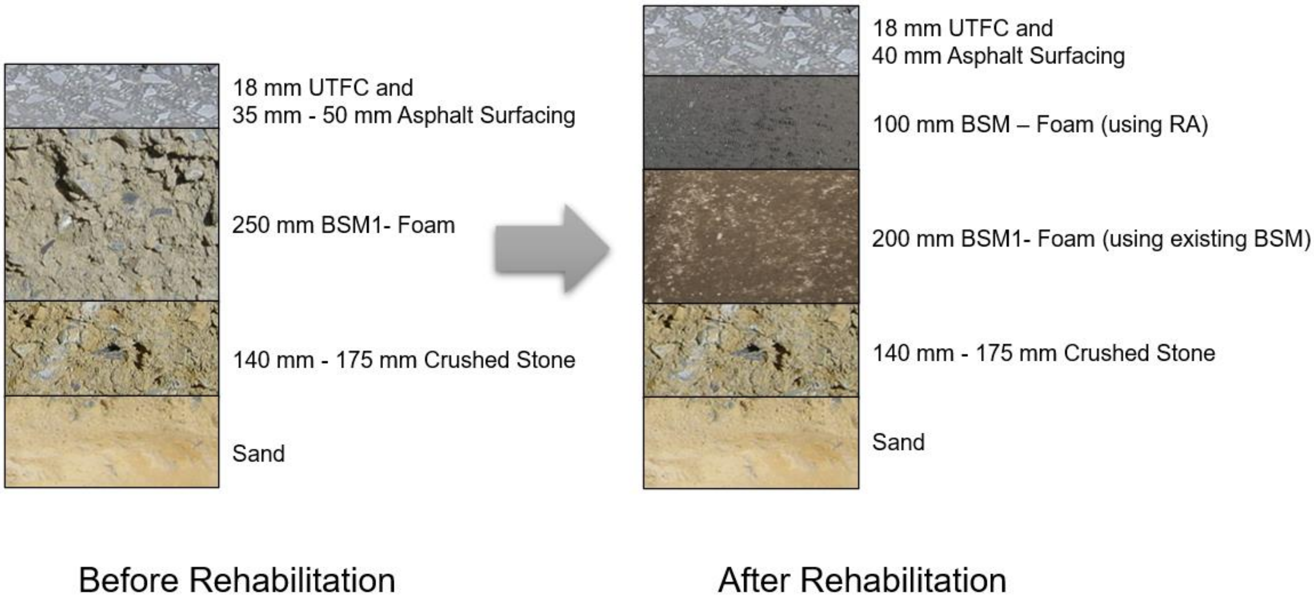

Fig. 4. Proposed pavement design

A critical factor in the design of the BSM option was the use of available transfer functions at the time of the design. Three design methodologies were considered for evaluation. The first was the Pavement Number (PN) method in accordance with TG, 2009 [7]. The second was to consider the BSM as a granular material using the South African Mechanistic-Empirical Design Method (SAMDM) [8] with increased cohesion and friction angle while the third methodology was to use the unpublished transfer function developed through Loudon International [9]. As the design traffic was in excess of 30 MESA, the PN method was considered doubtful to produce a pavement structure that could meet the required design traffic. The PN method was however used to ensure the pavement design proposed did not yield a structural capacity less than 30 MESA. The design equivalent material (DEMAC) was subsequently evaluated for all uniform sections with the resultant output indicating that the proposed pavement structure met the 30 MESA criteria.

The pavement structure was then evaluated using SAMDM as a granular material with a cohesion of 250 kPa and a friction angle of 40 degrees with typical published stiffness values of between 600 MPa and 1000 MPa [6]. This method yielded a predicted structural capacity in excess of 98 MESA for all uniform sections.

The third option involved using the unpublished transfer function from Loudon International. This transfer function was not used directly in the pavement design evaluation but was however used to introduce the concept of the Deviator Stress Ratio (DSR), also known as the Stress Ratio and how it related to the pavement [10]. The Loudon International transfer function was used as one input into the determination of the required DSR to achieve the design traffic of 45 MESA. The transfer function is presented below (1) [9].

$$N = 10^{[A+B(RD)+C(RetC)+D(PS)+E(SR)]} \qquad (1)$$

Where:

RD = Relative Density (%)

RetC = Retained Cohesion (kPa)

PS = Allowed Plastic Strain (%)

SR = Stress Ratio Parameter

A = -3.7 for a category B road

B = 0.1

C = 0.05

D = 0.1

E = -22.3333

The following DSR values and corresponding predicted structural capacities were calculated based on this transfer function and are represented in Table 6 below.

Table 6: Summary of Louden International transfer function life prediction

DSR (%)	Predicted Life (MESA)	Transfer Function	Cat
28.9	100	Loudon International unpublished	B
29.9	60	Loudon International unpublished	
30.4	45	Loudon International unpublished	
31.2	30	Loudon International unpublished	

As can be seen from Table 6, the Loudon International transfer function is sensitive to the DSR. A 2.3 % change in the DSR results in a predicted design traffic range of 70 MESA, based on the given inputs. Discussions with key BSM industry leaders assisted in determining the target DSR value of 30 % (0.3) for the evaluation of the pavement design options. The transfer function developed by Louden International [9] was also used in determining this value which was considered “best practice” at the time the pavement designs were undertaken.

A sensitivity analysis was carried out varying the cohesion, angle of friction and stiffnesses of the BSM layers including potential overloading scenarios associated with traffic in the southbound direction on the N7.

Table 7 summarises the ranges of input used in the sensitivity analysis.

Table 7: Input range for DSR sensitivity analysis

<table>
<tr><th>Parameter</th><th>Low</th><th>Probable</th><th>High</th></tr>
<tr><td>Cohesion
(kPa)</td><td>200</td><td>250</td><td>300</td></tr>
<tr><td>Angle of friction
(˚)</td><td>38</td><td>40</td><td>45</td></tr>
<tr><td>Stiffness
(MPa)
BSM1 (Upper 100 mm)</td><td>600</td><td>800</td><td>1000</td></tr>
<tr><td>Stiffness
(MPa)
BSM1 (Lower 200 mm)</td><td>400</td><td>600</td><td>800</td></tr>
<tr><td>Stiffness
(MPa)
Subbase (140mm to 175mm)</td><td colspan="3">170 to 250</td></tr>
<tr><td>Stiffness
(MPa)
Subgrade</td><td colspan="3">90 to 120</td></tr>
<tr><td>Wheel Load
(kN)</td><td colspan="2">20</td><td>30</td></tr>
</table>

The data was input into the Rubicon Toolbox Software for each combination of stiffnesses included in Table 7 and the principal stresses obtained using the Rubicon Toolbox stress strain calculator [9]. This data was then input into a spreadsheet whereby the DSR values were calculated for each combination of cohesion, angle of friction and principal stresses.

A summary of the analysis for the most critical uniform section, located on the northbound carriageway, for a 20 kN and 30 kN wheel load respectively are tabulated in Table 8 and Table 9.

Table 8: DSR summary for a 20 kN wheel load

Cohesion (kPa)		200			250			300		
Angle of Friction (º)		38	40	45	38	40	45	38	40	45
Stiffness (MPa) BSM Upper	Stiffness (MPa) BSM Lower	DSR (Target < 0.3)								
600	400	0.395	0.377	0.333	0.317	0.303	0.268	0.265	0.253	0.224
600	600	0.362	0.343	0.299	0.295	0.280	0.245	0.249	0.237	0.208
800	400	0.431	0.412	0.366	0.345	0.330	0.293	0.287	0.275	0.244
800	600	0.402	0.382	0.332	0.329	0.312	0.273	0.278	0.264	0.231
800	800	0.375	0.356	0.309	0.307	0.291	0.254	0.260	0.247	0.216
1000	400	0.450	0.431	0.383	0.360	0.345	0.306	0.300	0.287	0.255
1000	600	0.426	0.406	0.358	0.343	0.328	0.289	0.288	0.274	0.243
1000	800	0.397	0.377	0.329	0.324	0.308	0.269	0.273	0.260	0.228

Table 9: DSR summary for a 30 kN wheel load

Cohesion (kPa)		200			250			300		
Angle of Friction (º)		38	40	45	38	40	45	38	40	45
Stiffness (MPa) BSM Top	Stiffness (MPa) BSM Bottom	DSR (Target < 0.3)								
600	400	0.446	0.424	0.371	0.362	0.344	0.302	0.305	0.290	0.255
600	600	0.395	0.373	0.321	0.327	0.309	0.267	0.279	0.264	0.229
800	400	0.499	0.477	0.422	0.401	0.383	0.339	0.335	0.320	0.283
800	600	0.399	0.379	0.333	0.323	0.308	0.270	0.271	0.259	0.228
800	800	0.396	0.374	0.320	0.329	0.311	0.268	0.282	0.267	0.231
1000	400	0.526	0.503	0.447	0.421	0.402	0.357	0.351	0.335	0.298
1000	600	0.457	0.433	0.377	0.373	0.354	0.310	0.315	0.300	0.263
1000	800	0.415	0.391	0.336	0.343	0.325	0.280	0.293	0.278	0.241

Based on the analysis, the design structural capacity could be met for a standard wheel load for all stiffness combinations should the cohesion be greater than 250 kPa and an angle of friction greater than 45 degrees. Should a cohesion of 300 kPa be achieved then a lower friction angle would be sufficient to meet the structural capacity.

In order to use this data practically, mix designs were undertaken on the materials recovered from the materials investigation. It should be noted that the material used for testing was obtained through standard methods of test pitting and not through milling on site. These materials were crushed using a small jaw crusher to attempt to get a representative sample for testing.

The tests carried out are summarised in Table 10.

Table 10: BSM foam test results

Material	Active Filler (%)	Bitumen content (%)	Friction Angle (˚)	Cohesion (kPa)	Retained Cohesion (%)	Class (TG2)
Northbound existing BSM foam	1.0 Lime	2.0	45.8	294	82	BSM 1
Southbound existing BSM foam	1.0 Lime	2.0	43.8	279	82	BSM 1
Northbound existing BSM emulsion	1.0 Lime	2.0	40.4	243	81	BSM 2

The existing BSM foam base layer yielded good results and could be used to produce a BSM 1. The existing BSM emulsion however yielded a very coarse material with little to no fines. It was assumed that additional fines would be produced through the milling process however allowance would be made for blending-in of fines by the addition of crusher dust. The existing RA sampled was contaminated with asphalt from one of the test trenches taken on a patch which included fresh cold mix asphalt mixed together with the old brittle asphalt. It was recommended that additional samples be tested to ensure that the RA could be used to produce the required BSM1. This was however not carried out due to time constraints and an allowance was made in the contract to blend the RA with a G4 quality material as well as a crusher dust and to undertake mix designs with these materials prior to construction.

Based on the friction angle and cohesion of the BSM foam tests (Table 10), the pavement design would yield a DSR lower than 30 % and was deemed sufficient for a 45 MESA structural capacity.

Prior to construction, milled materials obtained from site were tested to determine the shear properties of the materials. Four mix designs were carried out. The first involved a level III mix design using 100 % imported G2 base material. This material would be used to commence construction and allow for the building-up and crushing of the reclaimed materials and the formation of an advance stockpile.

The second level III mix design was carried out on the 100 % RA mix from material obtained from the mill and replace operation taking place on Package 1 (km 2.00 to km 5.40) prior to the commencement of this project. The third level III mix design was carried out on a 90 % RA mix blended with 10 % crusher dust. Once the reclaimed BSM had been milled, crushed and stockpiled, the final level III mix design was carried out on the reclaimed BSM base material. All mix designs were carried out with road lime due to the high active cement content present in the existing BSM base layer.

A summary of the mix design properties and results are included in Table 11.

Table 11: BSM foam test results on milled materials and virgin G2

Material	**Active Filler (%)**	**Bitumen content (%)**	**Friction Angle (˚)**	**Cohesion (kPa)**	**Retained Cohesion (%)**	**Class (TG2)**
Virgin G2	1.0 Lime	2.0	43.3	281	84	BSM 1
100 % RA	1.0 Lime	2.0	40.8 (36)[a]	284	83 (66)[a]	BSM 1
90 % RA and 10 % crusher dust	1.0 Lime	2.0	42.1	276	72	BSM 1
100 % reclaimed BSM	1.0 Lime	2.0	42.4	386	76	BSM 1

[a.] Original test results. Mix design was retested.

Based on the mix design results, three of the mix design exceeded the BSM 1 criteria and fell within the design DSR criteria. The 100 % RA mix design had a much lower friction angle than expected and in addition, had a higher cohesion than the blended RA with crusher dust, which was counter intuitive to the theory. The 100 % RA was retested and found to have an increased friction angle of 40.8 degrees and increased retained cohesion of 83 %. When using the mix design shear properties for the 100 % RA mix, a DSR ranging from 24.4 % to 29.8 % was achieved with the design DSR value (assuming an 800 MPa BSM top layer and a 600 MPa BSM lower base layer) of 27.2 %. All mix designs were deemed sufficient for design purposes.

Based on the test results, 200 cubic metres of the virgin G2 material was implemented on the lower BSM base layer until a sufficient volume of stockpiled material could be generated through milling of the existing BSM base layer. This would be combined with a blend of RA (90 %) and virgin crusher dust (10 %) for the top BSM base layer as this was considered to be a safer design option. During the blending process for the trial mix for the upper BSM layer, inconsistencies in the blended mix including poor material uniformity, segregation, binder balling and excessive binder stringers led to the decision to utilise 100 % RA and to discontinue the blending process. Once a sufficient volume of stockpiled reclaimed BSM was produced, the 100 % reclaimed BSM would be constructed on the lower BSM base layer.

7. Execution

The works included in Package 1 (km 2.00 to km 5.40) and Package 2 (km 5.40 to km 8.40) of the project were tendered as a single contract in July 2018 and awarded in December 2018. This project would provide the necessary RA required for Package 3 between km 8.40 and km 18.02. The project (Package 3) was tendered in August 2018 and awarded in January 2019. Construction work commenced in February 2019 with temporary patching of severely defective areas prioritised to provide an acceptable wearing course for traffic in the interim. Due to numerous construction activities taking place around the Potsdam Interchange simultaneously, a partial practical completion was introduced for the 1st kilometre of the project (km 8.40 to km 9.40) to alleviate construction congestion in this area. This would ensure that the interaction between Package 2 and Package 3, as well as other projects in the area, could be mitigated. Virgin G2 material was imported to stockpile and tested to determine the mix design properties. RA was obtained from package 1 and stockpiled at the processing site where it was crushed and stockpiled.

Fig. 5. Crushing the RA at the Potsdam Interchange

The South-West quadrant of the Potsdam Interchange was used as the stockpile and processing site as all three packages were carried out by the same contractor.

Fig. 6. Stockple and BSM processing area at the Potsdam Interchange

Work on the northbound slow lane commenced with the milling off of the asphalt surfacing and BSM in two separate operations and transporting the milled materials to the stockpile and processing site. During the trial mixing process, blending of the crusher dust and RA proved problematic mainly due to the poor mix consistency, aggregate segregation, excessive balling and binder stringers within the blend and it was agreed to proceed with 100 % RA in the mix for the upper BSM base layer. A more consistent mix was obtained using only the 100 % RA material. This was subsequently implemented for the trial section. The works commenced under long term closures limited to two days production in length so as to limit the effect of moisture damage to the opened road sections. This would also limit the length of closure thus reducing the impact on the traffic.

Fig. 7. BSM processing at the Potsdam Interchange

After placement of the BSM lower and upper base layers, a temporary gap graded asphalt surfacing, 20 mm thick using 50/70 penetration grade bitumen, was placed to allow traffic to use the completed roadway. The temporary asphalt surfacing was placed to prevent surface damage and ravelling to the newly constructed BSM layer given the high traffic volumes on the route and the need to open the lane to traffic. This also allowed the BSM time to cure sufficiently and to reduce the moisture content within the layer, whilst under traffic [11]. Any defects or issues in the constructed layers could also be addressed early before the placement of the final wearing course. The project included an embargo period where no BSM works were allowed to take place during the winter months from May until August. This was based on previous works conducted on Camps Bay Drive where moisture contents increased substantially in the stockpiles during the rainy season which adversely affected the quality of the BSM layer [12].

A detailed construction methodology was included in the tender document which required all remaining asphalt works to be undertaken during the embargo period on the fast lanes thus ensuring that works still progressed during this period.

Fig. 8. 100 % RA BSM upper base layer

On completion of the BSM works, the top 20 mm of the BSM and the 20 mm temporary asphalt surfacing were milled off to ensure a smooth riding quality and better level control of the final asphalt surface. The milled BSM and asphalt were re-used on the weighbridge parking areas located alongside the project. All excess materials generated from Package 3 were used on the Package 2 project. Excess RA, generated from all three projects, was stockpiled for use on other Western Cape Governments projects identified in the area however, this valuable material resource was pilfered by local communities.

Fig. 9. N7 Complete

8. RETROSPECTIVE DESIGN

The pavement design for the Package 3 project was completed in November 2017 with construction commencing in January 2019 and completion in September 2020. The third edition of the TG 2 Technical Guideline: Bitumen Stabilised Materials published in August 2020 included the Stellenbosch published transfer function [6]. This newly developed transfer function was subsequently used to retrospectively re-evaluate the design for Package 3 effectively looking back at the project.

The Stellenbosch transfer function, as published in TG2, 2020 is as follows [6]:

$$LogN = A - 57.286(DSR)^3 + 0.0009159(P_{MDD}.RetC) \quad (2)$$

Where:

A = 1.79873 for a 90 % reliability and 15 mm rut limit

DSR = Deviator Stress Ratio, as a fraction

P_{MDD} = BSM Dry density expressed as a percentage of MDD (%)

RetC = Retained Cohesion (%)

The required DSR was calculated for the various predicted design lives assuming a retained cohesion of 75 %, a dry density of 100 % and a 15 mm rut depth.

Table 12: Summary of Stellenbosch transfer function life prediction

DSR (%)	Predicted Life (MESA)	Transfer Function	Cat
22.7	100	Stellenbosch (TG 2, 2020)	B
25.0	60	Stellenbosch (TG 2, 2020)	
26.1	45	Stellenbosch (TG 2, 2020)	
27.5	30	Stellenbosch (TG 2, 2020)	

As can be seen from Table 12, the Stellenbosch transfer function is less sensitive to the DSR when compared to the Loudon International transfer function (Table 6) but demands a lower DSR for the required design structural capacity. The target DSR for the pavement using this transfer function is 26.1 %, which is significantly lower than what was originally targeted (30 %) at the design stage of the project.

The BSM foam shear properties obtained for the various mixes in Table 11, with corresponding DSR values calculated, were input into (2) and predicted theoretical design structural capacity calculated. A summary of the shear properties, DSR and predicted theoretical design structural capacity for the various mixes is included in Table 13.

Table 13: BSM foam shear properties with corresponding DSR and predicted structural capacity (N)

Material	Friction Angle (˚)	Cohesion (kPa)	DSR (%)	N (MESA)
Virgin G2	43.3	281	25.7	49.3
100 % RA	40.8	284	27.2	32.7
90 % RA and 10 % crusher dust	42.1	276	27.0	34.7
100 % reclaimed BSM	42.4	386	19.6	171.3

Based on the shear properties and corresponding DSR for a 100 % RA mix, which was implemented on site during construction, a predicted theoretical design structural capacity of 32.7 MESA was obtained which, although still classifying as an ES30 pavement, is lower than the required capacity of 45 MESA. It should be noted however that the original design carried out in 2017 was undertaken using the most-up-to-date information, technologies and best practice available at the time. Western Cape Government, Department of Infrastructure will continue to monitor and assess this road asset, in terms of their asset management principles to contribute to the greater civil engineering industry on BSM and feed back into the published BSM transfer function.

9. Conclusion

The consulting team was tasked with developing a cost-effective, sustainable solution that would minimize disruptions to the traffic and local community during the rehabilitation of the N7. In response, an innovative approach to reprocess the existing BSM, which behaved somewhere between a BSM and cement stabilised material, as well incorporating the milled off asphalt for use in the base layer was proposed. The proposal effectively meant re-processing the original base material for a third time thereby providing a “third-life-cycle rehabilitation” of the original base layer. The N7 rehabilitation (Package 3) was seen as a major success and in the recent economic climate, together with limited material resources, this project has successfully set the benchmark for effective and sustainable rehabilitation and maintenance of a road network.

Reprocessing the reclaimed BSM and using high percentages of RA is a relatively new and specialized practice in road rehabilitation in South Africa. The project team in close collaboration with the employer optimised the design of the recycling for the two-layer base solution using existing materials by undertaking extensive testing and developing quality control measures to ensure consistency in the construction process. Even though plant mixed recycling has been undertaken for many years within the industry, a robust plan for knowledge transfer and lessons learnt session between the design team, the engineers site staff and the appointed contractors was implemented to ensure an in depth understanding of the design concepts, reprocessing methodology and construction methodology required for this unique project prior to implementation. The cost savings from reprocessing the existing BSM materials from the road have allowed the employer to fund additional roads projects and assist in reducing the backlog in road maintenance within the Western Cape.

The project involved the recycling of 30 500 cubic meters of existing road material (existing BSM base and asphalt surfacing) and placed back into the road as a new BSM base layer. The asphalt surfacing (approximately 26 000 t) included 20 % RA in the mix. Approximately 32 700 cubic meters of reclaimed material from the existing road was re-processed and placed back into the road primarily as a new BSM base and high RA content asphalt surfacing.

10. ACKNOWLEDGEMENTS

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11. REFERENCES

[1] K.J. Jenkins, S. Robroch, M.G. Henderson, J. Wilkinson and A.A.A. Molenaar. Advanced testing for cold recycling treatment selection on N7 near Cape Town. CAPSA’04, Sun City, South Africa, Septmber 2004.

[2] Sabita. Digest Magazine. Sabita. March 2003.

[3] Stellenbosch University. Pavement Evalauation and Rehabailitation Course, Cold Recycling Rehabilitation Solutions Presentation, Stellenbosch University. February 2012.

[4] J.K. Anochie-Boateng, W.J. Steyn, C. Fisher and L. Truter. A link of Full-Scale Accelerated Pavement Testing to Long-Term Pavement Performance Study in the Western Cape Province of South Africa. The Roles of Accelarated Pavement Testing in Pavement Sustainability pp 67 - 79, September 2016.

[5] C.R. Bierman. A design function for Bitumen Stabilised Material performance based on laboratory and field evaluation, Stellenbosch University. March 2018.

[6] Sabita. Technical Guideline: Bitumen Stabilised Materials Guideline for the Design and Construction of Bitumen Emulsion and Foamed Bitumen Stabilised Materials. Sabita, Technical Guideline TG 2, Third Edition, Pretoria, August 2020.

[7] Asphalt Academy. Technical Guideline: Bitumen Stabilised Materials Guideline for the Design and Construction of Bitumen Emulsion and Foamed Bitumen Stabilised Materials. Asphalt Academy, Technical Guideline TG 2, Second Edition, Pretoria, May 2009.

[8] H.L Theyse, M. De Beer and F.C Rust. Overview of South African Mechanistic Pavement Design Method. Transporation Research Record Journal of Transportation Research Board 1539. January 1996.

[9] Rubicon Solutions, Rubicon Toolbol Software. Rubicon Solutions.

[10] F.M. Long and H.L. Theyse. Mechanistic-Empirical Structural Design Models for Foamed and Emulsified Bitumen Treated Materials. Capsa'04, Sun City, South Africa, September 2004.

[11] D. Collings and K. Jenkins. The long term behaviour of bitumen stabilised materials (BSMs). CAPSA'07, Gaborone, September 2007.

[12] G. Llewellyn, I. Bowker and F. Snijders. Sustainable Road Rehabilitation Solution using Reclaimed Asphalt: Camps Bay Drive Case Study. CAPSA'19, Sun City, South Africa, October 2019.