Rut potential determination of marginal asphalt with low traffic volumes on a regional airport in Southern Africa

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***Abstract*—Asphalt production and paving in remote regions of Africa can be problematic in terms of logistics, material sourcing and production quality. Airport planning also tend to project high growth rates of aircraft movements which don't always realize. A further complication is the tendency to use roads design and specifications on airports in such remote regions. A recent airport runway upgrade and asphalt overlay on the west coast of Africa taxed these realities. Dynamic creep modulus was specified as main criteria for rut resistance. The dynamic creep modulus gave variable results partly due to the proven inherent unreliability aspects of the test procedure. Therefore the asphalt had to be evaluated against fit-for-purpose criteria. Actual asphalt thicknesses were variable and in some instances significantly thicker than the specified 50mm. The thicker asphalt, in particular, raised questions about perceived rut propensity. The air traffic movements at this remote airport are low and in future will also be low, which potentially lowers the potential impact of excessive rut development. A number of additional tests including the Light Weight Falling Weight Deflectometer (LFWD) and Falling Weight Deflectometer (FWD) were done to determine and confirm the overall structural pavement strength. . Asphalt rut propensity was evaluated with Model Mobile Load Simulator (MMLS) tests. MMLS tests were done on site and additional MMLS tests were done in South Africa on cores from site. Constant Height Repeated Simple Shear Tests (CH-RSSTs) were also done.. The latter test allowed comparison of data obtained from Heavy Vehicle Simulator (HVS) tests and associated CH-RSST data base developed in California, USA. All these tests to determine rut potential have different quirks and positives. A comparative triangulation approach had to be followed to arrive at a weighted best evaluation of this asphalt surfacing. The net result was positive**.

Key words: Rut resistance Repeated Simple Shear Test at Constant Height, Model Mobile Load Simulator, Heavy Vehicle Simulator , (Light) Falling Weight Deflectometer, Triaxial testing

I. INTRODUCTION

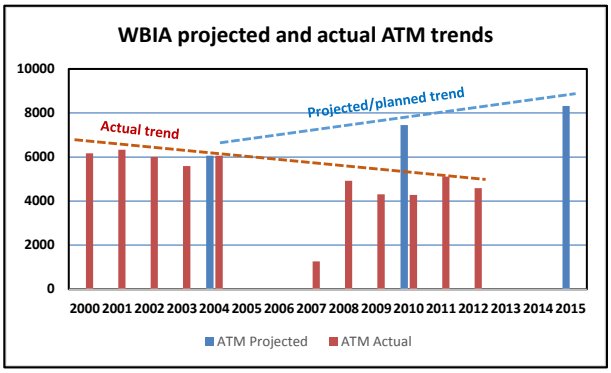
Walvis Bay is approximately in the middle of the long and arid coast of Namibia, on the west coast of Africa. The upgrade of Walvis Bay International Airport (WBIA) was planned in the late 1990s after the handover of Walvis Bay from the South African government to the then recently independent Republic of Namibia. Walvis Bay was identified as a gateway with the existing harbour facilities and airport in need of upgrade. The original plan was to upgrade the former Rooikop military airport runway by lengthening and widening it. The later planning stages coincided with the emergence of the new Airbus 380 size super wide body aircraft. It was thus decided to upgrade this new longer runway to an ICOA 4F standard [14 & 19].

Regional airports in remote regions of Africa often suffer from a number of planning, design and construction peculiarities. A first and common problem is a tendency of over estimation of air traffic movement (ATM) figures. Traffic forecasts with airport upgrades or development generally has a high probability to be wrong to start with due to various extraneous factors. It is often found that traffic figures are overestimated or projected at an inflated growth rate during the planning stage.

Socio-political motivations for airport upgrades are often linked with misplaced national pride. The expectancy level therefore often drives a positive over estimation of ATMs. The decade trend of long term down turn in world air travel figures have thus also showed a number of projects world-wide up to be over optimistic in their planning. In this regard Walvis Bay International Airport (WBIA) experienced a similar dichotomy in planning scenarios versus actual realised future traffic figures.

WBIA was designed in 2006 [14 & 19] to be upgraded from an ICAO 3C to a 4F runway. There is clear wisdom gained in hindsight in comparing original planned growth rates versus actual ATM growth rates experienced. In Fig. 1 the actual versus planned air traffic movement (ATM) figures trends are shown. The gap in the published ATM figures of 2005 and 2006 is when actual construction commenced. This negative impact in ATMs due to construction activities clearly lingered until at least 2007 due to prolonged contractual difficulties and eventual late completion date. The actual

downward trend in ATMs since 2007 is in sharp contrast with the upward trend planned over the medium term from 2004 to 2015. The factors listed above are the main contributors to this diverting planning scenario trend versus the actual observed negative growth ATM trend.



a.

Fig. 1. Planned and actual air traffic movements on WBIA

The new upgraded WBIA runway was to incorporate the old Rooikop military air force base by widening it and adding the required length at both ends of the old runway [22]. The stated socio-political optimism can also be observed in hindsight in regional comparative ATM figures when benchmarked with other regional Southern African airports. ATMs from airports under the jurisdiction of the Airports Company of South Africa (ACSA) [1] are compared with that of airports under the jurisdiction of Namibia Airports Company (NAC) [28] and are shown in Figure 2.

In general the ACSA airports are much busier than the NAC airports. On a comparative basis the ATMs from NAC airports often do not even show up on the same scale in Fig. 2 presented in a combined view. In the inset zoomed in graph it illustrates that even the busiest airport in Namibia, Hosea Kutako International Airport (HKIA) has ATM figures somewhere between George Airport (GA) or Bram Fischer International Airport (BFIA) (previously Bloemfontein Airport) in South Africa. The latter two airports are only the 6th and 7th busiest airports in South Africa. This inset graph also serves to show that WBIA has lower ATMs than the ACSA managed Upington International Airport (UIA). UIA ranks the least busy in the ACSA managed portfolio of airports. Therefore the conclusion can be made that significantly over capacity was provided at WBIA on a regional comparative context.

The second problem remote airports like WBIA suffer from is the lack of experienced airport design expertise available in the region. It is often found that the more dominant market leader in flexible pavements design and construction is in the roads field. Designs therefore tend to be transferred more or less blindly from this roads field to that of airfields when such local expertise is utilized. The significant differences between roads and airfield pavements design requirements are often glossed over and therefore largely treated as if it is merely a wider road pavement with a finite length and just a higher wheel load. Airfield pavements have far fewer load repetitions over their design lives than do highway pavements. Cooley et all [5] illustrated the busiest airfields in the USA can experience up to 20 times less traffic loading than a busy highway in the keel area of the runway (middle 22m on average) where 95% of aircraft travel. In addition wheel track wander of aircraft is much wider than that on highways, which is a function of the much wider pavements on airfields. There are literally large pavement areas on an airfield that may not have a single load applied during the pavement's entire life. These areas would typically

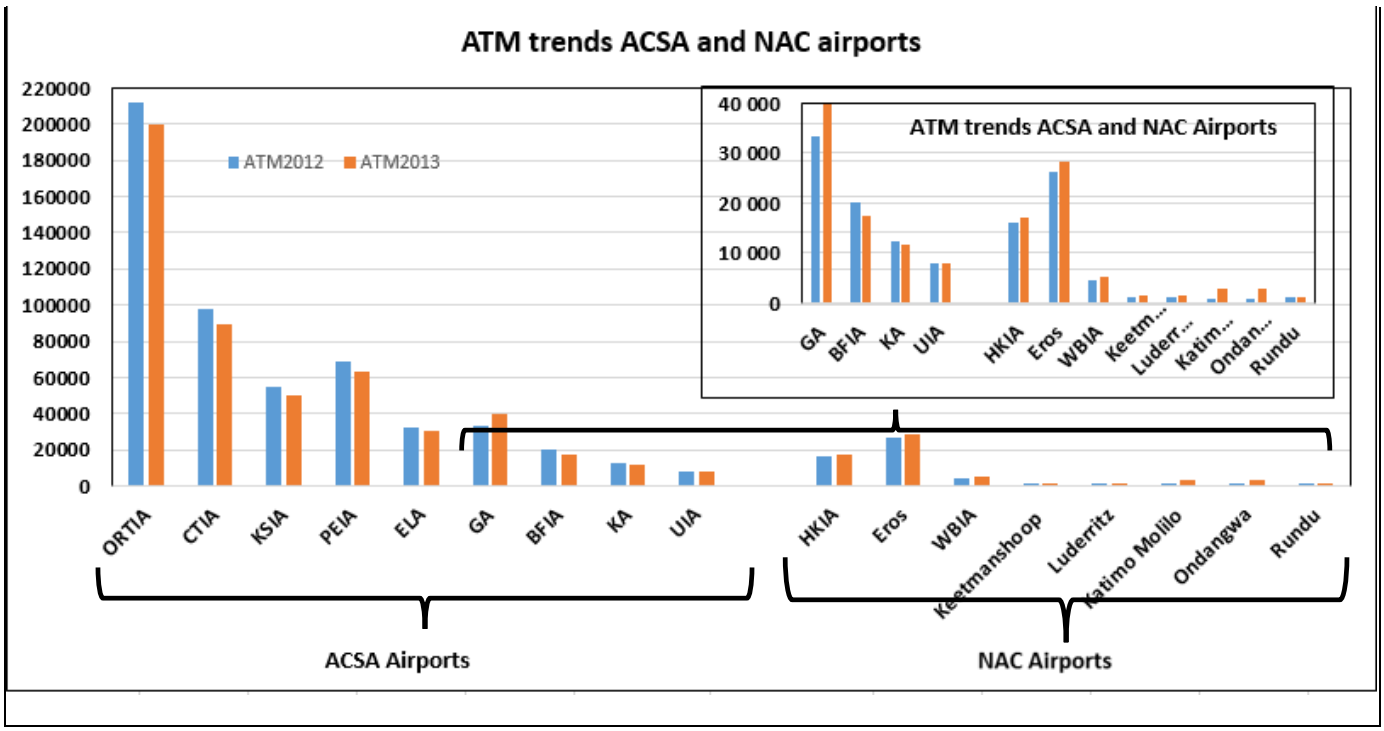


Fig. 2. Planned and actual air traffic movements on WBIA

be those of the overrun area, shoulder areas and even high strength pavement immediately adjacent to the shoulder interface and the keel area. Airport asphalt materials design in such remote regions with low ATMs therefore tend to require to be designed with an environmental durability focus and not like roads with a traffic fatigue and rut focus [20].

The third and more significant problem is a direct result of the remoteness of regions like Walvis Bay. It relates to the availability of quality purpose designed premixed asphalt. The remoteness aspect is further exacerbated by the size of the construction industry in Namibia. As previously mentioned the roads industry is the dominant market segment for pavement construction. Limited use is made of asphalt premix on roads in Namibia. Most of the rural roads in Namibia have surface seals and Cape seals which do not require an asphalt premix plant. Asphalt premix is therefore limited to a small niche market in urban streets in the capital city, Windhoek.

The current single asphalt plant in Windhoek has limited capacity and asphalt quality is not what is required for actual high traffic volume roads and therefore even less for airports. In the case of the WBIA upgrade a mobile asphalt plant had to be established on site. When Hosea Kotaku International Airport (HKIA) recently (2011/12) was rehabilitated and overlaid with a new asphalt surfacing the asphalt volumes dictated that a quality asphalt plant be established on site. In the case of WBIA the project was a design and build project. Various contractual problems led to stop start construction activity which prolonged the project. A variety of sub-contractors were used in the process with variable levels of expertise.

The problems and circumstances outlined above culminated in the construction of an asphalt surface layer on the WBIA upgrade project [14, 19]. The constructed asphalt layer, suited for a road design and largely meeting original roads originated specifications, would not fully meet modern day asphalt requirements for a major international airport. The asphalt layer had to be evaluated whether it is fit for purpose for such a low traffic volume regional airport. The scenario of much lower air traffic movements than planned for had to be taken into account as well as the use of older roads specific design and construction criteria. The potential for particularly rutting had to be determined. In this paper it is therefore described how the rut potential of this hot mix asphalt (HMA) was evaluated to determine whether it will meet rut criteria for the design life and real traffic over the design life of 10 years.

II. Pavement Structure and Strength Evaluation

A. Background

The airport is only 10 km from the coast and within the coastal sea-mist/fog belt [10, 14 & 19]. The runway was lengthened from 2134m to 3440m and the width was increased from 30m to 60m. A 15m wide non-keel widening was added on either side the old Rooikop runway. The pavement structure of the old Rooikop runway consisted of a 85mm accumulated overlays asphalt layer, 150mm Waterbound Macadam base (WMB), 300mm granular subbase on top of the in situ subgrade. A new apron with a connecting taxiway was also constructed. This upgrade contract was let as a turn-key, design and construct project funded by the Kingdom of Spain. The main contractor was a Spanish Consortium, who subcontracted the civil works to Namibian companies. The Government development agency was the Namibian Ministry of Works, Transport and Communication (NMWTC) and the operator is the Namibia Airports Company Ltd (NAC).

B. Pavement structure evolution

A number of pavement structural changes were required due to various practical issues that evolved during the construction phase. A short overview of the changes to the pavement structure is illustrated in Fig. 3. The changes to the pavement structure are highlighted and identified with colour changes as the layer material type or thickness changed with each stage of the contract development. This pavement structure was originally designed as far back as 1997 and was the basis of the 2005 design build contract awarded. In the following text all structural layer material classifications refer to the relevant technical Recommendation for Highways (TRH14) [6]. It shows the original pavement structure designed with a 70mm asphalt surfacing, a crushed stone base, with a typical upper and lower subbase layers of C2 and C4 quality on in situ subgrade.

In 2005, at the start of this contract, the pavement structure was changed due to the limited availability of good subbase material. The required high quality upper and lower subbase layers with cement and lime stabilisation were changed to a single subbase layer of a lower quality with the local pedocrete or gypcrete natural gravel stabilised with lime. A selected subgrade of 300mm was added and deep compaction of the selected and subgrade layer were done to improve the foundation layers bearing capacity in depth.

The remoteness of the region and logistic difficulties in getting lime as well as variable indirect tensile strength (ITS) results led to experimentation with mechanical modification of the pedocrete gravel with selected natural dune sand (wind blown). This mechanical modification alternative was accepted in 2007. This subbase layer, as well as the selected layers were extensively tested using a modified triaxial testing procedure and met all density and triaxial determined effective elastic modulus requirements [29].

However, the high salinity of the pedocrete (also called gypcrete) material in particular led to the reoccurrence of very rapid crystallization during the G1 base preparation. Experimentation with emulsion treatment did not provide a satisfactory solution. The normal procedure to cover this layer as soon as possible also did not have the desired effect of curbing the salt crystallisation. A design change to use a Dry-Bound Macadam (DBM) as base layer was thus proposed as alternative base construction. The DBM was paver laid. This DBM was placed on a bitumen rubber single seal applied on top of the subbase (where the pedocrete was) to act as vapour barrier and helped to successfully prevent the salt crystallization from re-occurring [14 &19].

The last layer to undergo changes late in 2007 was the asphalt surfacing. As can be seen in Fig. 3 this surface layer was originally designed as a 70mm continuously graded hot

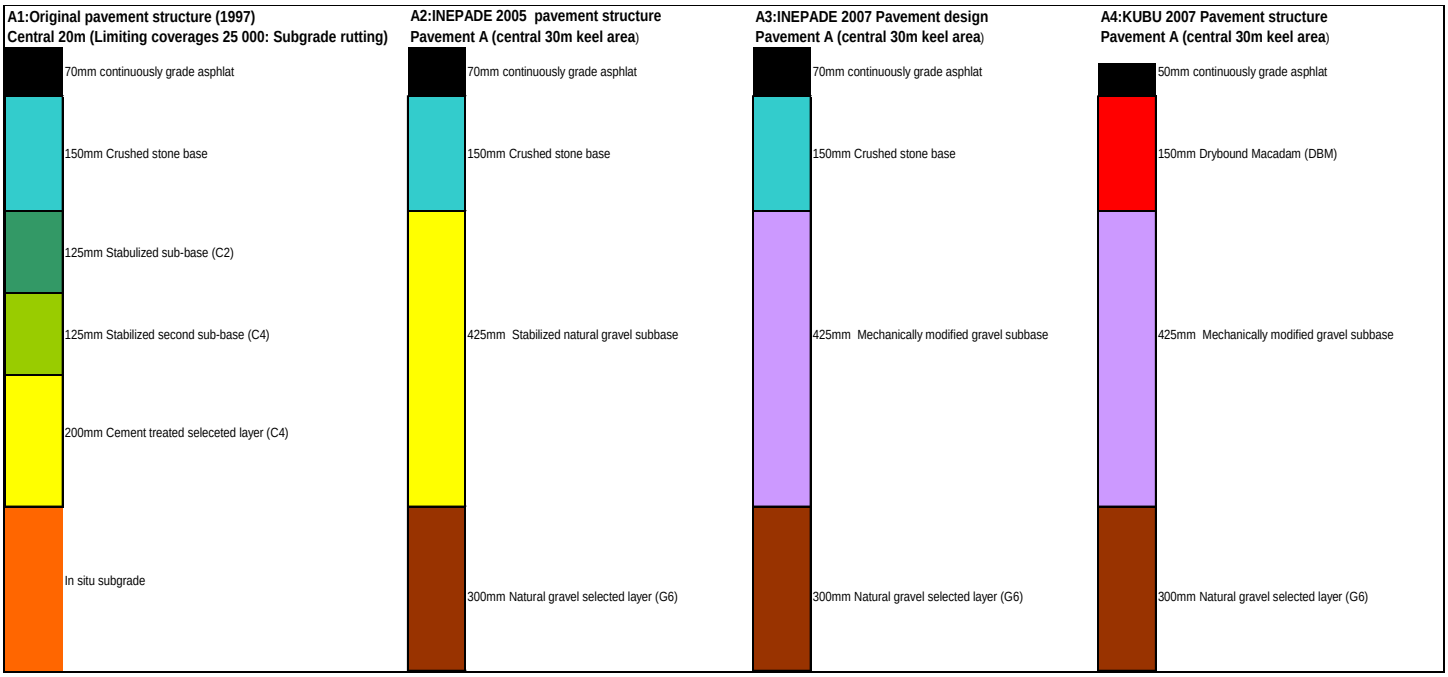


Fig. 3. Planned and actual air traffic movements on WBIA

mix asphalt (HMA) layer. However, cost and concern about the peaking of tensile strains at 70mm asphalt thickness were the motivation for the change to 50mm thickness [2, 12, 14]. This late thickness change to the asphalt layer could not accommodate the identified need for a change of the flat 0.6% cross fall closer to the ICAO optimum 1.5% cross fall. The skid resistance specification was originally the typical reigning roads specification and did not accommodate ICAO required skid resistance requirements [10]. This required change was resisted due to cost concerns linked to the original design build contract specifications.

The design of the asphalt layer was linked to the original 1997 design with specifications based on typical road sourced specifications. These specifications also preceded the more modern South African HMA specifications [33]. The 50mm dense continuously graded asphalt layer was designed with maximum stone size 13.2mm with a straight (no modification) 60/70 pen bitumen with a design bitumen content of 4.8%. This lowish binder content is a typical good economical binder content for a road, but is not a proper fit for an airfield where environmental durability is the more destructive influence[20].

The old Rooikop runway was also to receive a 50mm overlay to ensure it is structurally strengthened and linking in with the extensions. Level differences at the old runway edges and the new extensions required that an asphalt scratch/levelling coat be applied. This levelling coarse was of average depth 20mm, design binder content 5.3% with 60/70 unmodified bitumen, dense graded medium, with maximum stone size 13.2mm. Some areas had an additional correction layer on top of the base course, and the total asphalt thickness was afterwards found be up to 140mm thick.

C. *Structural evaluation*

The introduction of the DBM base layer raised questions about structural strength and material quality control. After the DBM received a slurry surface and partial scratch coat various structural evaluations were done. As mentioned the modified triaxial testing [30] was used on the evaluation of the selected and subbase layers. This same triaxial test could not be used on the larger aggregate of the DBM (maximum size 63mm to 75mm). Initially plate bearing tests were experimented with, but soon found to suffer from the same aggregate size effect as the triaxial testing as well as serious doubts about repeatability and interpretation for such a larger sized aggregate base layer [14 & 19].

The remoteness of this region meant that initial analysis had to be done with the more readily available light weight falling deflectometer (LFWD) [17 & 18] and subsequently with the falling weight deflectometer (FWD) [15]. Whilst the LFWD has only limited depth of influence due to the lighter dropped weight and smaller loading area, it did enable the calculation of surface modulus values reflecting adequate structural capacity for the constructed DBM layer and indicating a uniformly strong layer. However, more detailed analyses followed with the FWD done before and after the asphalt surface construction. In short the findings from this FWD high survey sample density on the completed runway are summarised here in short.

The FWD benchmark analysis [15] found that the new additions (extensions) with the DBM base layer were structurally sound in depth and capable of carrying more than the designed for aircraft loading. The old Rooikop runway section showed 10% to 15% of the subbase and selected layers may pose a warning structural condition, but after the 50mm asphalt overlay it was reduced to approximately 1% of area in

the warning condition making this old runway keel section also structurally strong and sound.

The FWD survey was also used to do detailed effective elastic moduli back calculation by means of three different back calculation software packages. The triangulated results found that the new, as well as old Rooikop runway sections, have very strong subgrade strength as reflected in effective elastic moduli ranging from 180MPa to 200MPa. The effective elastic moduli of the selected layers of both the old and new runway sections also ranged from 200MPa to 300MPa. The subbase of the new sections were not only deeper or thicker, but also of a better material quality and with higher effective elastic moduli values (up to 400MPa average) than that of the old Rooikop keel area (approximately 300MPa).

These high effective elastic moduli values for layers which make use of the in situ material, Gypcrete or Pedocrete, either as is or in a mechanical stabilization product, even showed evidence of self cementing which tend to provide for very deep and strong pavement structures, ideal for airport loading conditions. The effective elastic moduli values of the DBM effective elastic moduli values were in the range of at least 500MPa to 600MPa. This base also covered the thinner pavement structure of the shoulder area.

All these effective elastic moduli values of all layers in the various pavement structures are more than were initially assumed in the design and mechanistic modelling of the pavement structure. The effective elastic moduli thus back-calculated with the FWD survey was used to check the structural strength via the same linear elastic layered software models and found the whole runway to be more than adequate for the overestimated projected traffic figures over the foreseen 10 year horizon. The Pavement Classification Number (PCN) thus calculated was compared with the Aircraft Classification Number (ACN) of the design aircraft and also proved that the whole runway and new taxiway have more than adequate structural capacity.

It was therefore concluded that the pavement structure as constructed was in fact structurally stronger than what was initially designed for and should be more than capable of carrying the aircraft loading over the design period. This was also an important finding which ensures that rut propensity contribution therefore was limited in its potential to that of the asphalt surfacing and scratch coat layers [31] .

III. Rut Propensity Investigation

A. *The concern about rut posibility*

After the structural evaluation described above concern was expressed about the lack of quantitative evaluation of the rut propensity of the asphalt layers. The maximum aggregate size of 13.2mm coupled with the need for level correction layers and totalling up to 140mm asphalt layers were cause for concern that this layer may fail in rutting very rapidly in this arid region with its high temperatures. In Table 1 the summary of the basic binder contents and air voids values are shown.

TABLE I. Summary of selected asphalt wearing course results

	Binder content (%)	***Voids (%)***	***Average core thickness (mm)***
Project maximum	5.6	6.6	140
Project average	4.8	4.5	59
Project minimum	4.2	2.7	30

Using the conceptual relationship of Verhaeghe et al [36] to predict initial consolidation due to volumetric values it could be established that an asphalt layer in the range of 120mm to 150mm total thickness for air void content of say 5% or even 6.6% may lead respectively to as much as 1.5mm to maximum 3mm consolidation type rut. This however would only realise if the traffic figures were high and coinciding with high temperatures. Therefore the rut propensity had to be investigated in more detail.

The basic Marshal design method was prescribed and rut potential was to be checked with Dynamic Creep Modulus. However, no specific acceptance criteria have been listed in respect of this parameter even though contractually this was the only alternative test directly related to rut propensity prediction. Creep Modulus results were very variable and therefore this test and its status were questioned.

The SABITA Interim Design Guide [33] quoted here on this matter largely guided the further detailed investigation. "*In recent years, research work has raised some doubts concerning the ability of the dynamic creep test to properly and consistently evaluate the rutting potential of different mix types. One of the main criticisms of the test concerns the absence of a confining pressure as well as the apparent insensitivity of the test results to low void contents. The test is generally regarded as being inappropriate for evaluating mixes that rely on stone-to-stone contact to develop rutting resistance For these reasons, the use of the dynamic creep modulus as an acceptance criterion is not recommended for mixes other than densely graded sand-skeleton mixes manufactured with unmodified binders. Mixes designed for situations which require superior rutting resistance should rather be evaluated using a wheel-tracking test. However, the dynamic creep test can be used in conjunction with volumetric test data and other performance tests to serve as a general check on the overall rutting potential of a mix.* "

B. *Differences in roads and airports loading contribution to rut potential*

It had been stated in the introduction that the loading frequency on an airport and a road differs by a factor of at least 10. The quantum of loading also differs between the road and airport situation. In most countries, road traffic loading is converted to the legal axle load or standard axle load such as the equivalent 80kN axle load (E80) as used in South Africa as design input for roads. This can be converted to 40kN per dual wheel loading and typically a 0.7 MPa contact pressure per wheel. A runway design using a typical design aircraft such as the Boeing 747 has a wheel load of approximately 225kN per wheel on their main gear and a tyre pressure of 1.39MPa per wheel. The tyre pressure of such a typical design aircraft is roughly double that of a truck tyre pressure. This has implications particularly for rut development as the

resultant vertical stress distribution in the top 50mm to 100mm in the asphalt base and surface layers is very different if super-imposed with typical temperature variation and related asphalt stiffness variation as illustrated conceptually in Fig. 4. Apart from a Boeing 747 wheel load, a typical truck wheel load and the wheel load of a Model Mobile Load Simulator (MMLS) is shown as it will be referred to later when the translation of roads related rut specifications to that of fit for purpose airport rut calculations are discussed.

Finite element modelling of the effect of high aircraft tyre pressure on flexible pavements shows that the high aircraft tyre pressures and non-uniform contact stresses at the tyre-pavement interface cause high shear strains/stresses in the asphaltic mix layer which are responsible for rutting and near-surface cracking. This requires high stability and shear strength asphalt mixtures [37].

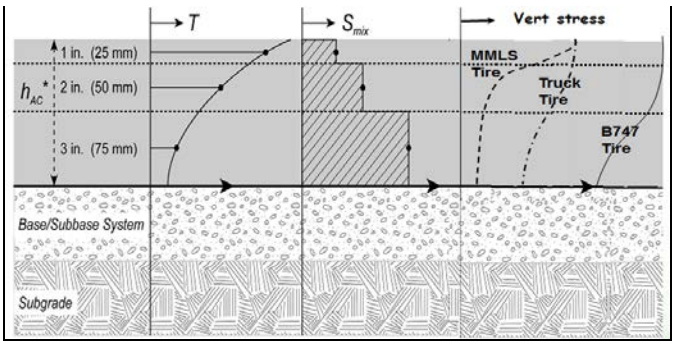


Fig 4. Temperature stiffness and vertical stress distribution in an HMA surface layer. (Adapted from Monismith et al [24]

C. *Creep or rut fundamentals*

The fundamental behaviour of creep, or better known as rut, in pavement engineering circles is illustrated in Fig. 5, which is applicable to hot-mix asphalt [8 & 9]. A clear distinction is made between the phase I of initial densification type primary creep or permanent deformation (rut) behaviour and the phase II of steady state secondary creep behaviour. The latter creep phase is linked primarily with shear related deformation which is the most common type of rut development over the life of a Hot Mix Asphalt (HMA) layer exposed to repeated loading. The last phase is tertiary creep when total failure occurs and is not the focus of the discussion as failure in that state is obvious and is beyond the scope of design considerations here.

Rutting in HMA layers exposed to repeated traffic loading occurs predominantly at elevated temperatures[23]. This was illustrated conceptually in Fig. 4 where stiffness is reduced with elevated temperatures in HMA. Shape distortion (shear) in the steady state phase (II) is the main contributor to permanent deformation in the asphalt-bound layer, compared to volume change (densification) [8, 9, 26, 36] which occur primarily during phase I. It was observed for highway loadings and subsequently also for airfield pavements, that rutting is mostly limited to the top 75mm to 100mm because of the high shear stress under the edge of a loaded tyre and just below the surface coupled with the higher pavement temperatures occurring at or near the surface of the HMA layer.

The Marshall asphalt design method has its origin in airfield asphalt material design. It is still the dominant design method for airfield hot-mix asphalt. Cooley et al [4 & 5] found that the Superpave design methodology has had limited application on airfield asphalt design , and research was still on-going to do this. This design methodology is clearly in need of an upgrade as the latest generation wide-body aircraft, such as the Airbus A340 and Boeing 777, are showing up the limitations of using Marshall design and therefore requires additional consideration of permanent deformation resistance [10]. HMA for highway pavements in the USA is most commonly designed in accordance with the Superpave mix design method as outlined in AASHTO M323, “Standard Specification for Superpave Volumetric Mix Design.” Whilst with road asphalt design there is invariably one optimum design, the same asphalt mix is not necessarily good for the varying demands on an airfield even if Superpave is being used.

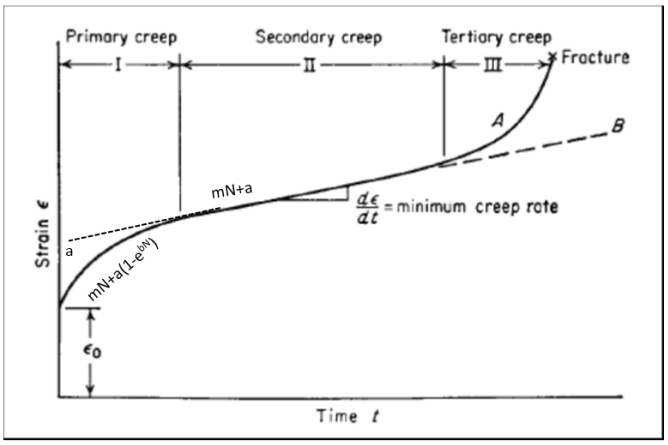


Fig 5. Generic creep behaviour of materials [3]

Marshall specifications utilize Marshall stability and flow as a proof test during mix design. However, the criteria used are not a true reflection of rut resistance performance over the life of the asphalt mix for reasons given above. The Superpave design procedure has moved forward in terms of trying to include proof tests [4], but currently there is still uncertainty in HMA design methodologies regarding which universal test should be used or how it is actually calibrated with real life rut occurrence [3 & 8].

When selecting optimum asphalt mixes, all these design methods mentioned are similar in that volumetrics are used as the basic criteria. Air voids, voids in mineral aggregate (VMA) and voids filled with asphalt (VFA) are all directly or indirectly specified. There are slight differences in the specified volumetric requirements, the most important is the use of a range in design air voids in the Marshall method [4 & 5].

D. Measuring rut

D.1. Introduction

As indicated before dynamic creep or "modulus" tests and calculations have serious problems of accuracy and repeatability. As suggested in the SA Interim HMA design guide [33] additional tests are needed. Full scale accelerated pavement testers (APT), such as the Heavy Vehicle Simulator (HVS), have been used on airfield pavements in South Africa, but this is a very costly test and not used for normal design purposes. Laboratory rut test devices are more commonly used for this purpose. Currently there is no universally accepted rut testing device which can accurately predict rut progression [3 & 8]. In most cases these devices (e.g. the Hamburg device) are used as benchmark tester to discern between rut propensities on a relative or comparative basis. Most of these testing devices are still attempting to establish criteria for road pavements and there is not such a large data base for airfield pavements yet.

The Model Mobile Load Simulator (MMLS or MMLS 3) is one such scale tester for evaluating the rut propensity performance of asphalt mixes in the laboratory or field, and this has been used with some success on airfield pavements [33,21, 11]. The test bed for trafficking by the MMLS in the laboratory allows nine 150 mm cores to be placed adjacent to each other, each fitted snugly into a restraining mould that provides circumferential support to the test specimens. Standardisation of test protocol is important and therefore it was decided to use the MMLS3 Baton Rouge Protocol [21] and its updated more recent version [31]. MMLS testing can be adjusted to simulate field conditions in the laboratory, accommodating wander, in-situ operating temperatures, applied tyre pressure and also traffic speed. The MMLS has value in comparing relative rut propensity by adjustment of testing protocol, e.g. speed and temperature, to discern between bitumen binders and possibly grading.

A more fundamental testing device which determines permanent strain accumulation in the steady state rut phase is the Repeated Load Test in Shear Test – the Superpave shear tester (SST) which was developed as part of the Strategic Highway Research Program to simulate the shear situation of HMA layers under repeated loading conditions [21, 26, 27,28 & 31]. The Repeated Simple Shear Test at Constant Height (RSST-CH), thus developed can perform a repeated load test in shear. A repeated haver-sine shear stress is applied to the 150mm test specimen and strain measured throughout the test. Lately a triaxial testing protocol is actually proposed as the most representative of confinement needed as well as shear inducement to accurately reflect actual phase II rut or creep development [3], but this test protocol was not available during the time of this investigation.

D.2. Demonstration of rut determination via MMLS testing

Various cores were sampled from WBIA and tested in the laboratory with the MMLS to determine the likely rut propensity linked to the anticipated traffic volume. The test temperature of 50°C was chosen to be representative of the 0-50mm asphalt thickness in the Walvis Bay area. Cores were prepared for the MMLS testing in South Africa. Tests were also done at 45°C on cores taken below the 50mm overlay. The bitumen content was 4.8% on average with a voids content of 4.5%.

The actual MMLS rutting in the testing ranged from 2.18mm to 2.31 mm for the 50mm surface asphalt. These MMLS tests were all done at the area representative surfacing temperature of 50°C, and at 100,000 load applications. These values are above the adjusted Baton Rouge 1.44 mm limit for airfield asphalt [21]. The 1.44mm rut depth was reached at just over 5 000 load applications in the worst case. These MMLS results need to be correctly interpreted in terms of the context of actual aircraft traffic on this specific airfield and other factors as will be shown.

The MMLS test done on 90mm thick cores at 50°C had the same rut propensity trend and final values as for the 50mm and 60mm core depth tested. This tends to confirm that due to the temperature and mix stiffness variance in depth of the asphalt layers, the main contribution to rut development is confined to the top 50mm.

The MMLS test data was used to calculate asphalt rut depths for the runway and taxiway at WBIA. This airfield is trafficked by low volumes of aircraft. The design aircraft was the Boeing 747-400 with 20 000 load repetitions and the design period is 10 years before asphalt overlay. The realities of the design traffic actually attracted and expected within the trends in the world airline market led to an adjustment of a lower traffic scenario of 6 500 load repetitions. The expected runway field rut depths are summarised in Table II for thin and thick layers of asphalt for the realistic scenario of 6 500 load repetitions as well as the initial optimistic 20 000 load repetitions.

ICAO compliant cross fall of 1.5% on runways would allow for 20mm to 25mm rut (and undulations) development as measured under a 3m straight edge which will still allow water drainage and water ponding prevention. However, the rut depth to indicate functional failure due to water ponding at WBIA with its cross-fall of only 0.6% is estimated as limiting rut development to less than 9mm. The calculated field rut depths are shown in Table II using the method discussed by Emery and Mihaljjevic [11]. It clearly shows that the rut estimated is less than 9mm.

The asphalt mix tested with the MMLS can thus be considered as meeting the low design traffic exposure applicable to the WBIA situation for the design period of 10 years even though it is marginally over the adjusted Baton Rouge criteria developed for high aircraft traffic volumes.

The draft Baton Rouge test protocol [20] suggests that age hardening due to known high ultraviolet exposure could be taken into consideration which further reduces potential rutting of the exposed asphalt mix. In general aging is accounted for by reducing the expected rutting as finally estimated from the MMLS rutting performance by as much as 30%.

TABLE II. SUMMARY OF CALCULATED FIELD ASPHALT RUT DEPTHS FOR WBIA RUNWAY (2009)

Airside Section	**Calculated rut depth at design traffic (mm)**			
	6 500 departures		***20 000 departures***	
	Thin asphalt (58mm thick)	*Asphalt + thick scratch coat (116mm thick)*	*Thin asphalt (58mm thick)*	*Asphalt + thick scratch coat (116mm thick)*
Runway	2.9	(2.9+1.7*)= 4.6 mm	3.8	(3.8+2.2*)= 6.0 mm
Taxiway	3.9	(3.9+2.2*)= 6.1 mm	5.0	(5+3*)= 8.0 mm
Functional limit	9.0 mm			

* The calculated Relative Stress Potential was calculated to adjust rutting measured in the MMLS to that which would be caused by the design aircraft at the appropriate depth of pavement

D.3. Demonstration of rut determination via RSST-CH testing

Four asphalt cores from WBIA were tested by the RSST-CH at the CSIR asphalt laboratory in South Africa. Table 3 below shows a summary of the results obtained and associated calculations. The deformation response of the material in the RSST-CH is characterized using the mathematical representation of a creep curve shown in Fig. 5. The slope of the creep curve in the secondary, or steady state creep phase, is a primary indicator of permanent deformation potential [23,28].

Values are provided for the total strain in the primary creep phase (a) and the slope (m), or strain rate, in the steady state creep phase. The permanent strain (expressed as percentage) was obtained from the RSST-CH test result graphs at the 5 000 repetitions.

TABLE III. SUMMARY OF RESULTS AND ASSOCIATED CALCULATIONS

Sample	G (Complex Modulus) [MPa]	m [ε/cycle]	a [mm]	Percent strain at 5 000 load repetitions	Percent strain at 25 000 load repetitions	Deacon* approximation rut calculation
4642-A	7.25E+01	2.75E-06	3.38E-03	1.7	7.2	4.25mm
4642-B	5.17E+01	6.73E-06	9.23E-03	4.3	17.8	10.75mm
4642-C	5.02E+01	9.09E-06	7.32E-03	5.4	23.5	13.5mm
4642-D	5.83E+01	3.22E-06	1.05E-02	2.5	9.1	6.25 mm
			Average	*3.5*	*14.4*	*8.75 mm*

a. * (Deacon et al, 1995)

RSST-CH results of two HMA mixes on international airfields in Doha, New Doha International Airport (NDIA) [23] and in California, San Francisco International Airport (SFIA) [28] have been used as a direct benchmark or comparison. The WBIA cumulative traffic is only approximately 5% of the NDIA and SFIA traffic totals. On a proportional basis this means WBIA should be compared with the derived strain and deformation calculations for 1 250 RSST-CH repetitions only. The test acceptance criteria previously developed of 5% permanent strain at 25 000 repetitions for NDIA and SFIA were determined at 1250, 3000 as well as 5000 repetitions for the WBIA RSST-CH permanent strain measurements. It was found that the asphalt mix at WBIA will be below 5% strain for 1 250 and 3 000 repetitions, but just marginally meet this adjusted test acceptance criteria for the 20 year design period at the 5 000 RSST-CH repetitions. Deacon's approximation [7 & 31] was also used to calculate rut estimates using the 1 250, 3 000 and 5 000 repetitions strain values. Only the 5 000 repetition and the normal benchmark of 25 000 repetition comparative rut values are shown in Table III. Roderiques et al [31] subsequently confirmed the validity of this approach by developing a generic equation which was calibrated with various real traffic and pavement evaluation data. This Deacon [7] approximation shows that the rutting in the WBIA asphalt mix would marginally be below 10mm at the end of the asphalt service period for the 5 000 repetitions criteria, while for the 1 250 and 3 000 repetitions related strain values it will be well below 10mm.

Age hardening was not included in any of the calculations, but literature available clearly indicates that it will have beneficial resistance to the rut propensity of the WBIA asphalt mix. Walvis Bay is an area with known high ultra-violet exposure and therefore age hardening. The fact that this analysis of the CH-RSST results was done over a comparative 20 year period will definitely count for a further reduction of the rut estimations.

IV. CONCLUSION

The remote region on the west coast of Namibia and Africa where the upgraded Walvis Bay International Airport (WBIA) had three under-laying problems which directly contributed to perceived rut propensity problems with the asphalt surfacing provided. These problems are:

The over estimation of air traffic growth which proved to be a blessing in disguise as the lower air traffic figures translates into less impact on a the potential rut propensity during the settling in or post construction compaction as well as the steady state period where rut is directly related to trafficking during hot periods.

The second problem of roads specific design approach and specifications led to outdated rut specification and testing requirements not specific enough for an airport environment. Road asphalt mixes tend to be lower than that of airport pavements which tend to give better resistance to deformation and rutting. Thus the 4.8% binder content used on WBIA is typical for a roads project and even though this may compromise durability on an airfield it helped to improve resistance to rut propensity.

The dynamic creep “modulus” specification was inherited from an outdated roads focussed design and led to uncertainties regarding actual rut propensity determination. Even though the rest of the new and old pavement structures were proved to be structurally adequate to the extent of being over designed, the asphalt layer designed and subsequently constructed had to be evaluated for perceived problems with rut propensity. Specialised rut testing with the MMLS and

RSST-CH tests were done and evaluated with the then current criteria. Benchmarking these results with similar recent investigations on other larger international airports put the rut propensity into perspective.

The lower traffic figures experienced on WBIA versus initial overestimation during the planning phase also helped to rate the rut propensity as relatively low for traffic actually experienced. This was true even though a more conservative or lower rut criteria of 9mm under a 3m straight edge had to be used due to the geometry of a relatively flat cross fall to prevent "bird baths" due to potential rut development..

The third problem of available construction technology and specific to the asphalt plant proved to have a minor impact on the rut propensity aspect investigated. The asphalt produced proved to be reasonably within the dated roads specific specifications. Actual laying and compaction are acknowledged to have identified other issues with asphalt thickness, longitudinal joint problems and air voids which are within the roads directed specifications used, but which would not be accepted under proper airports specific and newer specifications. These aspects are issues which have a strong impact on durability and the impact of environmental factors and were not the focus of the perceived problem with rut propensity initially envisaged.

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