Research Article

Effect of Unstable Mix under Severe Traffic Loading on Performance of Asphalt Pavements in Tropical Climate

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This paper is aimed at assessing the in-service performance of asphalt pavements in tropical climate under severe conditions. The main defect observed on the asphalt pavement was rutting of the asphaltic surfacing, with top-down cracking being experienced on a few sections and not widespread but rather intermittent. Field and laboratory investigations were conducted as well as a review of design and construction records. The defects observed were confined to the wearing course layer of the surfacing with the other underlying layers performing well. Rutting was a result of heavily loaded trucks that moved at very slow speeds due to steep gradients, hence resulting in severely loaded sections. High temperatures due to the warm tropical environment exacerbated the situation and caused the asphalt to flow, hence resulting in rutting and deformation. Also, low air voids in the asphalt mix which were below the recommended design air voids specification aggravated the situation as well as the air voids after refusal density compaction being below the specified critical minimum of 3% after secondary compaction. Top-down cracking was due to binder age hardening and embrittlement resulting from overheating of bitumen during the construction process coupled with heavy truck axles and high tyre pressures. Defects observed, therefore, resulted from an unstable asphalt mix that was not suitable for severe loading conditions; hence, the asphalt concrete laid was out of specification. The Modified Marshall Mix Design method should be used for severe sites where slow speed or heavier traffic is expected.

1. Background

Majority of the paved road network worldwide is comprised of bituminous pavements because they largely provide a safe, long-lasting, and comfortable surface capable of adequately performing over its design life with minimal maintenance. However, the prevalence of premature pavement failures on many road networks especially in developing countries has become a major concern to the road authorities who are responsible for developing and maintaining paved roads. Since roads are the most valuable public asset in most countries and are critical to the movement of goods as well as helping people get around, premature failures create funding challenges for maintenance interventions on deteriorating roads. Resources that would otherwise be invested in other sectors or used for expansion of the paved road networks are instead diverted to rectify the defects. This negatively

impacts on the economic performance of developing countries and results in direct costs to taxpayers through rising costs of deferred work or through a stopgap approach that does not represent good value for money in the long term. One of such roads on the paved national road network in Uganda that has experienced early pavement deterioration is Kampala–Masaka road. The road begins at the outskirts of Kampala City and is a trunk road carrying heavy traffic that is transiting through Uganda from the coastal seaport of Mombasa, heading to landlocked countries of Rwanda, Burundi, and the Democratic Republic of Congo. The road is mostly a single carriageway with relatively narrow lanes leading to channelization of traffic as well as numerous climbing lanes which impacts on the loading characteristics of the pavement. The road is located in the tropical climate with monthly air temperatures ranging between 14.5 and 38°C and annual precipitation of 952 mm.

The construction of the trunk road which started in 2009 was completed in 2015. It is surfaced with 130 mm asphalt concrete comprising of 80 mm and 50 mm binder and wearing courses, respectively. The pavement layers include a crushed stone base of 200 mm and a crushed stone subbase of 250–275 mm constructed on an existing lateritic base (now subgrade) or widened in some sections with fresh borrow material used as fill material. The longitudinal profile consists of numerous crest and sag curves resulting from an undulating terrain. However, the road that was designed to last 20 years has experienced significant defects in less than 5 years after completion including rutting, deformation of the road surface, and cracking.

Since the performance of asphalt concrete is dependent on many factors such as material characteristics, traffic and axle loading, construction quality control and assurance, and environmental and climatic conditions, an investigation was conducted to identify the clear cause of the observed deteriorations on the road. Comparison of the different key pavement performance parameters of the sections on the road that had not failed (control) with those that failed was undertaken through field and laboratory investigations as well as a review of the design and construction records. The study will come up with appropriate measures that can be undertaken to correct the current defects and provide valuable experience for application in future road development and maintenance projects.

2. Literature Review

2.1. Introduction. Asphalt pavements are fundamentally designed to last, but often they do not perform as expected because of unforeseen factors during their lifecycle. They therefore sometimes end up requiring rehabilitation after a relatively short period of time. This scenario is normally attributed to surface distress in form of cracking such as fatigue, longitudinal and transverse cracks, and/or surface deformations such as rutting, corrugation, and shoving. This distress in asphalt pavements is either environmental or structural in nature. Environmental distress is usually associated with factors affecting the pavement from outside such as air, water, and snow and is observed from top-down while structural distress is physical failures found on the pavement and the subbase as a result of overloading, wetting of the subgrade, and so forth. This distress is sometimes experienced much earlier compared to the design life of the pavements, with some bituminous pavements observed to experience top-down cracking just 4 years after construction [1]. In most cases, this distress is primarily experienced within the asphalt concrete surfacing of the pavement structure and maybe a result of many factors such as material characteristics, traffic and axle loading, construction quality control and assurance, and environmental and climatic conditions [2]. It is therefore imperative that designers find a balance of various parameters without compromising the performance of the mixes [3]. For instance, a stiff asphalt mix could have good bearing capacity but experience fatigue failure and crack during the winter due to low temperatures, while a mix without sufficient stiffness may be susceptible to plastic deformation or rutting during the summer due to high temperatures [4].

2.2. Rutting and Cracking. Rutting or permanent deformation is one of the common defects experienced on road pavements and is mainly manifested as surface depressions along the wheel path (ruts) resulting from the consolidation of the underlying road materials due to traffic loading. This can be a result of shearing of the pavement material which can be experienced in both the asphalt and foundation layers (base, subbase, and subgrade). Surface rutting is normally confined to the asphalt while structural rutting is a result of materials within the foundation undergoing shear. Surface rutting is often associated with inadequate compaction of the hot mix asphalt (HMA) leading to densification due to secondary compaction during trafficking of the pavement. Furthermore, it may be related to asphalt mix design problems where parameters such as binder content and grade, gradation, and aggregate properties are not well selected [5]. Structural rutting is usually a function of pavement settlement due to subgrade failure which could be caused by either excessive loading or a pavement that has been structurally underdesigned [6]. Asphalt concrete pavements have been observed to be prone to rutting at high pavement in-service temperatures. This is because, at such temperatures, the asphalt cannot resist deformation resulting from the combined effect of repeated loading and increase in irrecoverable deformation due to a reduction in the viscosity of the binder hence rutting [7, 8].

Asphalt pavements experience different modes of cracking including fatigue cracks, top-down cracks, reflection cracks, faulting, and thermal cracks. Fatigue cracking is normally due to failure of the surface due to traffic loading but can also be considerably influenced by environmental and climatic conditions. Top-down cracking is mainly resulting from stress concentration at the pavement surface while reflective cracking is due to stress concentration near the crack tip of the existing layers of rehabilitated pavement [9, 10].

2.3. Traffic and Loading. Knowledge of the actual loads applied to the pavement especially overloads is important in predicting the pavement life and defining the Equivalent Standard Axles (ESA) to be used in pavement design. This is because pavement failures in form of rutting and fatigue cracking are associated with heavy traffic loading. Permanent deformation due to repeated traffic and axle loading often happens within the first few years after road opening [11]. Traffic is considered in terms of the intensity and frequency of the loads, while axle loading focuses on the axles and tyre configuration of vehicles as well as the tyre pressures. The number of heavily loaded vehicles and their axle load configurations greatly contribute to pavement deterioration, hence compelling road authorities worldwide to institute and enforce load limitations [12, 13]. The impact of wheel loading on pavement performance is critical and must be carefully incorporated in pavement and bituminous mix designs. It is well known that asphalt concrete is a

viscoplastoelastic material and therefore its elastic modulus is very dependent on temperature and loading time [4]. Also, tyre pressures are very important since the contact area and pressure of tyres are greatly influenced by the magnitude and frequency of loading from the tyre to the pavement surface interface [14]. Furthermore, tyre contact area is dependent on the tyre inflation pressure and structure that, in turn, impact on the critical tensile strain at the bottom of the asphalt layer, surface deflection, and compressive strength at the interface between the base and the asphalt surfacing [15, 16].

2.4. Effect of Air Voids.

The amount of air voids in a mix are also known to greatly impact on rutting resistance of asphalt mixes, hence the durability of an asphalt pavement. Air voids are little spaces or pockets of air left between the coated aggregates after compaction of a mix. Asphalt mixes undergo secondary compaction due to traffic loading during the service life of a pavement. Therefore, an optimum amount of air voids is necessary for the durability of pavements and must be designed for dense-graded mixes to provide spaces to accommodate for the flow of the binder during secondary compaction [17]. For most mixes used for surfacing, a range of 3–5% of air voids in laboratory samples is considered adequate. Too high air voids lead to a permeable mix which is susceptible to the damaging effect of air and water. Conversely, too low air voids in a mix are known to lead to bleeding and flushing hence resulting in stability problems. Furthermore, low air voids are often associated with an excessively high amount of bitumen content and/or fines during asphalt production, with bitumen content generally having a greater impact on rutting compared to fines [18–20]. Low air voids would ideally not be problematic in relation to rutting provided adequate mix stiffness is achieved. However, stiff mixes usually have workability related issues since they are not easy to place and compact to the desired density. This is because most stiff mixes utilise stiff binders that are sometimes polymer-modified leading to an increase in binder viscosity, hence decreasing the workability substantially at a given temperature. In order to achieve the desired density within a compacted pavement, acceptable workability is paramount. Therefore, a combination of low air voids and low stiffness would, therefore, most likely pose significant pavement performance-related problems [21].

3. Materials and Methods

3.1. Selection of Study Sections.

Study sections, on which detailed investigations were undertaken, were selected by undertaking a drive-through and walk-through reconnaissance of the roads. Drive-through surveys were carried out at low speeds with frequent stopping to observe and assess the condition of failed and sound sections of road. Typical

defects on the road were recorded and used to select sections that represented all the defects observed on failed sections. In order to obtain a better understanding of the nature and causes of the premature failures observed on the road, control sections experiencing similar conditions of traffic loading and road environment but exhibiting good performance were also selected for comparison purposes with failed sections.

3.2. Field Investigations.

Site activities carried out on the selected sections of the road included roughness using the ROMDAS (Road Measurement Data Acquisition System), rutting using an automated ultrasonic profiler, visual condition surveys using a walked survey, core sampling of the asphaltic surfacing using a core cutter, and trial pitting for the sampling of asphaltic surfacing and pavement layers for laboratory testing. Furthermore, 100 mm diameter cores were extracted mainly for strength tests (i.e., Marshall Stability and Indirect Tensile Strength) and 150 mm diameter cores for observation of crack initiation, crack depth, visual quality assessments of the asphaltic layers, refusal density testing [22], and other material quality tests of the aggregates and the binder. The test pits measured 2 m by 1 m in the transverse and longitudinal direction, respectively, were about 1.2 m deep from the top of the road surface across the outer wheel path which allowed for observation of the rutted layer(s). In situ density testing [23] of pavement layers was also carried in the test pits.

3.3. Laboratory Investigations.

Material samples of asphaltic surfacing and pavement layers collected from the road were tested and analysed. Asphalt cores were tested for Indirect Tensile Strength (ITS) [24] and Marshall Stability and Flow [25]. Bitumen and aggregate recovered from the surfacing were tested to determine their properties and for comparison with specifications. Aggregate strength testing and determination of other quality characteristics such as particle size distribution was carried out for crushed stone or crusher run base and subbase [26, 27]. Subgrade soil material was tested for the plasticity index [28] and California Bearing Ratio (CBR) [29]. Tests were conducted on 3 samples and average values used for analysis. The methods used in the selection of sections, as well as conducting field and laboratory investigations, are as already presented elsewhere [30].

3.4. Other Sources of Data.

Axle load surveys were carried out along the road, analysed, and then converted into Million Equivalent Standard Axles (MESA). Truck tyre pressures were also measured for one load axle per vehicle while conducting the axle load surveys. Trucks measured were mainly Heavy Goods Vehicles (HGV) with over 3500 kg gross vehicle weight. Traffic count as well as the

design and construction data including reports were obtained from the road authority, and the information obtained was compared with the measured data.

4. Results and Discussion

4.1. Pavement Design

4.1.1. Traffic. Traffic assessment was carried out by comparing traffic data obtained (Average Daily Traffic) for both the current counts and those carried out at the design stage for trucks, semitrailers and trailers as shown in Table 1. It was observed that the actual growth rates obtained using the current data differed from the predicted values that were used at the design stage. This is expected since traffic prediction is generally very difficult and not so accurate. Since traffic prediction is very essential in coming up with realistic designs, a lot of care should be taken to avoid gross underestimation or overestimation of traffic.

4.1.2. Axle Load and Tyre Pressure Surveys. Axle load surveys carried out along the road are as shown in Table 2. It was observed that 2-axle trucks generally had heavier axles than other truck categories. It was further importantly observed that fifty percent of 2-axle trucks in the in-bound direction (towards Kampala) had axle loads that were above the legal limit of 10,000 kg. These trucks typically carry sand or agricultural produce towards Kampala city. A similar trend was exhibited by 3- and 4-axle trucks. Semitrailers (5- and 6-axle trucks) exhibited higher axle loads in the outbound direction (towards Masaka). However, the 75th percentile load (the load exceeded by 25% of axles) in the outbound direction is below the legal tandem axle load limit of 9000 kg/axle and only about 800 kg above the legal tridem axle load limit of 8000 kg/axle.

In order to compute traffic loading, Vehicle Equivalency Factors (VEF) were calculated from the axle loads using an equivalent single axle load of 80 kN as shown in Table 3. It should be noted that VEF values used at the design stage could not be found hence no comparison was done between the design and measured values. Using the measured values, an estimated design loading of approximately 49 MESA was determined. This was not significantly different from the actual design value of 44.1 MESA hence indicating that the designed pavement structure should be strong enough to carry the current traffic loading.

Furthermore, truck tyre pressures were measured and ranged between 570 and 1100 kPa with a median value of 900 kPa. These values are extremely high when compared to standard tyre pressure ranges of 550–700 kPa assumed in design. High tyre pressures in combination with high axle loads are well known to induce higher stresses hence resulting in significant damage to pavements [31].

4.2. Field Investigations

4.2.1. Reconnaissance and Roughness Surveys. The main defect observed on Kampala–Masaka road during the reconnaissance survey was the rutting of the asphaltic surfacing. Some sections of the road experienced cracking that was not widespread but rather intermittent. Sections were selected on the road for further detailed analysis as a function of the main defects observed and the description is as shown in Table 4.

A roughness survey was conducted on the road in both directions and analysed using the International Roughness Index (IRI). Except for two spots of 100 m each, a big portion of the road generally had IRI less than 6 as shown in Figure 1 which is indicative of a very good road condition. However, for sections that showed obvious rutting (i.e., rut depth ≥10 mm) such as S2 and S3, the roughness on the rutted lane was significantly higher than that on the lane with low rutting.

4.2.2. Cracking. Cracks observed on the road were evaluated based on their type, width, intensity, extent, and position. As shown in Figure 2, the cracking index (i.e., a product of crack intensity and extent) generally showed that the road did not experience severe cracking. The highest cracking index value of 16 out of the maximum possible value of 25 was observed in section S4 that generally exhibited longitudinal wheel path cracking and interconnected cracks. The other sections of the road exhibited values below 10 which implied minimal cracking.

Furthermore, cores extracted from the cracked sections of the road were visually evaluated. As shown in Figure 3, the top-down cracks were observed in the cores but were found to be confined only in the upper portions of the surfacing. This indicated that the cracking observed on road was confined to the surfacing layers and did not propagate through all the layers of the pavement.

4.2.3. Rutting. Rut measurements which were carried out in both directions and processed in terms of mm of rut depth indicated that some of the sections had obvious rutting. As shown in Figure 4, highly variable rutting was experienced with occasional peaks indicating variability in performance as a result of either construction or traffic behaviour.

However, test pits excavated in all rutted sections indicated that the rutting was confined to the surfacing and did not extend to the lower pavement layers, as shown in Figure 5. This further confirmed that the failures experienced on the road were mainly confined to the surfacing layers as already indicated by visual observation of cracks on cores. A more detailed understanding of the causes of defects observed in the surface layer is discussed in the results of the laboratory testing section.

4.3. Laboratory Investigations

4.3.1. Test Pit Materials. Test pits excavated after removal of the asphaltic surfacing showed that the material composition for the base and subbase layers was crushed stone and lateritic material, respectively. Materials obtained from the test pits were subjected to a range of tests and the results are

TABLE 1: Comparison of the design and actual growth rates.

Vehicle category	Number at design stage	Current number	Actual growth rate (%)	Predicted max. growth rate used for design (%)
Heavy trucks, 3-axle	123	N/A	8.3 (est.)	6.5
Medium/large single-unit trucks/ lorries	198	440	8.3	6.5
Truck trailers and semitrailers	116	136	1.6	6.5

est. = estimated from next vehicle class, N/A = current vehicle classification combines this with the next upper category.

TABLE 2: Percentile axle loads.

Trucks	Percentile	Towards Masaka	Towards Kampala
2 axles	75th % ile (kg)	9500	13100
	50th % ile (kg)	6300	12700
3 and 4 axles	75th % ile (kg)	8800	9500
	50th % ile (kg)	6800	9100
5 and 6 axles	75th % ile (kg)	8800	8700
	50th % ile (kg)	7900	7600

TABLE 3: Comparison of design and current Vehicle Equivalency Factors (VEF).

Vehicle category	Measured VEF	Remarks
Medium trucks (2-axle truck >5 tonne)	1.8	
Heavy truck (3-axle + 4-axle)	3.1	3-axles currently very rare
Semitrailer	5.8	
Trailer	5.4	Currently very rare

TABLE 4: Detailed sections selected on the road.

| Section	Start chainage	End chainage	Length (m)	Remarks/main defects
S1	25 + 000	25 + 200	200	Control section with minimal rutting. Mean gradient + 3%.
S2	31 + 100	31 + 300	200	Obvious rutting right-hand side (RHS). Mean gradient + 6%. No climbing lane.
S3	54 + 300	55 + 000	700	Obvious rutting on climbing lane, cracking on centre. Mean gradient + 8%.
S4	70 + 600	71 + 000	400	Several longitudinal wheel path cracks and interconnected cracks. Mean gradient + 1.8%
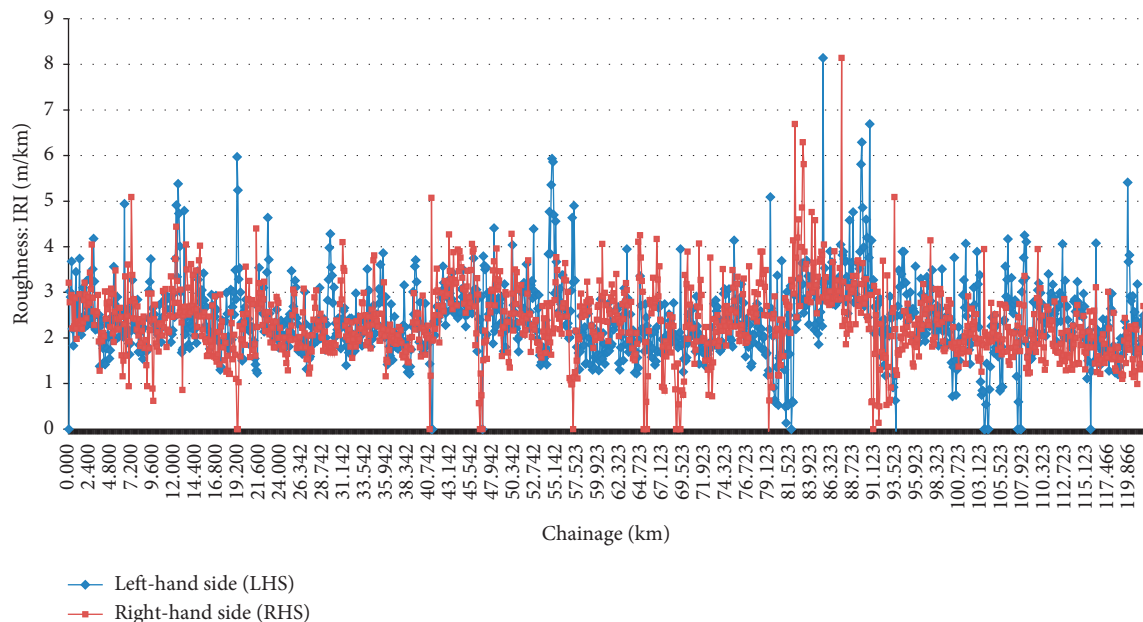
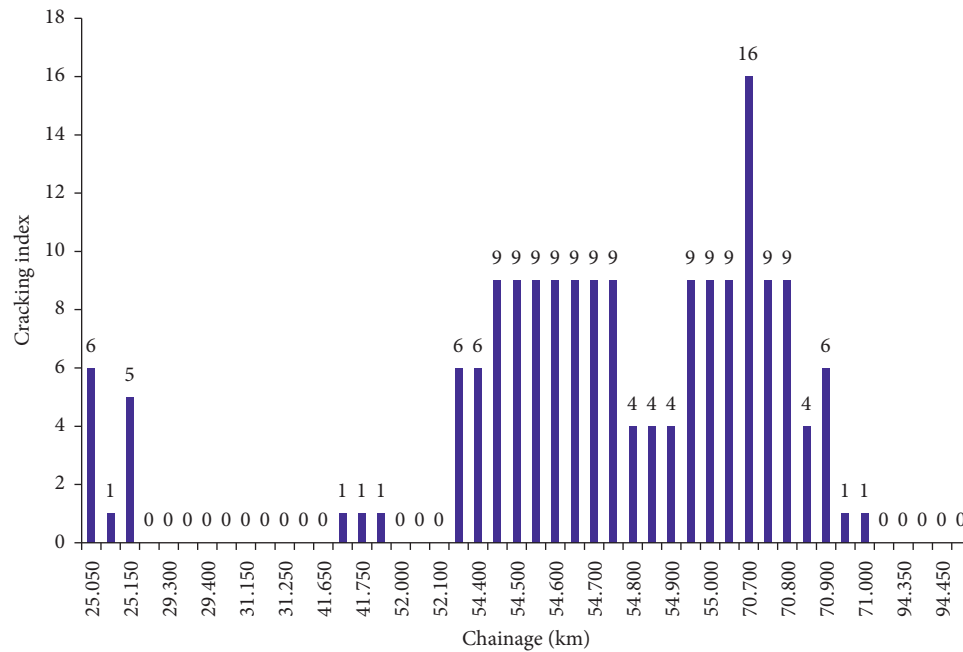FIGURE 1: Roughness trend along the road.

Figure 2: Cracking index for the road.

(a)

(b)

Figure 3: Interconnected cracks confined to the upper portion of the surfacing.
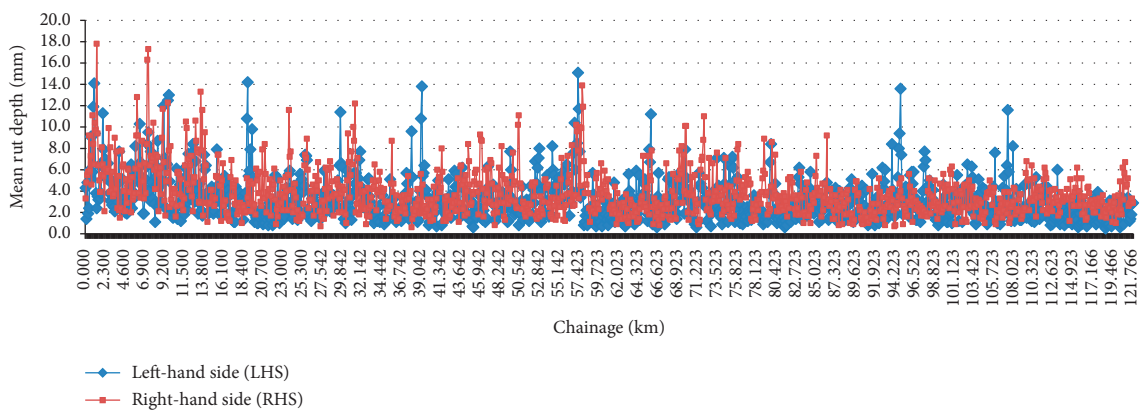

Figure 4: Rutting trend along the road.

Figure 5: Test pit at the rutted section of the road.

as shown in Table 5. Test results from control and failed sections were all found to be very similar, implying that failures observed on the road did not originate from the underlying layers below the asphalt surfacing. The 4-day soaked CBR values of subgrade from the control (S1) and rutted (S2) sections were 45% and 33%, respectively, which were all above the minimum specified soaked CBR value of 30% [32]. This clearly showed that the materials are of very good quality since the results were within specification. The above further confirmed the observations made in the test pits excavations that no structural rutting was experienced in all rutted sections but rather surface rutting. This is because structural rutting is usually a result of materials within the foundation undergoing shear while surface rutting is normally confined to the asphaltic surfacing [6].

4.3.2. Gradation. Figure 6 shows the particle size distribution (PSD) of aggregates (crushed stone) obtained from the base course.

It was generally observed that the grading of the materials obtained from the control and failed sections were all similar and within the specification envelope [32] for the base course. A similar trend was observed for aggregates obtained from the subbase course. Furthermore, Figure 7 shows the PSD of aggregates recovered from the wearing course after extraction of bitumen. Again, the recovered aggregates complied with the control points for Marshall Nominal Asphalt Concrete (AC 14) mixes used in the mix design [32]. The above clearly indicated that aggregate gradation was not responsible for the observed deteriorations on the road.

4.3.3. Indirect Tensile Strength, Marshall Stability, and Flow. Indirect Tensile Strength (ITS) testing was also carried out on dry cores extracted from different sections of the road and results are shown in Table 6. It was observed that all samples tested met the minimum specification requirement for ITS dry of 800 kPa [32].

Marshall Stability and Flow are the main strength and deformation characteristic tests of asphalt designed by the Marshall method. Since the road was designed by this method, it was important to assess the prevailing Marshall characteristics (Stability and Flow) and the results are shown in Table 6. For both the wearing and binder courses, it was observed that the minimum Stability and Flow values of the control section were within the acceptable specification requirements [32] for severe sites of minimum 9 kN and 3–5 mm, respectively. Similarly, the Stability and Flow values of the binder course of most of the failed sections were in conformity with the specifications. However, the stability values of the wearing courses for all failed sections did not conform with the specifications. This indicated that the failures observed on the road were limited to the wearing course. It further underpins the top-down cracking phenomenon observed during the visual assessment of the cores which was also confined in the upper portion of the surfacing. It is expected that the high traffic loading coupled with high tyre pressures led to loss of stability in the mix hence the failures observed on the road.

4.3.4. Air Voids. Cored samples obtained from different sections of the road were subjected to refusal density compaction and their bulk specific gravities and the theoretical maximum specific gravities were measured. The specific gravities obtained were used to determine the in situ air voids and air voids after Percentage Refusal Density (PRD). PRD is defined as the ratio of the initial dried bulk density of the sample to the final density (refusal density) expressed as a percentage [33]. Air voids and bitumen content obtained after extraction are as shown in Table 7.

Bitumen contents obtained after extraction were observed to be ranging between 4.6–5.0% and 4.2–4.7% for the wearing and binder courses, respectively. These were within the acceptable tolerance limits of the design bitumen contents of 4.5%–5.1% and 3.8%–4.4% for the wearing and binder courses, respectively. However, it was observed that almost all in situ air voids for the wearing and binder courses were below the recommended design air voids values. Asphalt mix design methods recommend acceptable design air voids ranging between 3 and 5% with 4% desired after

TABLE 5: Results from test pit materials.

Layer	Section	FMC (%)	OMC (%)	FMC/OMC	MDD (g/cc)	Field relative density (%)	Plasticity index
Base	S1	2.2	4.5	0.5	2.380	97.4	Nonplastic
	S2	3.0	5.5	0.5	2.286	97.8	Nonplastic
Subbase	S1	3.3	5.6	0.6	2.400	97.9	Nonplastic
	S2	3.2	6.0	0.5	2.295	99.3	Nonplastic
Subgrade	S1	9.3	7.8	1.2	2.138	96.7	9.8
	S2	11.2	8.9	1.3	2.191	98.6	9.7

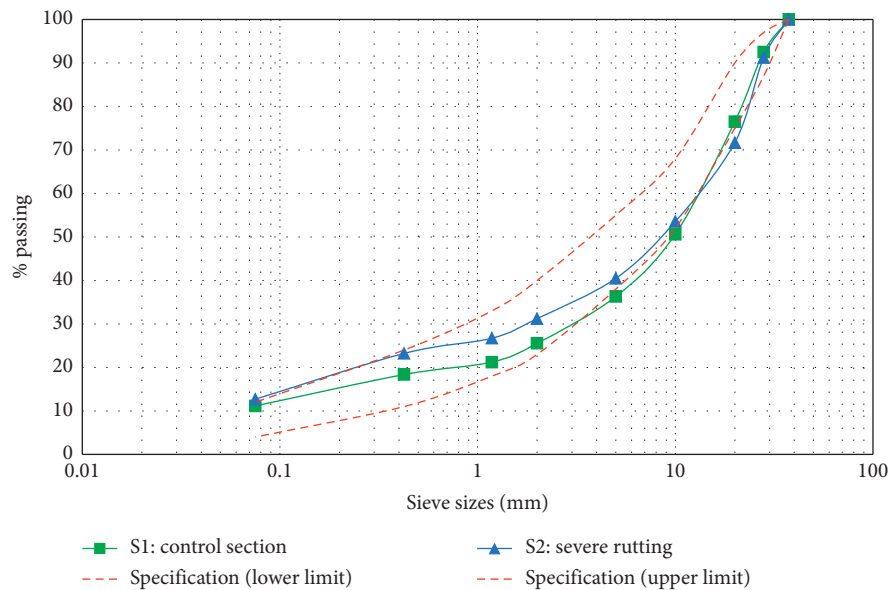FMC = field moisture content, OMC = optimum moisture content, and MDD = maximum dry density.
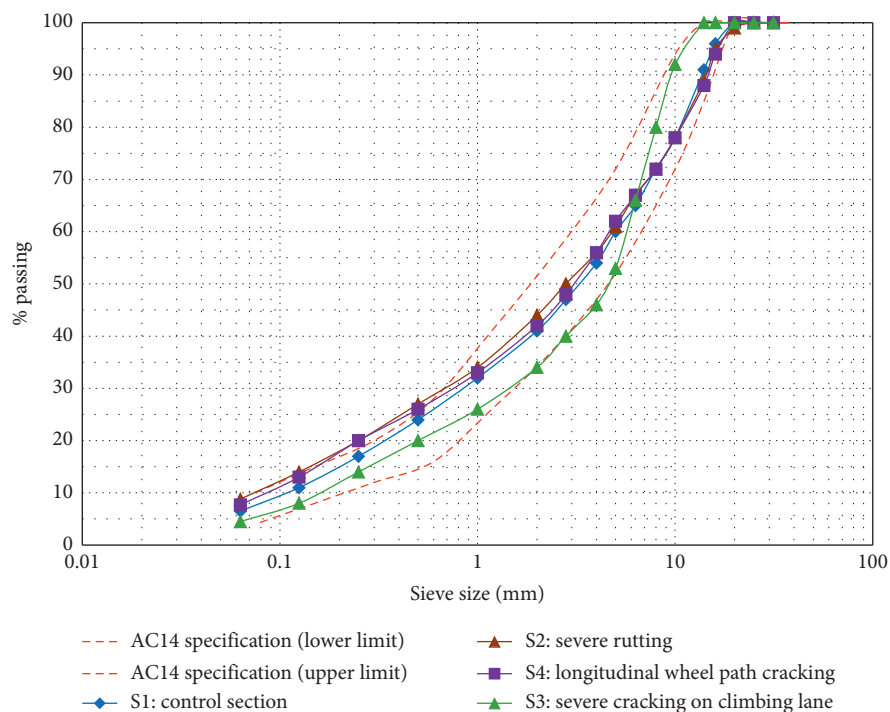

FIGURE 6: Particle size distribution—base course.

FIGURE 7: Particle size distribution of wearing course aggregate.

Table 6: ITS and Marshall results.

Section description	Wearing course			Binder course		
	ITS dry (kPa)	Stability (kN)	Flow (mm)	ITS dry (kPa)	Stability (kN)	Flow (mm)
S1: control section minimal rutting	1496	9.45	3.0	1290	9	4.1
S2: obvious rutting right-hand side (RHS), no climbing lane.	—	8.9	3.5	—	11.6	3.1
S3: obvious rutting on climbing, cracking on centre.	1281	7.0	3.1	1746	20.6	3.1
S4: several longitudinal wheel path cracks.	1972	4.5	1.9	1862	7.6	3.9

Table 7: Volumetric analysis of the asphaltic surfacing.

Section	Wearing course				Binder course			
	Bulk specific gravity	Air voids as received	Air voids after PRD	Bitumen content	Bulk specific gravity	Air voids as received (%)	Air voids after PRD (%)	Bitumen content (%)
S1	2.401	1.8	1.3	4.9	2.420	2.9	1.6	4.6
S2	2.427	1.3	1.2	4.6	2.445	1.3	0.7	4.2
S3	2.419	1.6	1.8	5.0	2.427	2.1	1.5	4.4
S4	2.394	2.6	2.5	5.0	2.413	3.5	3.0	4.7

Table 8: Penetration and Softening Point of recovered bitumen.

Section description	Penetration (dmm)	Softening point (°C)	Layer
S1: control minimal rutting	40	53.8	Wearing course
	42	56.0	Binder course
S3: obvious rutting on climbing, cracking on centre	35	55.8	Wearing course
	51	51.8	Binder course
S4: several longitudinal wheel path cracks	14	68.6	Wearing course
	8	64.4	Binder course

Table 9: Penetration and softening point before and after RTFOT.

Material	Brookfield viscosity (Pa.s)	Penetration (dmm)			Softening point (°C)		
		Before RTFOT	After RTFOT	Change in pen	Before RTFOT	After RTFOT	Change in SP
60/70 penetration grade bitumen	0.235	69	45	24	46.6	49.8	3.2

several years of trafficking [17]. Furthermore, it was importantly observed that air voids after refusal density compaction especially for the rutted sections were below the specification's critical minimum requirement of 3% after secondary compaction. Therefore, the observed rutting and deformation on the road can be attributed to the low voids in the asphalt mix. Research has shown that the amount of voids in a mix are paramount since they are very closely related to the stability and durability of a mix, with dense-graded mixes known to become unstable when the air voids go below 3% [17]. Generally, when the air voids in a mix are too low, then an asphalt mix is highly susceptible to permanent deformation in form of rutting, shoving, flushing, or bleeding. Provision of adequate air voids in the mix is important to ensure there is enough space in the mix to allow for expansion of the binder due to temperature increase as well as ensuring space for additional (secondary) compaction. This is usually achieved by carrying out a mix design to Percentage Refusal Density (PRD) [33].

Also, it was importantly observed that the rutting and deformation were more pronounced in the climbing lane of section S3 that had a steep gradient of 8% compared with the control section S1 that had a gentle gradient of 3% with minimal rutting being observed. It is expected that the steep gradient resulted in a severely loaded section, comprising of the already heavily loaded trucks observed during axle load surveys which were moving at very slow speeds. Such conditions in combination with the warm tropical environment (monthly air temperatures range between 14.5 and 38°C) experienced in the area exacerbate the situation and can cause the asphalt to begin to flow hence making rutting and deformation inevitable.

4.3.5. Recovered Bitumen. Bitumen recovered from cores was tested for penetration (pen) and softening point (SP) and the results are shown in Table 8. Furthermore, fresh (unaged) 60/70 pen samples of the same bitumen grade as

was utilised during construction were subjected to the Rolling Thin Film Oven Test (RTFOT) [34] in order to simulate short-term aging of the binder and results are shown in Table 9.

As shown in Table 8, it was observed that the penetration and softening point values of the recovered bitumen from the control (S1) and rutted (S3) sections were all within the normal expected range as defined by the fresh 60/70 pen samples after RTFOT shown in Table 9. However, the cracked section (S4) with severe longitudinal wheel path cracks had penetration values which were very low and softening point values which were very high compared with the fresh 60/70 pen samples after RTFOT. This simply implied that bitumen recovered from the cracked section had experienced more age hardening compared to that from the control and rutted sections. Furthermore, it was surprisingly observed that, for the cracked section, the recovered bitumen from the binder course had a lower penetration value compared to that from the wearing course. This is contrary to what would normally be expected since the wearing course is directly exposed to the sunlight and weather while the binder course is covered and shielded. Therefore, it would have been expected that bitumen recovered from the wearing course would age more and at a faster rate than that from the binder course since this course is protected from direct exposure to the environmental and climatic conditions by the wearing course. The plausible explanation for the severe ageing phenomenon observed in the cracked section is overheating of the bitumen during the construction process when mixing asphalt. Construction records showed that the different sections were constructed from different asphalt batches hence supporting the notion of overheating of bitumen in one of the batches during mixing of asphalt for the cracked section. Furthermore, it was importantly observed that cracking on the road was not widespread but rather intermittent hence further supporting the premise that bitumen overheating was specific to some few batches during mixing. It can thus be concluded that the cracking observed on a few sections of the road resulted from binder ageing due to overheating of bitumen during the construction process coupled with heavy truck axles.

5. Conclusions

Defects experienced on Kampala–Masaka road have been investigated in order to come up with appropriate measures to rectify them. The main defect observed on the road was the rutting of the asphaltic surfacing. Some sections of the road experienced top-down cracking that was not widespread but rather intermittent. Sections were selected on the road as a function of the defects observed and a detailed analysis was further conducted. Based on the investigation carried out, the following conclusions can be drawn:

(1) Rutting and deformation observed on the road are due to heavily loaded trucks that move at very slow speeds due to steep gradients, hence resulting in severely loaded sections. Such severe loading in combination with high temperatures in the area due to the warm tropical environment exacerbate the situation and can cause the asphalt to begin to flow, hence making rutting and deformation inevitable.

(2) This was further made worse by the low air voids in the asphalt mix whereby in situ voids were found to be below the recommended design air voids specification and air voids after refusal density compaction also being below the specified critical minimum of 3% after secondary compaction. The defects were thus a result of an unstable asphalt mix which was not suitable for the severe loading conditions caused by heavy trucks travelling at slow speeds and therefore the asphalt concrete that was laid was out of specification.

(3) Top-down cracking observed on a few cracked sections of the road was determined to have resulted from binder age hardening and embrittlement due to overheating of bitumen during the construction process coupled with heavy truck axles. Since cracking on the road was not widespread but rather intermittent, this supported the premise that bitumen overheating was specific to some few batches during mixing. It is expected that the high traffic loading coupled with high tyre pressures and binder age hardening and embrittlement due to overheating of the bitumen led to the loss of stability in the mix, hence the defects observed on the road.

(4) Defects observed on the road were confined to the wearing course layer of the surfacing with the other underlying layers performing well. This was confirmed by visual assessment of cracks on cores and test pits excavated in all rutted sections. Also, materials obtained from test pits in both control and failed sections were found to be very similar and within the acceptable specification requirements, implying that failures observed on the road did not originate from the underlying layers below the asphalt surfacing.

Based on the findings of this investigation, the following recommendations are provided:

(5) For severe sites where slow speed or heavier traffic is expected, the Modified Marshall Mix Design method (a combination of the Marshall Mix Design and Refusal Density testing), should be used. Also, the binder grade should be adjusted so that a stiffer binder can be used.

(6) Since the asphalt concrete laid was out of specification, improved quality control and assurance especially for bitumen is required when handling bitumen during the mixing and placing stages to avoid overheating.

(7) If a rehabilitation design is undertaken, it should take advantage of the strength of the existing underlying pavement layers which were found to be performing well in order to minimise the remedial costs while achieving the required design life of 20 years or estimated cumulative traffic loading in MESAs.

Data Availability

The data used to support the findings of this study are included within the article.

Conflicts of Interest

The data used to support the findings of this study are included within the article.

Acknowledgments

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