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RESEARCH ARTICLE

MECHANISTIC ANALYSIS OF EARLY PAVEMENT DETERIORATION: APPLICATION TO THE GASSI - DOURBALI SECTION

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Abstract

This work deals with the empirical mechanistic analysis of pavement behaviour: application to the Gassi - Dourbali axis in Chad. It consists of carrying out a study to determine the probable causes of the deterioration observed on the section in question, and of recording the deterioration. Test pits on the asphalt section, distributed according to the severity of the deterioration, will be used to determine the thickness of the various layers of pavement and their geotechnical characteristics. Deflection measurements will be carried out over a distance of two thousand (2000) metres and on particularly deteriorated areas in order to determine the residual capacity of the pavement. The traffic study included a 3-day vehicle weighing campaign and a counting campaign at at least three (03) stations for a period of seven (7) days. Based on the information available, we reviewed the project from its conception to its completion, and even its termination. The main reason for the work being halted was the early deterioration of the first 54 kilometres of asphalt. What went wrong?The Deming wheel is a tool for analysing and diagnosing projects. Plan Do Check Act is a question and answer method. It has enabled us to plan our actions, namely: surveys of deterioration, the organisation of traffic counts, the axle weighing campaign, and then the launch of geotechnical tests. Following an analysis of the rutting mechanism, recommendations are proposed to reduce the risk of rutting and to repair the pavement. Cracks appearing on the surface as a result of the combined causes we have listed are accentuated by rain, wind and vehicle movements, which crumple the earthenware and cause it to break up into debris or masses.

Introduction:

The development of traffic and transported loads can lead to significant deterioration of pavements, in particular rutting, which is the main pathology of flexible pavements [1-4]. This phenomenon can result from deformations in the surface layer or deformations in the other layers of the pavement [2].

Flexible pavements are composed of three layers: an asphalt surface layer, a gravel base layer and a sub-base layer. This type of structure leads to complex behaviour, because the different layers can behave in a way that is dependent or independent of time, reversible or irreversible, and dependent or independent of temperature.

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Reinforcing a pavement means adding new layers of material to its upper part so that the whole (the old and new pavements) has a sufficient structure to support current and future traffic adequately [3]. In order to carry out this reinforcement in the best technical and economic conditions, it is necessary to know the residual value of the old pavement. [3].

In the initial reports of the control mission, a good start was made by reviewing the entire project and identifying any points that seemed absurd or ambiguous. But were its analyses tested, and if so, did it find any errors that needed correcting? And were the errors identified incorrectly assessed? Did they themselves review their own decisions and carry out a mid-term check using indicators that were nonetheless perceptible? Perhaps the error lies in the decision taken after reviewing the tender documents. And the main decision taken was to change the foundation layer from cement soil to a layer of litho-stabilised natural materials. The Deming wheel leads us to question every project. In this way, the sizing or organisation of the work can be questioned and corrected as each stage of the project is completed. Our programming will use this tool and our methodology will use it to assess the entire project. Diagnose and analyse the causes. Programme actions in the field, monitor inputs and outputs, look for the cause of any failure and correct it.

Hypothesis, diagnosis.

- The bedrock: Test pits have shown that it rests on a layer of clay, sometimes sandy, sometimes sandy with silt and clay. In short, a clay soil
- The sub-base: to correct the quality of the sub-base soil is a litho-stabilised mix (silty sand and quarry sand). Such a mixture will not do much to improve the bearing capacity of the sub-base soil, especially in the presence of water.
- The sub-base layer: This is a litho-stabilised mix. Whatever its compaction rate, it will be sensitive to water and its bearing capacity will drop in the rainy season or as a result of infiltration of the underlying water. In appearance, with compaction, it will be a resistant soil, but its resistance may drop with the presence of water.
- The base layer: This is made of crushed gravel. It is a good filter for water drainage. It rests on a sandy clay soil made more impermeable by compaction. Water penetrating the base layer will be trapped because the bitumen is above and the compacted clay below. It is to be feared that the horizontally circulating water will wash away the gravel, gradually extracting fine elements. A void will be created between the bitumen and the base. From a behavioural point of view, the sub-base course and the foundation will settle together, dragging along some of the wet particles from the base course and causing an imbalance in the base course. The latter in turn loses more and more of its adhesion to the wearing course.
- The wearing course: this should act as a waterproofing layer, protecting the pavement layers and ensuring that they are bonded together for traffic comfort. But given the state of its cracks, this role has failed. There are several reasons for this:

a. The rapid drying out of the asphalt concrete surface due to sunshine, ambient temperature and dry air;
b. Failure to bond properly to the impregnation layer or failure of the impregnation layer to bond to the crushed gravel;
c. HGVs driving over the uncured bitumen at high speed;
d. the tensile forces developed by the braking of site machinery or traffic, as the bitumen behaves like a skin stretched over a drum that punctures in several places;
e. The shattering of tiles by heavy goods vehicle braking.

Diagnostic conclusion

In the light of the above, we report the following;

- the clay layer under the subgrade has not been sufficiently investigated. It is likely to be the cause of uniform or deferred subsidence of the entire subgrade and foundation made of compacted clay material. Similarly, the water table has not been highlighted for the project, despite the proximity of the Chari River, which runs alongside the entire route of the road and rises and falls during periods of high water. These phreatic movements affect the water content of the subgrade soil and contribute to the softening of the structure of the clay layers.
- The sol-cement rejected during the checking mission was the best solution for a weak soil such as the clay base of the pavements. It would give rigidity to the sub-base and absorb the shocks transmitted to the sub-base soil, while reinforcing the waterproofing capacity of the sub-base. In this way, consolidation of the subgrade with the base soil would be more uniform. Below is a model of the behaviour of the pavement produced.

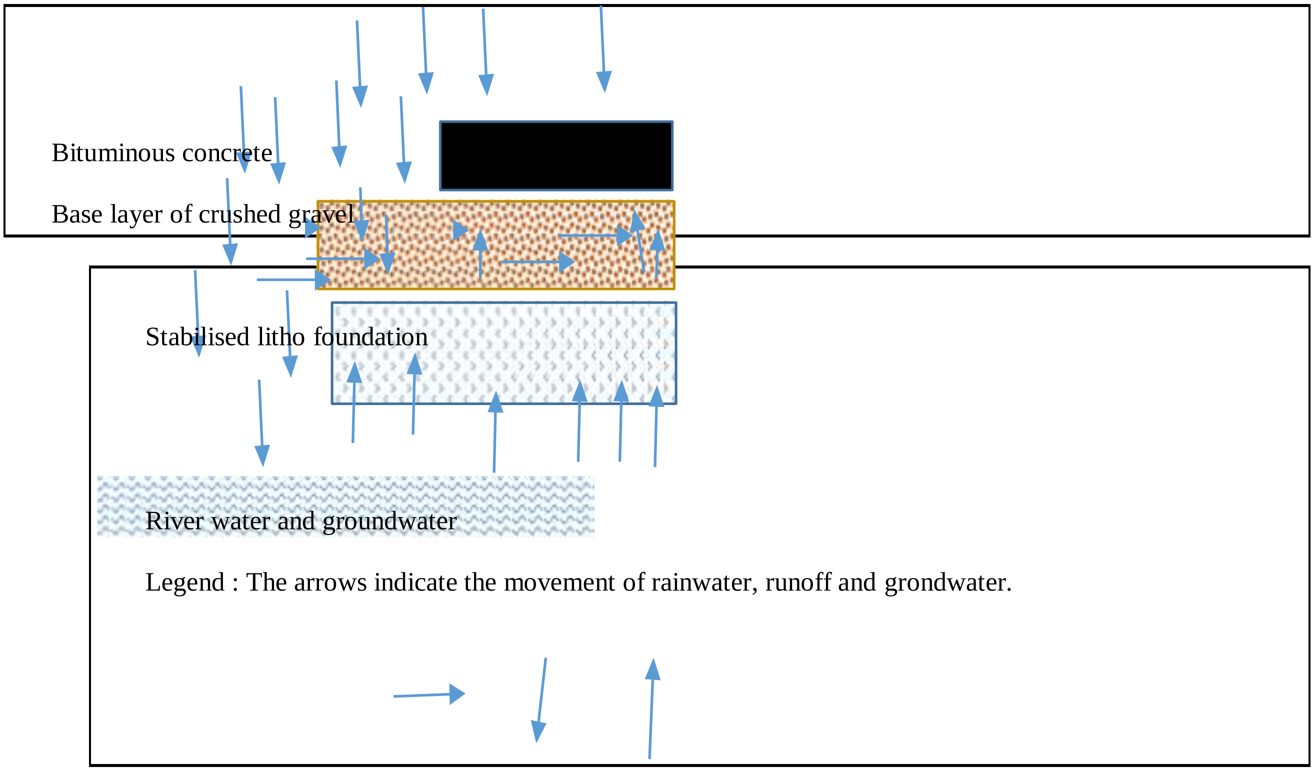


Figure 1:- Modelling.

Degradation survey

Altitude(m)	PK(Km)	Degradation	Total surface area m2	Intensity (m2/km)	Location
297	0		0	0	Gassi
306	0,1	Cracks	102,44	1024,4	
303	0,2	Earthenware	236,26	1181,3	
305	0,676	Pothole	2378	3517,751479	
307	1,003		3936,36	3924,586241	
305	2,007	Spalling	5729,83	2854,92277	
309	4,008		14959,87	3732,502495	Bakara
310	5	Earthenware	17358	3471,6	
315	10	Earthenware	23057,62	2305,762	Klessoun
307	15	Earthenware	23699,18	1579,945333	Mabrio
303	20		26188,82	1309,441	
312	25,264	Pothole	45738,28	1810,413236	
307	30	Pothole	70237,2	2341,24	
312	35		99780,58	2850,873714	

301	40	Earthenware	106214,04	2655,351	
311	45		106217,44	2360,387556	
316	50	Earthenware	106217,44	2124,3488	

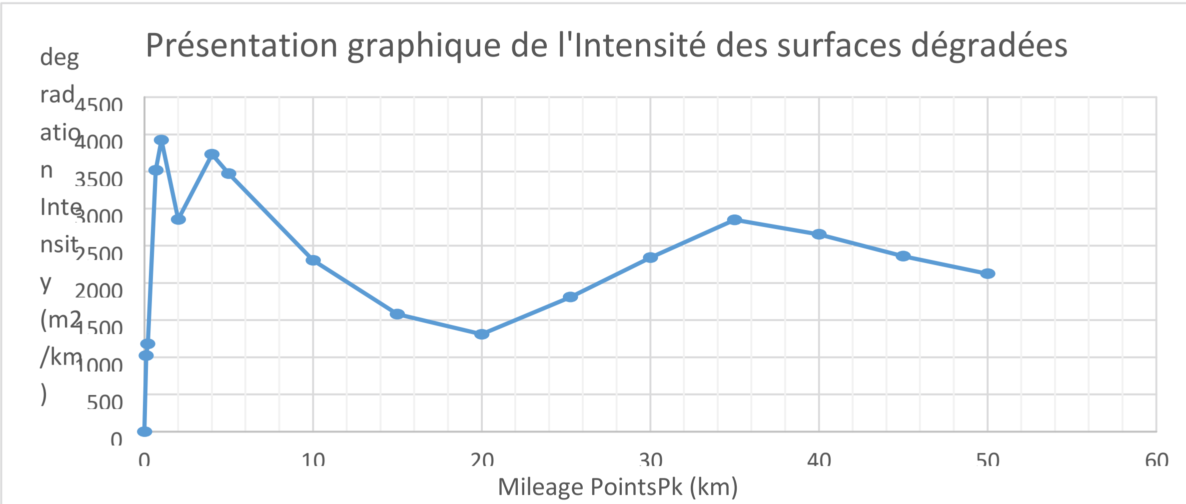


Figure 2:- Trend curve of pavement deterioration.

Rutting is a permanent deformation in the wheel arch. It is attributed to an increase in tyre pressure [1] and to the development of traffic and axle loads [4].

Ruts are classified into three categories: small (6 to 12.5mm), medium (12.5 to 25mm) and large (>25mm) [5]. However, three types of rutting are reported in asphalt concrete pavements:
- Wear rutting: this is caused by the gradual loss of aggregate particles from the surface layer. This type of rutting is caused by a combination of the environment and traffic.
- Structural rutting: this is the result of permanent vertical deformation in the pavement structure under repeated traffic loading.
- Instability ruts: these are caused by the lateral displacement of materials in the pavement layer and are all the more important for pavements made of materials with poor structural properties.

Several mechanisms can be at the root of rutting [7]. Rutting in asphalt is caused by densification and increased shear. The initial rutting is caused by the densification of the pavement under the passage of tyres. The development of rutting is linked to shearing, which leads to the formation of a bead around the wheel arch. However, for well compacted pavements, shear stress in the asphalt concrete layer is the main rutting mechanism. Pavement rutting is caused by the following mechanisms [8]:
- Vertical plastic deformation in the asphalt layer.
- Lateral creep in the asphalt layer (resistance to this mechanism can be defined as the ability of an asphalt mix to maintain its mineral skeleton intact).
- Deformation of the lower layers.

Several factors can influence this mechanism, such as pavement construction, traffic-related factors (tyre type, axle load, travel speed, inflation pressure) and environmental factors.

Figure 3:- Pavement crazing.

Looking at this photo, we can see a large hole in the pavement and widespread cracking with deep cracks.

The hole is due to punching of the already cracked surface. Wind and water, plus heavy goods vehicle traffic, were among the potential causes of this localised deterioration. This allows us to accept that heavy goods vehicles overloaded the pavement.

As for the faience, it is the opening of surface cracks on the asphalt concrete that has lost its adhesion with the base layer. The reasons for this can range from the way the bitumen is applied, depending on the temperature and climate at the time, to the phasing time between successive layers.

Figure 4:- View of cracking and destruction of pavement.

This example of pavement deformation gives the impression that there was no bonding layer before the asphalt concrete. The structure of the asphalt itself could be involved. It is a break in a composite aggregate of materials placed in bulk by a tremor. This breakage assumes that there was a void between the base layer and the bituminous concrete, as we showed above in theory. The over-compaction of the base layer may also be at the origin of its rigidity and the rapid drying of its face before the application of the bituminous concrete may also be the cause of this brittleness of the GNT.

Figure 5:- View of a pothole in the middle of the cracks.

This photo shows a deep pothole where the base layer has been reached. This base layer has, of course, disintegrated without cohesion. The tiles surrounding the pothole are already in loose patches. All it will take is the braking of a vehicle or some kind of traction force to tear them apart. The base course should be protected by the wearing course. The structure of this base course also reveals a non-conforming grading.

The figures above clearly show the serious deterioration of the pavement. Useful comments will be supported by deflection tests and a description of the pavement structure revealed by the geotechnical tests.

Geotechnical campaign

Field tests were carried out and continued with deflection tests along the paved section of the road (54 km) at the end of January 2023. The first wave of tests concerned the first 15 km, which were the most degraded, and consisted of open-pit boreholes down to a depth of 1.20 m. These tests made it possible to establish the lithology of the layers crossed with the thicknesses of the pavement layers. The results are summarised in the tables below.

Laboratory tests were also carried out on the soil samples taken during the manual well tests. They were used to characterise the materials of the layers encountered. The

Summary of geotechnical drilling results

This section sets out the lithological cross-section of the soils encountered from the surface course to beneath the platform. The thicknesses of these layers, the nature of the materials in the layers and the characteristics of these materials, in this case their grain size, their possible plasticity and their ICBR bearing capacity, are the essential elements determined and reported in this table. The manual pits were used to re-establish the pavement structure along the road. The structural formula can be summarised as 0.05 BB - 0.15 GNT0/31.5 de principe - 0.20 GN litho stabilised - 0.30 Sable argileux - plateforme infini. In other words, a surface course of bituminous concrete + a base course of untreated gravel 0/31.5 in principle, but with a grain size whose maximum diameter varies between 20 and 31.5; + a sub-base course with a mixture of clay and gravelly soil + a backfill course of clay material resting on silty soil. The thicknesses are not consistent from one profile to another. This lithological cross-section of the roadway confirms our hypothesis from the first section, when we made a geotechnical assessment of the structure of the roadway. From bottom to top:

- In relation to plasticity and argillosity

- The subgrade is fine silty sand, but with a considerable plasticity index of between 16 and 19. This index indicates that the soil is not just silty but also clayey;
- A backfill layer of clayey sand with a plasticity index ranging from 18 to 20, i.e. more plastic and clayey than the platform soil;
- A foundation layer of a mixture of clay and gravel with a plasticity index of between 15 and 17, i.e. a plastic clay soil;

- A base layer of crushed gravel with a grading of 0/31.5, but generally between 0/20 and 0/31.5 depending on the profile;
- A bituminous concrete surfacing layer.
- In relation to ICBR bearing capacity
- An ICBR low bearing capacity subgrade floating between 6 and 12, i.e. a deformation modulus of between 30 and 60 MPa;
- A subgrade with an ICBR bearing capacity of between 15 and 21, i.e. a modulus of deformation of between 75 and 105 MPa;
- A sub-base layer with an ICBR bearing capacity of between 31 and 38, i.e. a modulus of deformation of between 115 and 190 MPa;
- A base layer with an ICBR bearing capacity of between 95 and 108, i.e. a modulus of deformation of between 455 and 540 MPa;
- A BB wearing course with a deformation modulus at 40°c equal to 1000 MPa.

Calculating of axle equivalence and coefficient of aggressiveness during weighing operations

Axle weight class (in tonne)	Axle frequency : f		Equivalence of the pavement a = (P/13)^4	Agression on the road = fxa
1_2	2	3%	0,00018	0,000
2_3	2	3%	0,001	0,000
3_4	0	0%	0,005	-
4_5	4	6%	0,014	0,001
5_6	5	7%	0,032	0,002
6_7	6	9%	0,063	0,006
7_8	7	10%	0,111	0,012
8_9	3	4%	0,183	0,008
9_10	4	6%	0,285	0,017
10_11	13	19%	0,426	0,083
11_12	2	3%	0,612	0,018
12_13	6	9%	0,855	0,077
13_14	5	7%	1,163	0,087
14_15	3	4%	1,548	0,069
15_16	4	6%	2,021	0,121
16_17	0	0%	2,595	-
17_18	1	1%	3,284	0,049
18_19	0	0%	4,101	-
19_20	0	0%	5,063	-
TOTAL	**67**	**100%**		**0,549**
Average number of axles in the HGV population				**2,792**
Average aggressiveness coefficient (CAM)				**1,532**

Design verification of the residual road structure after geotechnical testing

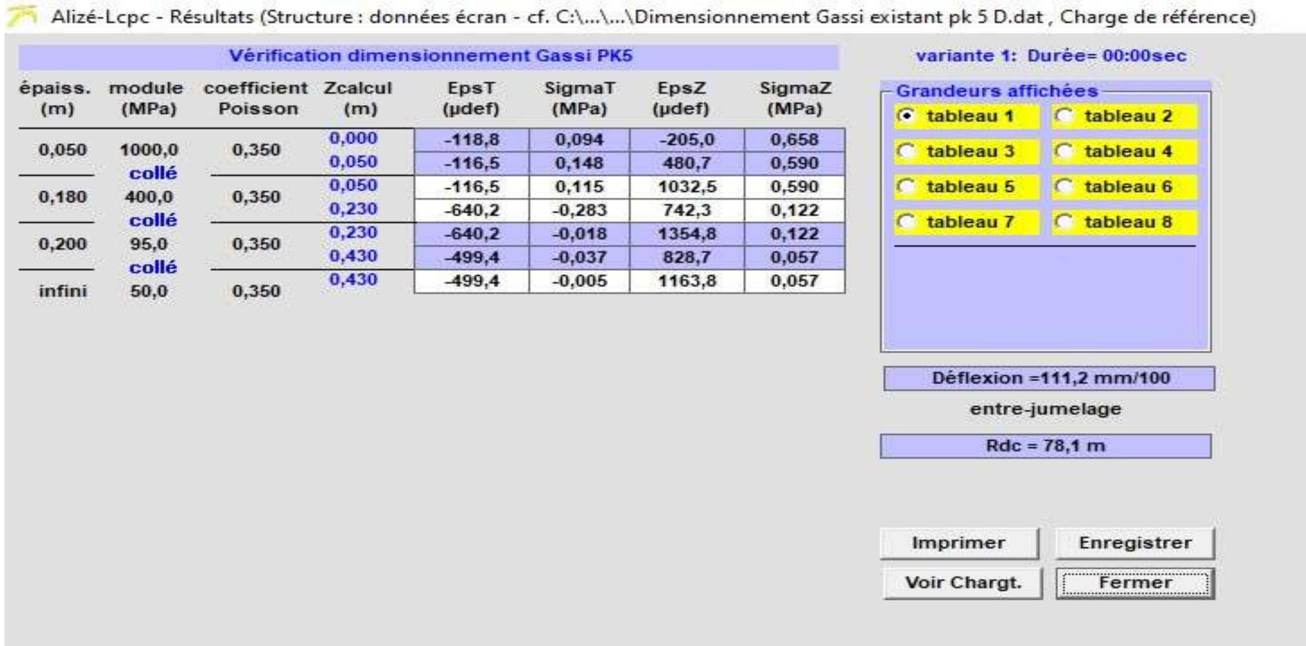
We carried out test pits along 30 km of the asphalt section to assess the thickness of the layers. Using the Alizé programme, we checked the residual deflection at the present time. The deflection values obtained are outside the permissible range and confirm the deflection tests carried out, whose spectrum characterises a fully damaged road.

The pavement structure encountered is formulated as follows:

1. 5 cm asphalt concrete wearing course BB
2. Natural gravel base course GNT 0/31.5
3. Sub-base layer of gravel plus stabilised litho clay
4. Forming layer with clayey sand fill
5. A silty sand platform

CBR tests were carried out on each layer, enabling us to calculate the deformation modulus E= 5 x ICBR in MPa.
The sections concerned by this verification are KP 5 (Gassi), KP 10+200, KP 19+300, KP 22+300 and KP 31+300.
The results of the calculations are presented in the following tables :

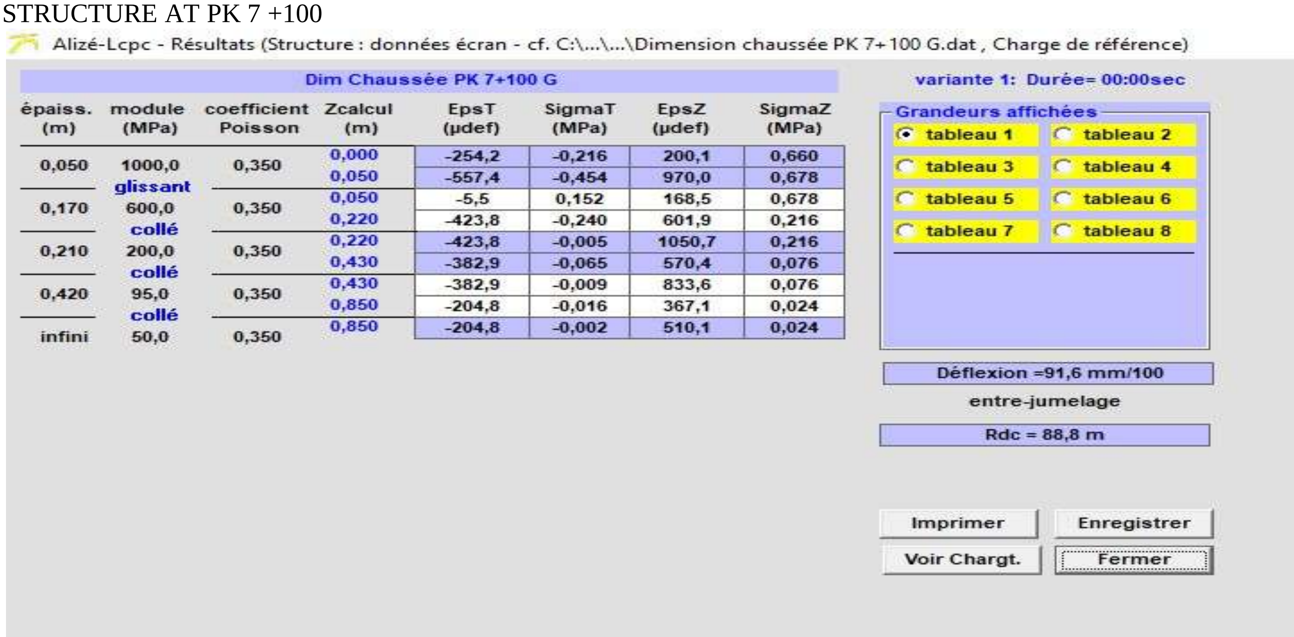
STRUCTURE AT KP 5+000 D



épaiss. (m)	module (MPa)	coefficient Poisson	Zcalcul (m)	EpsT (µdef)	SigmaT (MPa)	EpsZ (µdef)	SigmaZ (MPa)
0,050	1000,0	0,350	0,000	-118,8	0,094	-205,0	0,658
	collé		0,050	-116,5	0,148	480,7	0,590
0,180	400,0	0,350	0,050	-116,5	0,115	1032,5	0,590
	collé		0,230	-640,2	-0,283	742,3	0,122
0,200	95,0	0,350	0,230	-640,2	-0,018	1354,8	0,122
	collé		0,430	-499,4	-0,037	828,7	0,057
infini	50,0	0,350	0,430	-499,4	-0,005	1163,8	0,057

At PK 5 on the right-hand side leaving Gassi for Dourbally, with the materials found on site, the calculated deflection gives us a value of 111.2 mm/100. This value already exceeds the range of deflection values generally aimed for, between 60 and 90 mm/100.

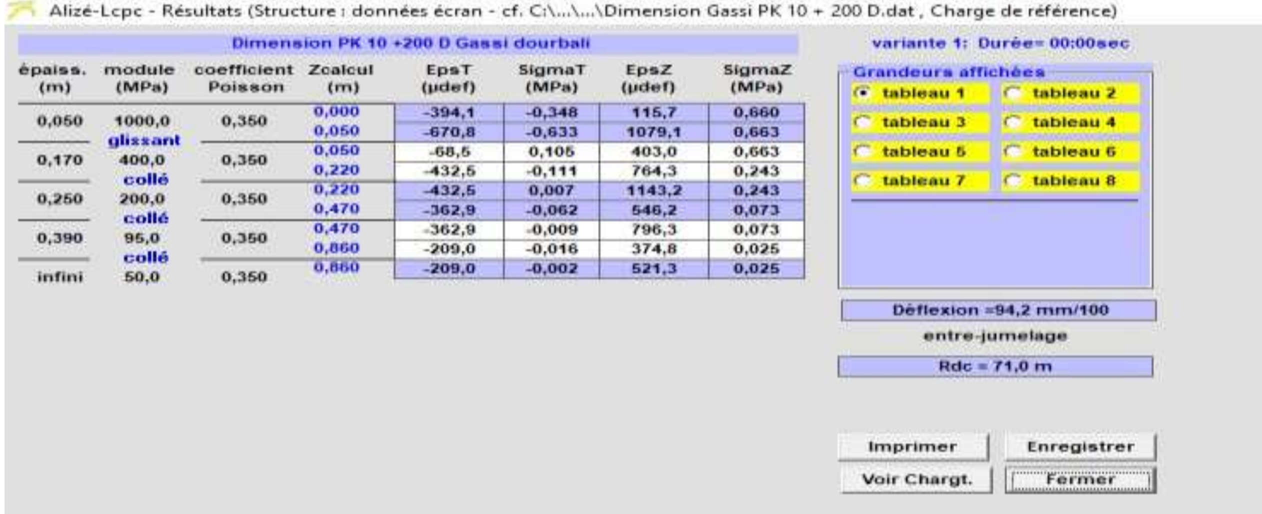
STRUCTURE AT PK 7 +100



épaiss. (m)	module (MPa)	coefficient Poisson	Zcalcul (m)	EpsT (µdef)	SigmaT (MPa)	EpsZ (µdef)	SigmaZ (MPa)
0,050	1000,0	0,350	0,000	-254,2	-0,216	200,1	0,660
	glissant		0,050	-557,4	-0,454	970,0	0,678
0,170	600,0	0,350	0,050	-5,5	0,152	168,5	0,678
	collé		0,220	-423,8	-0,240	601,9	0,216
0,210	200,0	0,350	0,220	-423,8	-0,005	1050,7	0,216
	collé		0,430	-382,9	-0,065	570,4	0,076
0,420	95,0	0,350	0,430	-382,9	-0,009	833,6	0,076
	collé		0,850	-204,8	-0,016	367,1	0,024
infini	50,0	0,350	0,850	-204,8	-0,002	510,1	0,024

The deflection is 91.5 mm/100

STRUCTURE AT PK 10+200


Alizé-Lcpc - Résultats (Structure : données écran - cf. C:\...\...\Dimension Gassi PK 10 + 200 D.dat , Charge de référence)

Dimension PK 10 +200 D Gassi dourbali

épaiss. (m)	module (MPa)	coefficient Poisson	Zcalcul (m)	EpsT (µdef)	SigmaT (MPa)	EpsZ (µdef)	SigmaZ (MPa)
0,050	1000,0	0,350	0,000	-394,1	-0,348	115,7	0,660
	glissant		0,050	-670,8	-0,633	1079,1	0,663
0,170	400,0	0,350	0,050	-68,5	0,105	403,0	0,663
	collé		0,220	-432,5	-0,111	764,3	0,243
0,250	200,0	0,350	0,220	-432,5	0,007	1143,2	0,243
	collé		0,470	-362,9	-0,062	546,2	0,073
0,390	95,0	0,350	0,470	-362,9	-0,009	796,3	0,073
	collé		0,860	-209,0	-0,016	374,8	0,025
infini	50,0	0,350	0,860	-209,0	-0,002	521,3	0,025

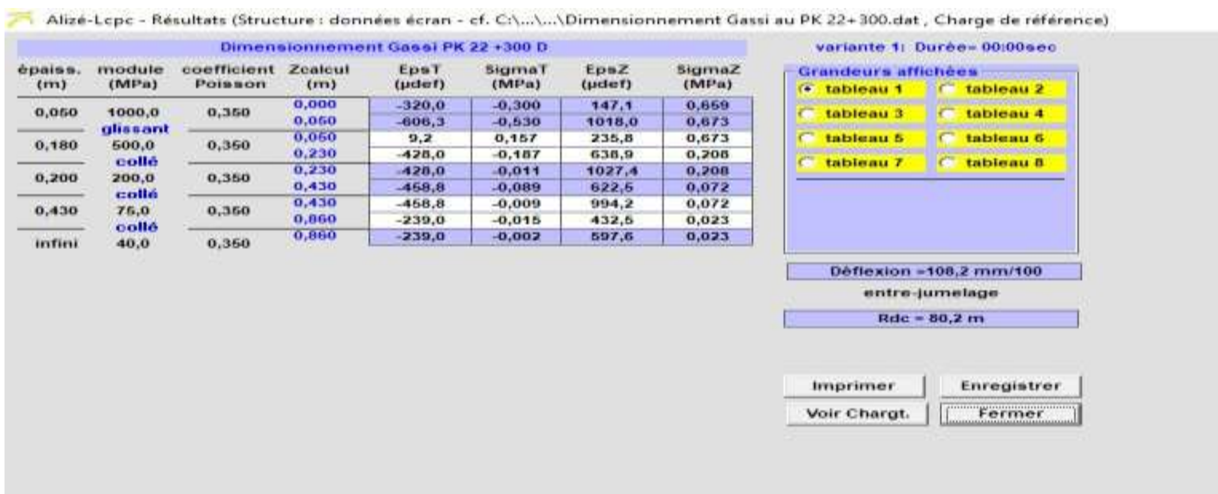
variante 1: Durée= 00:00sec

Grandeurs affichées
tableau 1 tableau 2
tableau 3 tableau 4
tableau 5 tableau 6
tableau 7 tableau 8

Déflexion =94,2 mm/100
entre-jumelage
Rdc = 71,0 m

Imprimer Enregistrer
Voir Chargt. Fermer

STRUCTURE AT PK 22 +300 D


Alizé-Lcpc - Résultats (Structure : données écran - cf. C:\...\...\Dimensionnement Gassi au PK 22+300.dat , Charge de référence)

Dimensionnement Gassi PK 22 +300 D

épaiss. (m)	module (MPa)	coefficient Poisson	Zcalcul (m)	EpsT (µdef)	SigmaT (MPa)	EpsZ (µdef)	SigmaZ (MPa)
0,050	1000,0	0,350	0,000	-320,0	-0,300	147,1	0,659
	glissant		0,050	-606,3	-0,530	1018,0	0,673
0,180	500,0	0,350	0,050	9,2	0,157	235,8	0,673
	collé		0,230	-428,0	-0,187	638,9	0,208
0,200	200,0	0,350	0,230	-428,0	-0,011	1027,4	0,208
	collé		0,430	-458,8	-0,089	622,5	0,072
0,430	75,0	0,350	0,430	-458,8	-0,009	994,2	0,072
	collé		0,860	-239,0	-0,015	432,5	0,023
infini	40,0	0,350	0,860	-239,0	-0,002	597,6	0,023

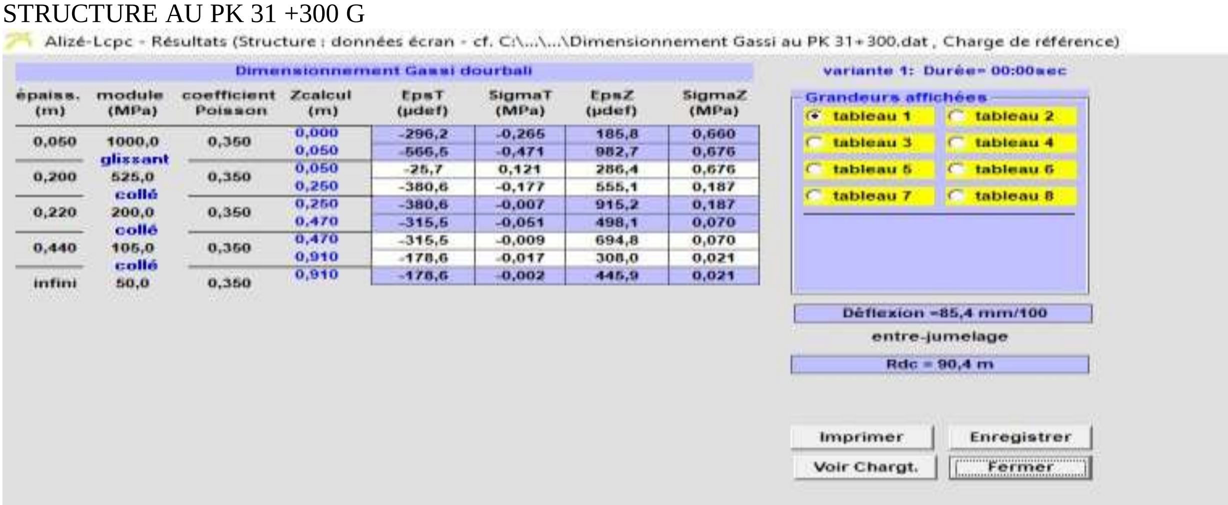
variante 1: Durée= 00:00sec

Grandeurs affichées
tableau 1 tableau 2
tableau 3 tableau 4
tableau 5 tableau 6
tableau 7 tableau 8

Déflexion =108,2 mm/100
entre-jumelage
Rdc = 80,2 m

Imprimer Enregistrer
Voir Chargt. Fermer

STRUCTURE AU PK 31 +300 G


Alizé-Lcpc - Résultats (Structure : données écran - cf. C:\...\...\Dimensionnement Gassi au PK 31+300.dat , Charge de référence)

Dimensionnement Gassi dourbali

épaiss. (m)	module (MPa)	coefficient Poisson	Zcalcul (m)	EpsT (µdef)	SigmaT (MPa)	EpsZ (µdef)	SigmaZ (MPa)
0,050	1000,0	0,350	0,000	-296,2	-0,265	185,8	0,660
	glissant		0,050	-566,5	-0,471	982,7	0,676
0,200	525,0	0,350	0,050	-25,7	0,121	286,4	0,676
	collé		0,250	-380,6	-0,177	555,1	0,187
0,220	200,0	0,350	0,250	-380,6	-0,007	915,2	0,187
	collé		0,470	-315,5	-0,051	498,1	0,070
0,440	105,0	0,350	0,470	-315,5	-0,009	694,8	0,070
	collé		0,910	-178,6	-0,017	308,0	0,021
infini	50,0	0,350	0,910	-178,6	-0,002	445,9	0,021

variante 1: Durée= 00:00sec

Grandeurs affichées
tableau 1 tableau 2
tableau 3 tableau 4
tableau 5 tableau 6
tableau 7 tableau 8

Déflexion =85,4 mm/100
entre-jumelage
Rdc = 90,4 m

Imprimer Enregistrer
Voir Chargt. Fermer

Summary of deflection and streed values for calculated profiles

Designation	PK 5+00	PK 7+100	Pk 10+200	Pk 19+300	PK 22+300	PK 31+300
Deflexion mm/100 C	111	91,5	94,2	102,5	108,2	85,4
sigmaZ stress in the base layer (MPa)	0,590	0,678	0,663	0,661	0,676	0,676
Deformation of epsilon Z on the plateforme (µdéf)	1163,1	510,1	521,3	818,7	597,6	445,9

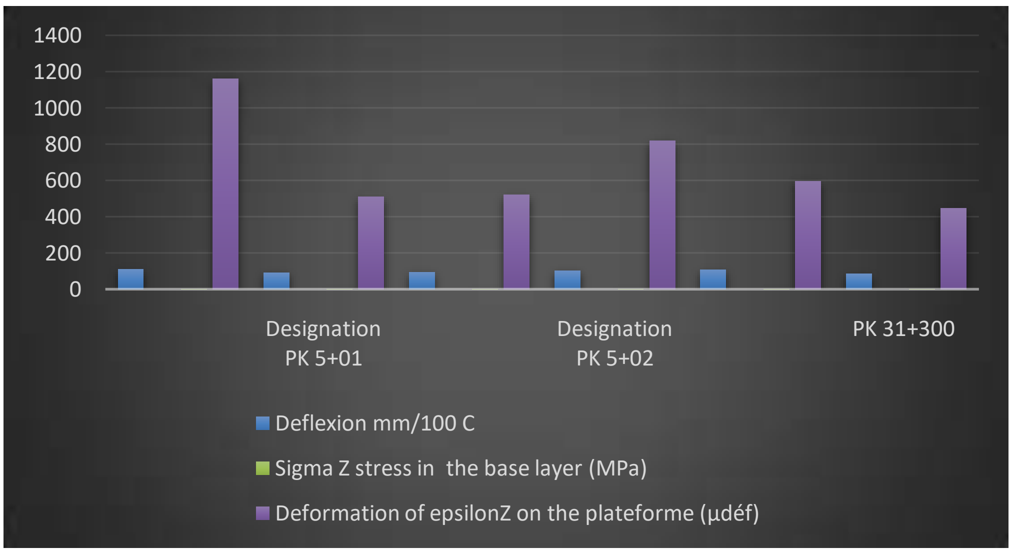


Figure 6:-Deflection – deformation.

This figure shows that the vertical deformation of the platform follows the general deflection and varies according to the vertical stress borne by the base layer. The values or orders of magnitude vary according to the kilometre points (KP). Overall, with the exception of kilometre point KP 31+300, the deflection values are greater than 90, which is the maximum permissible limit for values expected from field measurements. This indicates that the materials used in the construction of the carriageway were of low bearing capacity.

Summary of characteristic results in situ

Work area	Position	Nomber of test	Mini	Maxi	Medium	Deviation	Charactéristic deflexion
Pk 0+00 à PK 53+500	Gauche	107	32	202	103,5	40	155,3
	Droite	107	26	198	99,5	36,5	147,0

Interpretation of the deflection

According to the table above, the characteristic deflection values to the left and right of the road axis are sufficiently greater than the tolerable D90. They indicate the advanced state of deterioration of the road. The appropriate solution is to systematically rehabilitate the entire 54 km of road.

Conclusion:-

On the basis of the fieldwork carried out and the interpretation of the results obtained, the following points can be made:

1. The probable causes of the deterioration are

1.1. The nature of the subgrade (platform), which is a clay soil along the entire length of the road;

1.2. The pavement structure, in this case the materials of the sub-base (sand) and those of the litho-stabilised sub-base (sandy-granular) then the base course in GNT. This superimposition of all-granular layers makes the structure unstable, weakened by the percentage of fines which could lose their place in favour of voids;

1.3. The laying of the layers and the compaction gaps with the phasing time between two layers. Protection of previous layers while waiting for the active layer to be laid. Because of the grainy nature of the layers, the water content at the time of application should be saturated to enable them to adhere. One thing is to have achieved an acceptable level of compactness, but another is to be able to apply the next layer at this water content, which is often not the case on site. If the compaction rate exceeds 95% of the prescribed OPM, the material is already drying out. However, in the site logs we read rates of 98% to 105%, and it's only after another long period of time that the next layer is applied. Another point to note is the time between the impregnation layer and the laying of the asphalt concrete. Over time, the impregnating layer becomes a waterproofing agent on the base layer and the temperature at which the bituminous concrete is applied detaches it from the base and sticks to it. In this way, the base is detached and the impregnation layer is bonded to the bituminous concrete. This is what our damage surveys revealed. The photos above also show this;

1.4. The early traffic on the unprotected pavement. This shows that although there is not a lot of traffic on the road, the few lorries that use it are overloading. And these overloads are aggressive on the road;

1.5. The weighing campaign showed that despite the low traffic levels observed, some heavy goods vehicles are overloading on the road;

1.6. The cracks appearing on the surface as a result of the combined causes we have listed are accentuated by rain, wind and vehicle movements which crumple the tiles, causing them to break up into debris or masses.

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