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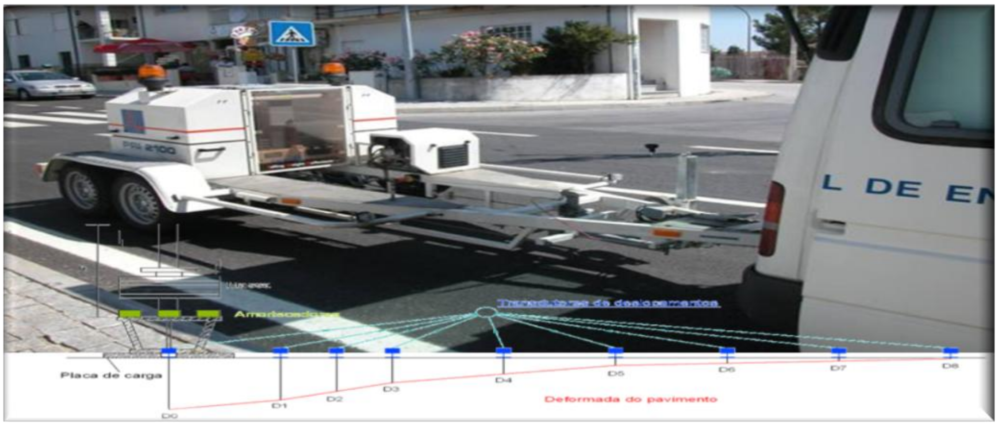

REHABILITATION OF FLEXIBLE PAVEMENTS THROUGH RECYCLING WITH CEMENT

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1. Introduction

With the elaboration of this study was intended to evaluate the load capacity of a road pavement submitted to rehabilitation with resource to cold in situ recycling with cement technique, and compare it with the load capacity of the same pavement, this time submitted to a normal reinforcement with bituminous mixtures. The results were obtained from load tests with a Falling Weight Deflectometer (FWD) device, and were interpreted having in account the effects of climatic conditions in which they were made, particularly the air temperature and the temperature of bituminous layers. The tests performed allowed to characterize the mixtures from the point of view of their stiffness, fatigue and permanent deformation.

2. Road Pavements Structural Behaviour

2.1. General Considerations

The main road pavement function is to ensure a free and unwrapped rolling surface that allows the circulation of vehicles in security, comfort and economic conditions, during pavement lifetime and being submitted to different traffic levels and a variety of climatic conditions.

According to the way in which we may associate layers formed by different types of materials, so results different types of pavements, which present different types of behaviours when requested by different traffic levels in combination with climatic conditions which they were submitted. Depending on the type of materials and its stiffness three pavement types can be distinguished. Flexible pavement using bitumen as a binder, rigid pavement using cement as a binder and semi-rigid pavement using bitumen and cement as a binder [1].

2.2. Quality Control

Together pavement and subgrade structural capacity can be evaluated in accordance with certain parameters, of which special mention is made to the surface vertical deflection, which is considered as the response from the pavement to the intensity of loads under certain conditions [1].

Among the equipment designed to study the deflections of pavements, only the Falling Weight Deflectometer (FWD) was considerate, because it was the device used in the bearing capacity evaluation of the case study pavement. With the application of these non-destructive load tests, the establishment of structural behaviour models for each study areas was intended. The most used design model for this purpose is the Burmister model.

The establishment of a pavement structural behaviour model involves not only the estimate of layers stiffness moduli, but also the adoption of a thickness soil top layer, since usually it becomes necessary to subdivide the subgrade soil into two layers, a top layer more deformable, which in the case study was adopted 2,5 meters, and another layer under the first one, with a semi-infinite thickness, designated by rigid layer with a significantly higher stiffness module than the first one [2].

3. Flexible Pavements Structural Rehabilitation

3.1. Strengthening of Flexible Pavements

The term pavement strengthening is referred as signifying the action or actions capable of increasing the structural capacity of an existing degraded pavement, and support, together with the subgrade, the loads caused by the passage of vehicles under certain conditions of application [1]. A possibility is the application of one or several bituminous overlays, another possibility may consist of milling the cracked layers and their replacement by applying new layers, aiming to adjust the pavement to the new traffic demands, and prevent the reflection of existing deterioration to the new surface course.

3.2. Pavement Recycling

The main purpose of road pavements recycling is to transform one or more layers of a deteriorated pavement into a homogeneous layer adapted to the new traffic demands. This technique consists in the reuse of the milled material and their application in the construction of a new layer, usually a base course that represents the new pavement main structural layer.

Considering aspects like the place of recycling, the production temperature of the recycled mixture and the used binder, thus can be defined several recycling processes [3]:

- Cold in plant recycling with bituminous emulsion.
- Cold in plant recycling with foamed bitumen.
- Semi-hot in plant recycling with bituminous emulsion.
- Hot in plant recycling with bitumen.
- Cold *in situ* recycling with cement.
- Cold *in situ* recycling with bituminous emulsion.
- Cold *in situ* recycling with foamed bitumen.
- Hot *in situ* recycling with bitumen / additives.

4. Cold *in situ* Recycling with Cement

4.1. General Considerations

Of the milled layers mixture with cement results in a new layer as like an extensive grading aggregate treated with cement (*AGEC*), which presents a higher bearing capacity than any of the previously existing [1]. The pavement that initially was flexible will become semi-rigid.

4.2. Description of the Constructive Process

The execution of this process includes the following operations [4] [5]:

- Prior study of the old pavement material and study of the mix design for obtaining the working formula for each distinct characteristics section.
- Mix the milled surface course material with water (optimal content), cement, additives (for improvement of recycled material characteristic) and aggregates (for grading correction of the final recycled mixture).
- Laying and pre-compaction with a steel wheel roller to avoid humidity losses
- Levelling and profiling the treated material with a grader.
- Compaction the profiled mixture to acquire the optimal density with a pneumatic tyre roller.
- Cure treatment of the recycle mixture for its protection.

In Figure 4.1 is shown the sequence of elementary tasks [5].

Figure 4.1 – Sequence of elementary tasks [5]

4.3. Pavements Design Methodologies Using Layers with Addition of Cement

Semi-rigid pavement in general, in terms of design, is treated like a flexible pavement. However, the existence of a rigid base course that includes the mixture of recycled material with cement reduces the stresses on the subgrade leading to an improbable permanent deformation occurrence, which exists in flexible pavements. For that, generally, permanent deformation is not a considered state limit of ruin.

With the purpose of calculating stress and strain states of the pavement, and using the most relevant of these values, the admissible numbers of passages by the standard axle $N_{130\ kN}$ that the pavement can support, three main criteria of design limit states of ruin were used.

The Shell criteria for design limit states of ruin:

- Shearing by fatigue on the basis of bituminous layers criteria:

$$\varepsilon_t = (0{,}856V_b + 1{,}08) \times E_{MB}^{-0{,}36} \times N_{130\,kN}^{-0{,}2} \qquad (4.1)$$

- $N_{130\ kN}$ – Accumulated number of passages of the standard axle;
- ε_t – Maximum traction stress induced by the standard axle;
- V_b – Bitumen volumetric percentage;
- E_{MB} – Stiffness moduli of bituminous mixtures (Pa).

- Permanently deformed on the top of the subgrade criteria (this criteria is generally not considered in this type of pavement, but it's of great importance in flexible pavements):

$$\varepsilon_z = 1{,}8 \times 10^{-2} \times N_{130\,kN}^{-0{,}25} \qquad (4.2)$$

- ε_z – Maximum vertical compression strain at the top of the subgrade.

And the shearing by fatigue at the bottom of mixtures of materials with hydraulic binders criteria, which is considerate of great importance in flexible pavements:

$$\frac{\sigma_t}{\sigma_r} = 1 + a \log N_{130\,kN} \qquad (4.3)$$

- σ_t – Maximum tensile stress induced by the standard axle (from ELSYM5 program);
- σ_r – Resistance to traction under bending ($R_{bending}$);
- a – Constant, which depends on the mix composition and properties, admitting values ranging -0,06 to -0,1.

5. Recycling With Cement Application Study on Rehabilitation of EN 226

5.1. General Considerations

This chapter presents the case study, concerning pavement structural rehabilitation of National Road 226 (EN 226), located in the district of *Viseu*, in which a recycling with cement technique was applied (designated by B solution) in the worse part of the pavement, and an overlay reinforcement with new bituminous mixtures in the remaining extension (A solution).

Comparison of the bearing capacity of the pavement section that was submitted to rehabilitation with resource to cold *in situ* recycling with cement, obtained of load tests with a FWD device, with the bearing capacity of the same section of pavement, only this time submitted to normal overlays reinforcement with new bituminous mixtures that previously were used in the better sections of the EN 226, was carried through.

Evaluated the pavement before rehabilitation two zones were chosen and were distinguished by presented the degree of degradation (zone 1 between km 14+000 and km 43+000 that appeared less degraded and zone 2 between kms 46+000 and 64+000, that presented a widespread degradation) [6], the next step was to find the solutions that ensure proper conservation/rehabilitation of the pavement.

For zone 1, particularly the extension in study between km 38+000 and km 40+000 designated *Solution A*, it was opted to use an overlay reinforcement with new bituminous mixtures, like the structure illustrated in the Figure 5.1

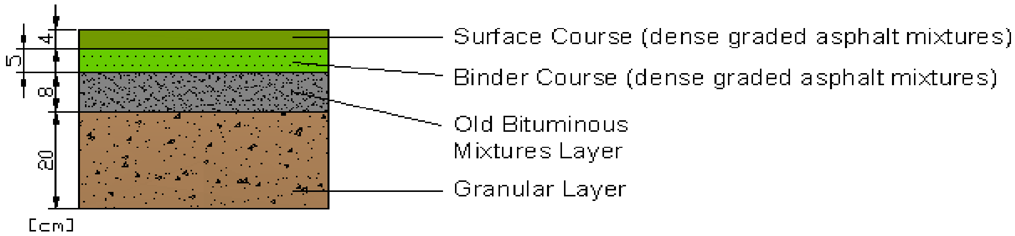

Figure 5.1 – Pavement structure for solution A (km 38+000 to km 40+000)

Throughout the zone 2, designated *Solution B*, it was considered the recycling of all the extension of the pavement. The new pavement structure presents an *in situ* recycled layer with 4% addition of cement in a thickness of 20 cm (base course with high mechanical strength), on top of it was applied a Stress Absorbing Membrane Interlayer (SAMI), in order to reduce the crack propagation phenomenon and to promote bonding with the following layer. The pavement structure solution is illustrated in Figure 5.2.

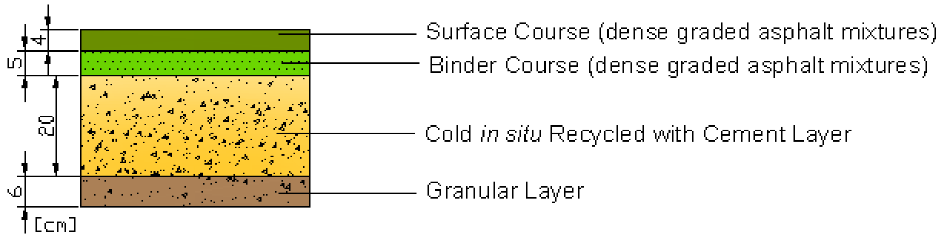

Figure 5.2 – Pavement structure for solution B (km 46+000 to km 64+000)

5.2. Pavement Load Capacity after Rehabilitation

Load tests were performed with a Falling Weight Deflectometer device in each one of the directions (D1 and D2), since pavements in the external rutting area may present distinct behaviours. The campaign took place between 24 and 26 June 2008. Initially, in each point, the load cell was adjusted to the pavement surface. With a second impact, being the drop height set to match the desired peak load, the deflections were measured. The peak load applied by the LNEC device was 65 kN, because it´s the value that simulate the standard axle of 130kN.

The pavement deflections induced by the impact load were measured at various points through geophones supported on the surface of the pavement. The distances from start point are located: D_0 - 0 m; D_1 - 0.30 m; D_2 - 0.45 m; D_3 - 0.6 m; D_4 - 0.9 m; D_5 - 1.2 m; D_6 - 1.5 m; D_7 - 1.8 m; D_8 - 2.1m. The Figure 5.3 illustrates the FWD load tests.

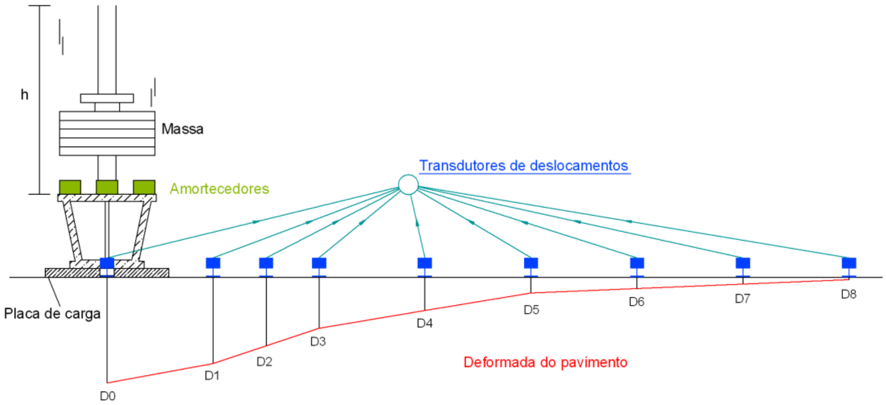

Figure 5.3 – Deflection obtained from start point[7]

With deflection values measured its normalization is realized, since that exist tiny impact forces variations in each point. The normalization is given by the following expression (5.1):

$$D_n = 65 \times \frac{D_m}{F_m} (\mu m) \tag{5.1}$$

- D_n – Normalized Deflection; D_m – Measured Deflection; F_m – Measured Force

The next step was the pavement division in several zones with homogeneous structural behaviour. This zone division for D1 and D2 directions is obtained by the method proposed by AASHTO (2001) [8], known as the cumulative differences area method (CAD). In the Zx cumulative difference graphics each slope of Zx changes indicates a behaviour change. A resume of the segment zones division adopted is presented in Table 5.1

Table 5.1 – Resume table of segment zone division

<table>
<tr><th rowspan="2"></th><th>Solution A</th><th>Solution B</th></tr>
<tr><th>km 38+000 to km 40+000</th><th>km 46+000 to km 64+000</th></tr>
<tr><td rowspan="3">Direction D1</td><td rowspan="3">Zone A: km 38+000 ao km 39+000</td><td>Zone B1: km 46+200 to km 50+400</td></tr>
<tr><td>Zone B2: km 51+000 to km 54+600</td></tr>
<tr><td>Zone B3: km 55+200 to km 64+000</td></tr>
<tr><td rowspan="2">Direction D2</td><td rowspan="2">Zone A: km 38+050 to km 39+750</td><td>Zone B1: km 46+250 to km 54+550</td></tr>
<tr><td>Zone B2: km 55+150 to km 63+850</td></tr>
</table>

Concluded the segment zone division and with the deflection values purified of non representative values, it was necessary to make a statistical treatment of the obtained data, aiming to calculate the percentile 85. The reliable interval of 85% is used since it considers that the representative deflection in each segment zone is the one that matches the deflection whose probability of being exceeded is inferior to 15%.[2]. Representative deflection of each zone is chose, which is the one that most closely approximates percentile 85.

5.3. Behavior Modeling in Load Tests

Knowing the pavement structure and its material characteristics and using an appropriate model for structural analysis, the fowling step consisted in determinate the stiffness moduli (E) for each different layer. It was used the retro analyse method, using for that purpose the automatic calculation ELSYM5 program, developed by Berkeley University (USA). Stiffness moduli are attributed to the different pavement layers and the deflections are calculated. These are compared with the deflections measured in the place. The procedure is repeated until the calculate deflection is as close as possible to the deflection measured in the representative test of this zone [9]. So, using this method, the representative stiffness moduli of structural behaviour of the pavement is obtained [10].

In Table 5.2 and

Table **5.3** the pavement characteristics are summarized, in particular stiffness moduli, corresponding to the values obtained using the ELSYM5 program, who's calculated deflections more approached the measured ones.

Table 5.2 – Stiffness moduli estimated by ELSYM5 program – Solution A

Solution A (Direction D1 e D2)				
Layers	**Thickness (m)**	**E (MPa)**		**ν**
		Zones		
		A-S1	**A-S2**	
Bituminous Layers	0,17	2400	2400	0,40
Granular Layers	0,20	150	120	0,35
Top Subgrade Layer	2,50	80	80	0,35
Rigid Layer	–	400	400	0,35

Table 5.3 – Stiffness moduli estimated by ELSYM5 program – Solution B

Solution B (Direction D1 e D2)							
Layers	**Thickness (m)**	**E (MPa)**					**ν**
		Zones					
		B1-S1	**B2-S1**	**B3-S1**	**B1-S2**	**B2-S2**	
Bituminous Layers	0,09	2000	2000	2000	2400	3000	0,40
Recycled Layer	0,20	1100	1200	1300	1000	1400	0,30
Top Subgrade Layer	2,50	60	50	60	60	60	0,35
Rigid Layer	–	300	250	300	300	300	0,35

A key aspect which has to be present when one pretends to evaluate pavements bearing capacity using models for structural analysis is the fact that climatic conditions may interfere in the mechanical behaviour of the layers materials and of the subgrade [2]. The temperature of asphalt concrete is a very important data item that is required for analysis of pavement deflection data, because of its influence on the stiffness moduli (E).

Thus it was necessary to make stiffness moduli correction to be possible to compare the design modulus with the ones estimated from tests. The design temperature of bituminous layers was 24° C.

The followed method to measure the bituminous layers d depth temperature (half of the total thickness of bituminous layers) was the BELLS3 method [11]. The new stiffness moduli are presented in Table 5.4

Table 5.4 – Estimated bituminous layers stiffness moduli to design temperature

Direction	Zone	Td – Pavement Temperature at desired depth d, (ºC) a)	E_t^{BM} - Design Temperature (MPa)	$E_{20^0C}^{BM}$ - Reference Temperature (MPa) b)	$E_{24^0C}^{BM}$ - Design Temperature (MPa) c)
D1	A	25,9	2400	2954	**2582**
	B1	22,1	2000	2137	**1868**
	B2	27,0	2000	2568	**2245**
	B3	29,8	2000	2901	**2536**
D2	A	30,3	2400	3566	**3118**
	B1	33,4	2400	4158	**3635**
	B2	28,0	3000	4021	**3515**

a) Pavement temperature at desired depth d (ºC) according to *BELLS3* equation [11]

b) $E_{20^0C}^{BM} = \frac{E_t^{BM}}{1{,}635 - 0{,}0317 \times T_d}$: *Expression for conversion of "E". Formula LNEC by Antunes, M. L.* [2] (5.2)

c) $E_{24^0C}^{BM} = E_{20^0C}^{BM} \times (1{,}635 - 0{,}0317 \times 24)$: *"E" at design temperature* (5.3)

5.4. Criteria for Design

Obtained the asphalt concrete stiffness moduli for design temperature $(E_{24^0C}^{BM})$, the following step consisted in the calculation of the acceptable number of passages of the standard axle $(N_{130\ kN})$ using for this purpose the design criteria referred in 4.3, now applied to semi-rigid and flexible pavements.

From the developed pavement model for structural response, horizontal tensile strain (εt) at the bottom of bituminous layers and vertical strain (εz) at the top of the subgrade were determinate, respectively to verify the shearing by fatigue criteria and the permanently deformed criteria (rutting) developed, as already been said, by Shell [10]. For the case of the recycled with cement layer, horizontal tensile stress (σt) was calculate at the bottom of this layer, to verify the shearing by fatigue of materials with hydraulic binders criteria [12].

The constraint data of strain and tensile stress calculate by ELSYM5 program for all zones, are presented in Table 5.5 and Table 5.6.

Solving the equations (4.1) and (4.2) in order to N, the $N_{130\ kN}$ is obtained to the Shell criteria, as shown in Table 5.5:

Table 5.5 – Constraint data of strain and $\boldsymbol{N_{130\ kN}}$ values from *Shell* criteria

Direction	Zona	Constraint Data Shear. Fatigue (ε_t)	Constraint Data Perm. Def. (ε_z)	$\boldsymbol{E_{BM}}$ (MPa)	$\boldsymbol{N_{130\ kN}}$ Fatigue a)	$\boldsymbol{N_{130\ kN}}$ Perm. Def b)	$\boldsymbol{N_{130\ kN}}$ Lowest Value
D1	A	2,81E-04	6,67E-04	2582	5,47E+05	5,30E+05	5,30E+05
	B1	7,33E-05	7,69E-04	1868	8,07E+08	3,00E+05	3,00E+05
	B2	6,94E-05	7,81E-04	2245	7,61E+08	2,83E+05 c)	2,83E+05
	B3	6,71E-05	6,86E-04	2536	7,22E+08	4,75E+05	4,75E+05
D2	A	2,65E-04	6,20E-04	3118	5,16E+05	7,10E+05	5,16E+05
	B1	1,13E-04	7,28E-04	3635	2,86E+07	3,74E+05	3,74E+05
	B2	7,16E-05	6,32E-04	3515	2,91E+08	6,57E+05	6,57E+05

a) By applying the equation (4.1) solved in order to N, with $V_b(\%) = 10$;

b) By applying the equation (4.2) solved in order to N;

c) Constraint value of $N_{130\ kN}$.

In the third criterion because the tensile resistance was evaluated from direct compression tests in several specimens extracted for this purpose [13], it´s admissible to use a 1,5 factor to convert the obtained value into the one expected from the bending test [14].

$$R_{bs} \approx 1{,}5 \times R_{cd} \tag{5.4}$$

- R_{bs} - Resistance to traction under bending stress (σ_r);
- R_{dc} - Resistance to traction under diametral plane compression.

Applying the equations (5.4) and (4.3) in order to N the data obtained is shown in Table 5.6:

Table 5.6 – Constraint data of tensile stress and $\boldsymbol{N_{130\ kN}}$ values from shearing by fatigue of materials with hydraulic binders criteria

Direction	Zone	Constraint Data Shear. Fatigue Mat. Hidr. Binders $(\boldsymbol{\sigma t})$ MPa	$\boldsymbol{\sigma r}$ a)	$\boldsymbol{\sigma t/\sigma r}$	$\boldsymbol{N_{130\ kN}}$ Fatigue b)
D1	A	n.a.	n.a.	n.a.	n.a.
	B1	0,43	0,93	0,46	unlimited
	B2	0,47	0,93	0,50	unlimited
	B3	0,45	0,93	0,48	unlimited
D2	A	n.a.	n.a.	n.a.	n.a.
	B1	0,38	0,93	0,41	unlimited
	B2	0,45	0,93	0,48	unlimited

n.a. - not applicable

a) By applying the equation (5.4), with $a = -0{,}08$;

b) By applying the equation (4.3) solved in order to N.

This last criterion is considered not constraint because the quotient σ_t/σ_r is less than $0{,}55$, by that it´s considered unlimited the number of passages of the standard axle ($N_{130\ kN}$).

With the results obtained it could be concluded that the acceptable number of passages of the standard axle is of similar order of magnitude for the two solutions, being slightly higher for solution A (Table 5.5). However the comparison of pavement solutions A and B made this way would not be representative, since the initial conditions found for both solutions weren´t similar. Effectively the traffic classes, the initial pavement degradation, the initial bituminous mixtures thickness (8 cm to solution A and 6 cm to solution B) and the different subgrade stiffness moduli which testify that the solution A pavement is founded on a better subgrade than the one of the solution B, do not allow that this comparison occurs under equality conditions.

To overcome this situation a hypothetical pavement was adopted in the section that presented the worst initial conditions and it was applied an overlay reinforcement as the one applied for solution A. This way, it was already possible to make this comparison, this time leaving of the same initial conditions.

Representative values of expected stiffness moduli for B zone were considered as if it was chosen the traditional overlay reinforcement used in A zone. For bituminous mixtures layers it was used the average value of corrected stiffness moduli determined for the A zone. The pavement structure solution is illustrated in Figure 5.4.

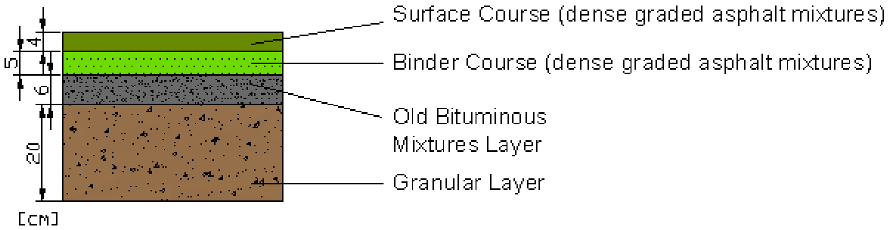

Figure 5.4 – Hypothetic Pavement structure the type of solution A applied to B solution section

It should be noted that in this section the pavement showed a high level of degradation for several distress types, namely cracking, by which the old bituminous layers behaved as granular material, and so, with a considerably smaller stiffness moduli than the new layers, as shown in Table 5.7.

Table 5.7 – Characteristics of hypothetical solution A pavement

Solução Hipotética A para Aplicação na Zona B			
Layers	**Thickness (m)**	E **(MPa)**	ν
New Bituminous Layers	0,09	3300[a)]	0,40
Old Bituminous Layers	0,06	300	0,35
Granular Layers	0,20	140	0,35
Top Subgrade Layer	2,50	60	0,35
Rigid Layer	–	300	0,35

a) Stiffness Moduli corrected to design temperature ($E^{BM}_{24^0C}$)

Then the procedure was the calculation of the acceptable number of passages of the standard axle ($N_{130\ kN}$) using the design criteria of Shell method and the automatic calculation ELSYM5 program.

In Table 5.8 are presented the constraint values of (ε_t) and (ε_z) relating the hypothetical pavement. Solving the equations (4.1) and (4.2) in order to N, the $N_{130\ kN}$ is obtained, as also shown in Table 5.8.

Table 5.8 –$\boldsymbol{N_{130\ kN}}$ values for Shell criteria (hypothetical pavement)

Direction	Hypot. Zone	ε_t	ε_z	E_{BM} (MPa)	$N_{130\ kN}$ Fatigue [a)]	$N_{130\ kN}$ Perm Def.[b)]	Traffic Classes
D1 /D2	A	3,65E-04	1,112E-03	3300	9,40E+04	**6,87E+04** [c)]	T5

a) By applying the equation (4.1) solved in order to N, with $V_b(\%) = 10$;

b) By applying the equation (4.2) solved in order to N;

c) Constraint value of $N_{130\ kN}$.

It´s now intended the confrontation of the $N_{130\ kN}$ obtained values of both solutions, thus to compare the bearing capacity of those pavements and to select the constraint design criteria.

5.5. Comparative Analysis of Rehabilitation Alternative Solutions

The results of the alternative solutions comparison are presented in Table 5.10 by comparing the obtained values for the different solution B zones which appears in Table 5.5, with the obtained results of the hypothetical A solution shown in Table 5.9.

Table 5.9 – Acceptable number of passages of the standard axle ($\boldsymbol{N_{130\ kN}}$)

Direction	All B Zones (B1,B2,B3 de S1 e B1,B2 de S2)	Traffic Classes	Hypothetical A Solution		
			$N_{130\ kN}$ Fatigue	$N_{130\ kN}$ Perm. Def.	$N_{130\ kN}$ Lowest Value
S1/S2	B	T5	9,40E+04	**6,87E+04**	**6,87E+04**

It is noted that for all zones and in both directions the constraint $N_{130\ kN}$ values are reached by the permanently deformed ruin criteria.

In relation to solutions A (hypothetical) and B both applied in the zones of solution B, it is verified, for analysis of Table 5.10, that for all B1, B2 and B3 zones of direction 1 and for all B1 and B2 zones of direction 2, the lower bearing capacity pavement is the corresponding to the overlay with bituminous mixtures reinforcement hypothetical A solution, which demonstrates the advantage of recycling with cement solution in rehabilitation of very damaged pavements.

Table 5.10 – Comparison between bearing capacities

Direction	Zone	Comparison of Constraint Values		
		B Solution	**Hypothetical A Solution**	**$N_{130\ kN}$ Lowest Value**
D1	B1	3,00E+05	**6,87E+04**	**6,87E+04**
	B2	2,83E+05	**6,87E+04**	**6,87E+04**
	B3	4,75E+05	**6,87E+04**	**6,87E+04**
D2	B1	3,74E+05	**6,87E+04**	**6,87E+04**
	B2	6,57E+05	**6,87E+04**	**6,87E+04**

6. Conclusions

This work presents the issue of pavement structural rehabilitation through recycling with cement. Have been reviewed diverse rehabilitation techniques that results in a set of actions with the aim to promote an improvement of the pavement characteristics, in particular the structural ones, facing to new requests provided for a new period of life. The traditional overlay reinforcement of flexible pavements and pavement recycling techniques has been revealed. In this last were referred the main pavement recycling techniques and given special reference to cold *in situ* recycling with cement.

The main conclusion reached in this case study is that the recycling with cement solution led to a greater pavement bearing capacity that would be obtained if had adopted structural overlays.

Before the achieved results, it is pertinent to affirm that the resource to pavements recycling as a structural rehabilitation technique, particularly the cold *in situ* recycling with cement, is viable, and must be taken into account, or at least should be seen as a possible hypothesis in future rehabilitation interventions in pavements which initially are found on a generalized ruin state.

It should also be noted that in the recycling process great environmental and economic benefits occur, not only by the existing deteriorated material that can be reused reducing the disposal of waste materials, but also by reducing the use of new aggregates with consequent reduction of material transportation and the use of natural resources. Throughout this process there is also reduction of energy expenditure. All this leads to a decrease in pavements rehabilitation costs reflected in important economic gains.

By way of example in Figure 6.1 are presented pavement photographs of EN 226 after rehabilitation, where can be certified the good state of the surface course.

 

Figure 6.1 – Pavement of EN 226 after rehabilitation

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