First Test on the CEDEX Test Track

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The test track built by the CEDEX Road Research Center at El Goloso on the outskirts of Madrid is a step forward in the field of lineal tracks because it extends the length of the test area and also increases the load application speed. It was built in 1987 and came into operation in January 1988. A description of the installation and of the first test carried out are given, including the definition of parameters and criteria used and an initial analysis of the results obtained (surface evenness, surface cracking, and deflections). Finally, the general conclusions are given and the consequences of the test for the Spanish Design Catalogue are discussed.

The objective pursued with full-scale test tracks is to provide a tool that enables the passage of vehicles over a pavement to be simulated in an accelerated and controlled way. The lineal test tracks built to date simulate back-and-forth movement. In spite of the advantage of simulating linear movement, they have the drawback of short length (up to 12 m) and low speed (up to 30 km/hr) because the need for the corresponding acceleration and braking areas limits both parameters to a large extent.

The test track built by the Centro de Estudios y Experimentacion de Obras Publicas (CEDEX) at El Goloso on the outskirts of Madrid is a step forward in the field of lineal tracks. The length of the tests area and also the load application speed exceed those existing up to now. The track was built in 1987 and came into operation in January 1988; it is the largest test track with simulated traffic in the world today (Figure 1).

The first test demonstrated the way the installation functioned. Some improvements or modifications of its different elements were developed along with the data collection and analysis systems, so it has taken 3 years to apply 1 million loads. At the end of this period the installation was in perfect condition for constant use and two new vehicles are being built to optimize its performance.

An analysis of the results obtained in the first test is presented. This analysis does not include the results of the instruments placed on the pavement.

DESCRIPTION OF INSTALLATION

Infrastructure

The CEDEX test track consists of two straight stretches joined by two curves. The straight sections and the curves are all approximately 75 m long, giving a total length of track of 304 m in the average transverse path of the load simulator vehicle (Figure 1).

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The curved sections are not used for pavement tests. Their main application is for the testing of surface materials: paints, wearing courses, surface dressings, and so forth. The curves have therefore been given a pavement section that has a reinforced-concrete base course and a lean concrete subbase designed to withstand an indefinite number of tests without deteriorating. In this way it will be necessary to replace only the wearing course in successive tests.

Without the transition zones between the straight and curved sections, there is 67 m left for pavement structure tests in each straight stretch. Because 20 m is considered the minimum length for a test to have any significance, the track's testing possibilities include six sections, each approximately 20 m long (Figure 1).

Although the pavement on the curves rests on the ground, the test sections on the straight stretches are constructed inside U-shaped watertight boxes made of reinforced concrete. The purpose of this system is to isolate the behavior of the pavement from that of the surrounding ground, and it allows the subgrade to be flooded for testing under different groundwater conditions.

The boxes are 2.60 m deep, so that embankments can be built at least 1.25 m high. The 8-m width allows the use of conventional machinery and the usual road-building procedures.

On the inside perimeter of the oval band, a reinforced-concrete rail beam has been installed to serve as a guide for the traffic simulator vehicle and to permit total control of the load path. This rail beam is anchored in the straight stretches

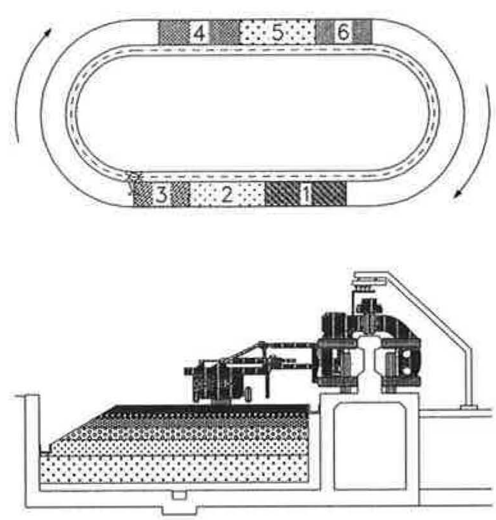

FIGURE 1 Plan and cross section of test track.

to underground, accessible galleries that house the connections of the permanent measuring equipment for each type of test (Figure 1).

Finally, a device has been installed that enables the test stretches to be covered, if desired; water sprinklers to simulate rain or any other mechanism to control the climatic conditions can be installed under the cover (Figure 2).

Traffic Simulator Vehicle

The traffic simulator has two distinct parts: the guiding cart and the load cart (Figure 3). The latter applies the gravity stress. Its total weight (including the ballast) is 6.5 tonnes, the equivalent of 13 tonnes/axle, which is the maximum permitted for single axles in Spain. The load is applied by a pair of twin wheels with conventional tires and an inflated pressure of 8.5 kg/cm^2. This cart is self-propelling and therefore generates the movement of the whole unit. The motor is electric and runs on power from the overhead rail above the rail beam.

The vehicle has tires and suspension similar to those used by heavy road vehicles. It is possible to apply single or twin wheels. The design speed during the tests is 30 to 45 km/hr and the maximum speed is 50 km/hr.

The load cart can travel crosswise because of the action of a pneumatic jack incorporated in the vehicle. The maximum shift is ± 400 mm, which produces a tread strip 1.3 m wide. The crosswise change in position occurs automatically by means of a centralized system that programs the control of the vehicle. The distribution of the passes follows a standard curve responding to actual distributions measured on roads.

The guiding cart runs along the concrete rail beam and is coupled to the load cart. Both are joined by articulations, allowing the load vehicle to move in a normal plane in the direction of movement. The articulation consists of two bogies mounted on the ends of a metal beam with two turning crown gears, which enable it to adapt perfectly to changes in path in the curved sections.

There are two cupboards on the guiding cart beam, which contain the alarms and control mechanisms; the compressed air generating equipment, which keeps the load wheel pressure constant and serves the pneumatic suspension and braking when the driving axle needs them; and the power switch trolleys fitted with oscillation-absorbing devices.

The control center, in a small building inside the loops of the track, automatically controls the movement of the vehicle (speed, transverse path change, etc.) and the orders communicated by radio. In addition, countless alarm sensors have been fitted into the vehicle itself to detect any anomaly that may occur during the test. If any problem occurs, the vehicle stops and the time and cause of the breakdown are stored in the control computer. This enables automatic unmanned functioning to take place with no risk to the installation and the track to be used continuously.

FIGURE 2 General view of test track.

FIGURE 3 Vehicle traffic simulator.

FIRST TEST

Pavement Section

The sections tested cover two different treatments for the same traffic situation (50 to 200 heavy vehicles per day) and subgrade [California bearings ratio (CBR) between 10 and 20] in accordance with Spanish Design Standards 6.1. and 6.2-IC. One treatment uses a large thickness of granular material (25 cm of crushed rock plus 25 cm of gravel) under an average 15-cm layer of asphalt mix. The other uses a much thinner layer of granular material (25 cm of crushed rock) and an increased asphalt layer (18 cm). In the sections located in the remaining four substretches, the thickness and type of lower layers remained the same, with the only change being in the thickness of the asphalt layer. The sections used in the first track test are shown in Figure 4.

The objectives of the test were to reach conclusions on

1. Which of the two standard sections of the Design Catalogue provided a better service life, and
2. The relative decrease in service life of these sections for each centimeter less of asphalt layer.

Materials

At the bottom of the embankments and in the side walls of each box, a drainage material was installed, separated from the ground or pavement by a geotextile. The scale of thicknesses was introduced into the ground at the top level of the subgrade. The pavement was laid on a subgrade with a CBR

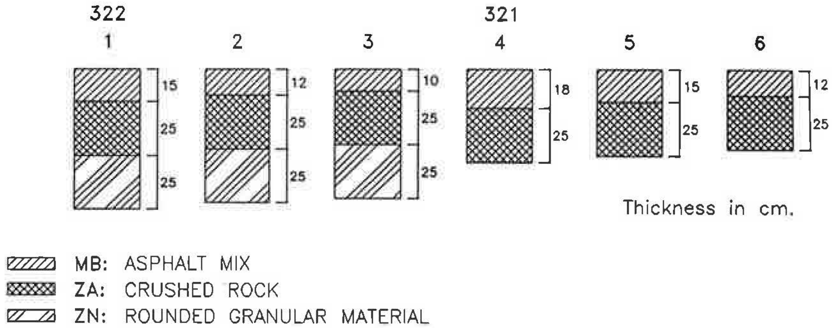

FIGURE 4 Sections tested.

of 10. Table 1 gives the characteristics of the gravel and crushed limestone rock used.

The asphalt mixes used were of two types following customary practice. The 5-cm-thick wearing course contained coarse siliceous aggregate with a maximum grade of 12 mm. The intermediate layer contained coarse limestone aggregate with a 20-mm maximum particle size. The thickness of this layer varied in the different stretches between 5 and 13 cm and was laid in two passes in the thickest test sections.

The characteristics of the asphalt mixes are shown in Table 2. These results correspond to samples extracted from the pavement. Table 3 shows the rheological properties, defined by the dynamic modulus and the fatigue law. Simple dynamic compression tests on the set of asphalt mixes determined the dynamic modulus, and samples from the binder layer gave the law of fatigue in stress-controlled four-point bending tests.

PARAMETERS AND CRITERIA IN THE ANALYSIS

The comparison of the different sections was made by determining the number of applied loads that each one could withstand before failure.

The deterioration considered was

- Loss of surface evenness, and
- Loss of bearing capacity, assessed by
  - Cracking of the pavement surface and
  - Progression of the deflections.

The loss of surface evenness was understood to be the diversion of the surface throughout the test compared with that of the wearing course when the road was completed. The deterioration was evaluated directly by obtaining transverse profiles of the different stretches involved using a transverse profilograph.

The parameters taken into account in the analysis were rut depth in the center of the trafficked area, an absolute value or the difference between two consecutive evaluations (PRC or DRC in Spanish), and maximum rut depth of the trafficked area, an absolute value or the difference (PRM and DR). PRC and PRM were used to calculate the deterioration and its evolution during the test, and PRM and DR to evaluate the speed of this deterioration.

The measuring profiles were placed at equal intervals 1 m apart. The measurements taken were digitized to facilitate

TABLE 1 Characteristics of Granular Materials

	ROUNDED GRANULAR MATERIAL		CRUSHED ROCK		SUBGRADE	
SIEVE SIZE - % PASSING						
. 25 mm	94,5		100,0		100,0	
. 12,5 mm	74,5		67,0		100,0	
. 5 mm	55,5		51,5		100,0	
. 200 μm	12,0		13,5		39,0	
. 80 μm	7,5		9,5		26,4	
MODIFIED PROCTOR						
. Density (t/m^3)	2,18		2,23		1,95*	
. Optimum moisture (%)	5,8		7,0		10,5 *	
RESILIENT MODULUS (MPa) ** $E = K_1 \left(\frac{I_1}{\sigma_0}\right)^{K_2}$	K_1 (MPa)	K_2	K_1 (MPa)	K_2	K_1 (MPa)	K_2
	70,51	0,19	13,47	0,39	11,76	0,34

* Normal Proctor
** $I_1 = \sigma_1 + 2\sigma_3$
$\sigma_0 = 1$ KPa

TABLE 2 Composition of the Mix (from Samples)

MIX TYPE	COMPOSITION							BITUMEN	
	BY WEIGHT				BY VOLUME				
	STONE (%)	SAND (%)	FILLER (%)	BITUMEN (%)	V_A (%)	V_b (%)	V (%)	PEN	T_{RB} ºC
S-12 (wearing course)	44,0	49,0	7,0	4,7	81,0	10,2	8,8	39	53
S-20 (binder layer)	47,0	46,5	6,5	4,5	82,9	9,9	7,2	37	53

TABLE 3 Complex Moduli and Fatigue Characteristics of the Mix (Extracted Cores)

DYNAMIC MODULUS (MPa) (f = 10 Hz)				FATIGUE LAW (f = 10 Hz)
T = 40ºC	T = 25ºC	T = 0ºC	T = -15ºC	
1.100	6.000	13.000	18.000	$\log \epsilon = -2,5 - 0,23 \log N$

computer processing and once the above-mentioned parameters were obtained, statistical analyses were made on individual sections and evolution curves were drawn up.

The following were considered to be criteria of generalized failure:

- PRC parameter:
  - Average PRC of the substretch ≥ 15 mm,
  - 20 percent or more of the profiles measured having a PRC ≥ 25 mm.
- PRM parameter:
  - Average PRM of the substretch ≥ 20 mm,
  - 20 percent or more of the profiles measured having a PRM ≥ 30 mm.

The appearance of surface cracking was another of the types of deterioration considered in the comparison of test sections. Data collection began with visual inspection. Each inspection surveyed the test sections and noted any cracks obvious to the naked eye when the operator was upright. Once the cracks were located they were marked with paint for subsequent identification and photographed with a superimposed metal grid to make measurement easier. The data were subsequently fed into the utilization of results program by means of digitization. This program then provided evaluation indexes automatically, calculating the representative parameters.

The analysis took into consideration the following parameters:

- Percentage length of cracked area (LF in Spanish),
- Percentage extent of cracked area (AF in Spanish), and
- Gravity of cracking, testing the cracked area with different weights as a function of the type of crack (GF in Spanish).

The failure criteria adopted were LF ≥ 50 percent, AF ≥ 40 percent, and GF ≥ 60.

Two pieces of equipment were used to evaluate the deflections: a KUAB double mass falling-weight deflectometer (FWD) with a maximum equivalent load of 50 kN and the Benkelman beam with a load of 130 kN, as standardized in Spain. The results will be presented in another paper (*1*).

ANALYSIS OF RESULTS

Surface Evenness

The data corresponding to the evolution of the rut depth (PRC) in average values per section according to the number of load cycles involved are shown in Figure 5. Table 4 gives corresponding deformation speeds (DRC/number of cycles).

After 1,000,000 test cycles, the order of the sections, from smaller to larger degree of deformation, was 1, 2, 3, 4, 5, and 6. The corresponding average rut depths were 12.4, 13.4, 18.4 (780,000 cycles), 26.6, 14.7 (500,000 cycles), and 15.8 (500,000

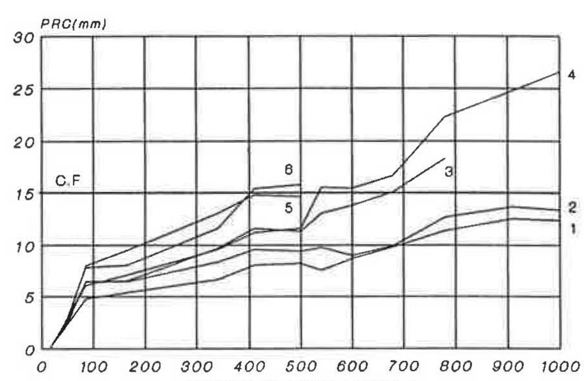

FIGURE 5 Evolution of rut depth (PRC) with loading [CF = failure (maximum average PRC)].

TABLE 4 Evolution of Velocity of Formation of Rut Depth (VDRC, in 10^{-5} mm/cycle) with Loading (S = section; IC = number of loads × 1,000)

S \ IC	15-50	50-85	85-165	165-340	340-410	410-500
	VDRC					
1	6,9	6,6	4,8	0,7	1,5	0,3
2	8,6	10,0	6,4	1,1	1,7	0,0
3	7,1	11,3	6,3	1,9	2,8	0,0
4	8,5	9,9	6,8	1,4	2,3	0,5
5	8,0	14,9	9,7	2,0	3,1	0,0
6	9,0	13,4	8,0	2,0	5,4	0,4

S \ IC	500-540	540-600	600-680	680-780	780-910	910-1000
	VDRC					
1	0,0	1,2	1,6	1,6	0,7	0,0
2	0,8	0,0	1,1	2,8	1,1	0,0
3	5,0	1,2	1,6	2,7	-	-
4	8,4	2,8	1,5	5,0	2,5	2,5
5	-	-	-	-	-	-
6	-	-	-	-	-	-

cycles). Sections 5, 6, and 3 contained deformations so great at certain points that they had to be repaired at 500,000, 600,000, and 780,000 cycles, respectively.

Sections 1 and 2 did not reach failure by any of the criteria established. Section 3 failed according to all the criteria at 700,000 cycles. Section 4 failed between 540,000 cycles and 700,000 cycles, depending on the criteria. Sections 5 and 6 failed at 550,000 and 500,000 cycles, respectively.

In the initial cycles (0 to 165,000), the deformation speed (Table 4) ranged between 6×10^{-5} and 15×10^{-5} mm/cycle. Between 165,000 and 1,000,000 cycles, deformation speeds were lower, with some exceptions. Standard Section 1 had a lower deformation speed than Section 4. The speed values were arranged in each group of sections according to the bearing capacity (thickness of asphalt) of the sections.

Figure 6 gives a month-by-month summary of the temperature and rainfall data recorded in the track during the test, and Figure 7 gives an example of the evolution of rut depth.

Three periods had very high temperatures: 50,000 to 85,000 cycles and 85,000 to 165,000 cycles, corresponding to the first summer, and 680,000 to 780,000 cycles, corresponding to the third summer (during the second summer the vehicle was stopped for modifications). The first two periods correspond to the greatest deformation speed, and the third one, although it has relatively low values, has greater speeds than the preceding and next cycles (2.2×10^{-5} to 4.8×10^{-5} mm/cycle versus 1.3×10^{-5} to 2.1×10^{-5} mm/cycle). The large deformation that occurred in the initial cycles (with low and medium temperatures) is attributed mainly to postcompaction. Between 50,000 and 165,000 cycles postcompaction and high-temperature creep were added.

There were three rainy periods. The first two periods (15,000 to 50,000 cycles and 50,000 to 85,000 cycles) correspond to the first phase of large deformation speed, so it is impossible to separate the effect of the rain from the other factors. The third rainy period (500,000 to 540,000 cycles) was during the period that had low speeds, but again an increase in the speed with respect to the surrounding cycles is noted (2×10^{-5} to 10×10^{-5} mm/cycle versus 0.4×10^{-5} to 3.5×10^{-5} mm/cycle). The influence of the rain is greater in pavements with cracks at the surface.

Table 5 presents data from cores of the zones in the trafficked area (taken as initial values) and from the trafficked area in the different sections. Comparing Table 5 with Figure 5 it can be seen that an important part of the deformation is due to settlement of the noncohesive materials. Asphalt mixes had an average initial 8 percent of voids (range between 7 and 11 percent) and underwent recompaction up to average

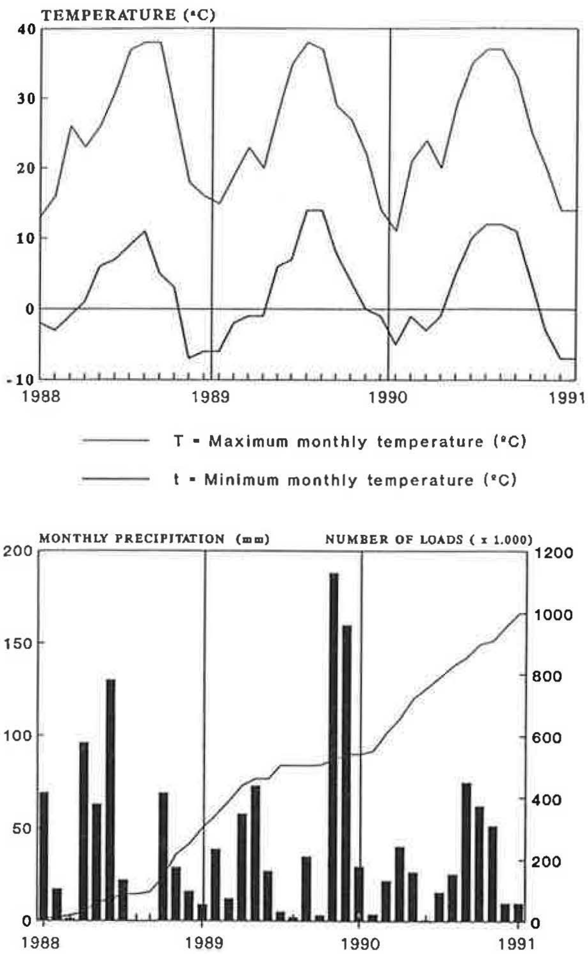

FIGURE 6 Air temperature and precipitation history of test area.

values of 4 percent (curves), 5 percent (Sections 1, 2, and 3 with 50 cm of granular layers), and 7 percent (Sections 4, 5, and 6 with 25 cm of granular layers). The end values did not depend on the initial voids. The corresponding percentages of the Marshall reference density were 101, 100, and 98 percent.

The information obtained with the PRC parameter (rut depth in the center of the two layers) was very similar to that obtained with the PRM parameter (maximum rut depth), with the maximum differences between the values of the parameters being 1 to 2 mm.

The failure criterion according to the maximum average value was more restrictive than the criterion by the measurements over a maximum value, and the PRC failure criteria were more restrictive than the PRM criteria.

Surface Cracking

Figure 8 is a summary graph of the evolution of cracking, with the number of cycles for one part of a section, which is a direct output of the computer crack-processing program.

The wear initially manifested itself in the form of transverse cracks. This progressed to longitudinal cracks at the same time that the transverse cracks grew longer, forming the characteristic alligator crazing at the end.

As a function of the analysis of the samples taken from the pavement confirmed by a stress-strain analysis, the most likely hypothesis is that the cracking in the asphalt went through the following process:

- Loss of adhesion of the different asphalt layers involved,
- Failure through fatigue in the top layer,
- Failure of the intermediate layer, and
- Failure of the bottom layer.

For the sections tested, Figure 9 gives the deterioration curves formed by the LF parameter. As can be seen, the beginning of the deterioration did not appear continuously. Once the deterioration had started, the cracking speed was greater in damp and cold weather than in hot weather.

At the end of 1,000,000 cycles the sections were arranged in ascending order of cracking as follows: 1, 2, 4, 5, 3, and 6. This order pointed to a better durability of standard Section 1 than 4. In addition, standard Section 4 was more sensitive than Section 1 to cracking under traffic when reductions of thickness existed.

Within each standard section the sections were arranged in terms of surface cracking evolution according to the total thickness of the mix (the lower the thickness of the mix, the greater the cracking).

Section 1 had not reached failure point after 1,000,000 cycles and did not present any deterioration at the end of the test. In Section 2, 39 percent of the length and 22 percent of the surface area showed cracking. Section 3 had reached the failure point at 670,000 cycles (average of LF and AF). Section 4 reached failure at the end of the test, and Sections 5 and 6 after 500,000 cycles.

GENERAL CONCLUSIONS

Section 322 of the Spanish Design Catalogue (Section 1 in the test), consisting of 15 cm of asphalt mix over 50 cm of granular material (25 cm of crushed rock and 25 cm of gravel), constitutes a better technical solution than Section 321 (Section 4 in the test), consisting of 18 cm of asphalt mix over 25 cm of crushed rock. This conclusion is based on two facts, brought out in the test:

- Section 322 withstood 1,000,000 load cycles without showing signs of any significant wear, whereas Section 321 showed 44.2 percent cracked length at the same point.
- Section 322 was less sensitive to any decreased thickness of the asphalt layers. The sections tested that had the same bottom layers as the preceding ones but 2 cm less pavement thickness showed very different behavior. With some of the criteria used, the one similar to Section 321 reached the failure point at 500,000 load cycles. The one analogous to Section 322 has not failed with 1,000,000 load applications even with 39 percent of cracked length.

Section 321 is not suitable for the working conditions seen in Spanish Design Standards 6.1 and 6.2-IC. This test revealed at least one situation, with a subgrade and climatic condition

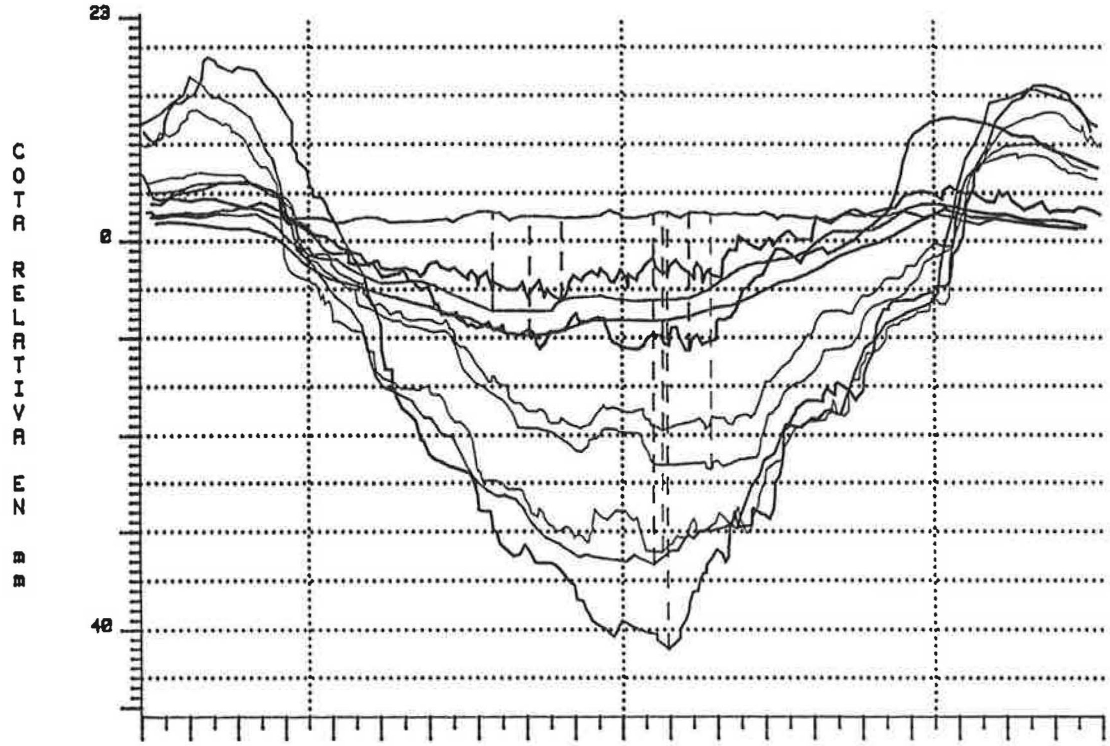

FIGURE 7 Example of evolution of rut depth.

very common for Spanish roads, in which the section did not fulfill the hypothesis of the catalogue. This section reached failure with 600,000 loads under rutting criteria and with 1,000,000 loads under cracking criteria. Consequently, in climatic conditions similar to those of Madrid (Figure 6), there is a great probability of its durability being between 12 and 15 years for a high level of traffic within the variation interval seen for its type in the road catalogue. For an average level of traffic within this interval, it would, however, meet the design conditions. Nevertheless, with 500,000 load applications, the cracked length in the test was over 15 percent. In practice this means that this structural section might require frequent maintenance and rehabilitation.

In the sections with 25 cm of granular base course layer placed directly on the subgrade, an asphalt mix thickness equal to or less than 18 cm can be considered critical since any small reductions in the thickness of the layers or in the quality of the materials cause rapid failure of the pavement.

TABLE 5 Final Characteristics of the Asphalt Mix

SECTION	INITIAL THICKNESS (mm)	DECREASED THICKNESS (mm)
1	155	6,1
2	125	7,0
3	105	6,6
4	160	6,0
5	145	13,0
6	120	10,0
North Curve	75	2,1
South Curve	75	5,1

In these sections exceeding the thickness by going from 15 to 18 cm meant a 20 percent increase in service life with regard to rutting or multiplied it by 2.5 with regard to cracking.

In the sections with 50 cm of granular layers, a pavement thickness equal to or less than 10 cm can be considered critical for the same reasons as those above. Because two of the sections did not reach failure at the end of the test, final figures cannot be given regarding the increase in service life when the 10-cm thickness is increased. What can be stated is that the service life of the section with a 12-cm thickness has proved to be at least 50 percent longer than that of the 10-cm-thick pavement.

The section composed of a 10.5-cm-thick asphalt layer and 50 cm of granular layers is more or less equivalent to one consisting of 16 cm of asphalt layer plus 25 cm of granular materials, since both withstood some 450,000 load cycles (12 years under average traffic and 8 with heavy traffic within the limits set). In other words, in this type of pavement, 1 cm of asphalt mix is the equivalent of 4 cm of gravel. These equivalences have been drawn up by composing the failure criteria of loss of evenness and cracking. If each type of failure is considered individually, the first section is slightly better than the second regarding evenness and also slightly worse in cracking, with equivalences of 1:3.3 and 1:4.5, respectively.

The relationship between the sections with noncritical thicknesses (Sections 1 and 2) and Sections 4, 5, and 6 is different, and it can be stated that an increase of 4 cm of granular material in a pavement creates a better service life than an increase of 1 cm in the asphalt layer.

As a consequence of the foregoing and for the reasons already given, it has been proposed, on the basis of the test results,

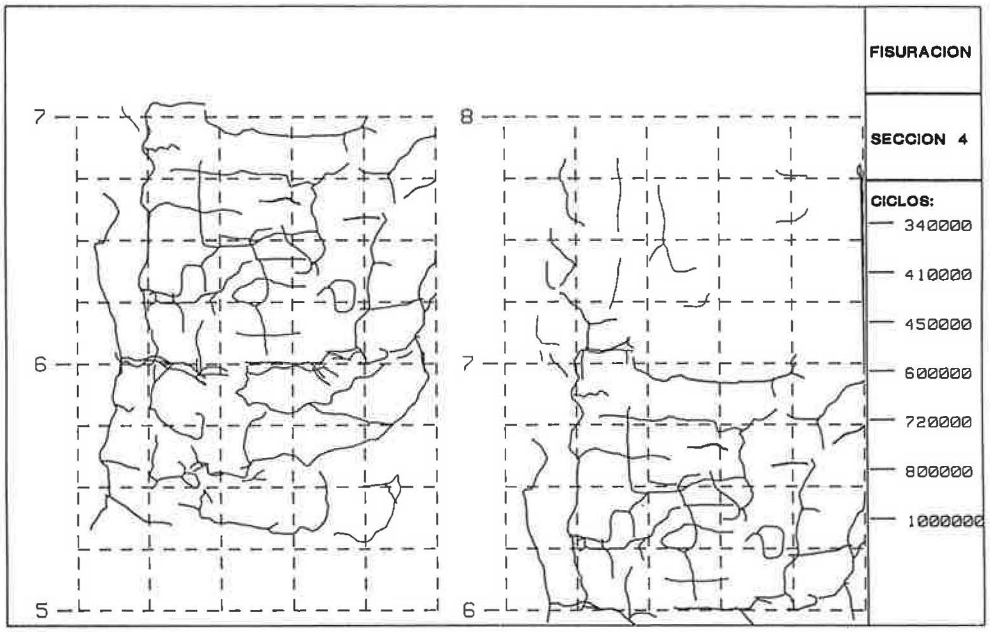

FIGURE 8 Summary of cracking in Section 4.

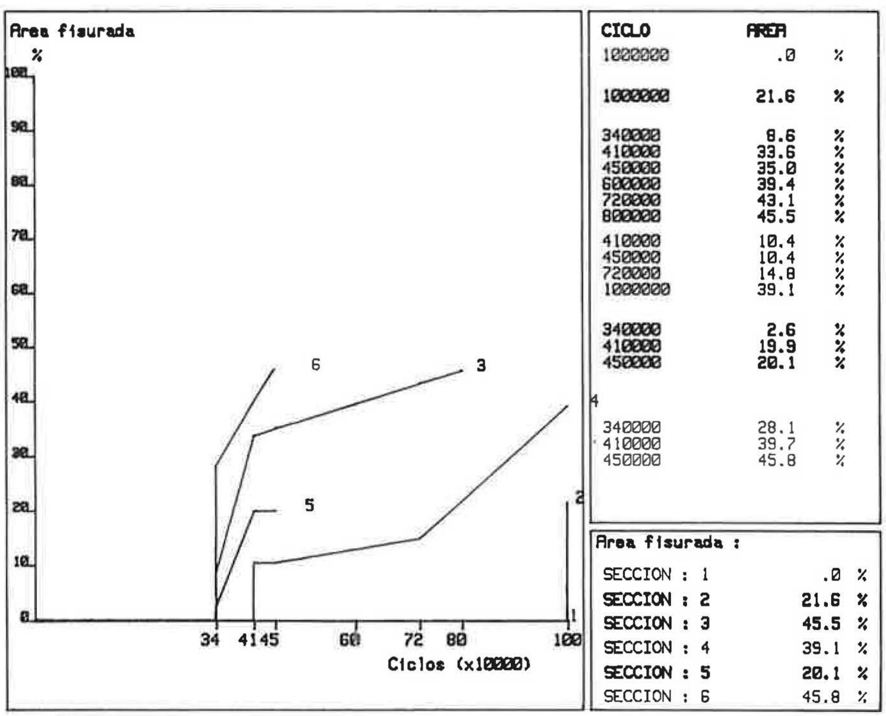

FIGURE 9 Evolution of cracking (LF) with loading.

1. To eliminate Section 321 from the Spanish Design Catalogue, since its use could give rise to excessive maintenance costs or cause premature total failure, and

2. To keep Section 322—even though the asphalt thickness seems to be higher than necessary to bear the design traffic, a reduction in the thickness could lead to critical situations.

As regards the installation, the main conclusion reached has been that the installation for the testing of pavements created and built by CEDEX is perfectly valid for the purpose for which it was designed and has numerous advantages over the test tracks in existence.

ACKNOWLEDGMENT

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REFERENCE

1. A. Ruiz, R. Romero, and A. Gonzalez. Analysis of Deflections on a Test Track. Presented at Symposium on Capabilities and Limitations of Nondestructive Testing and Backcalculation, Nashville, Tenn., Aug. 1991.