TWELVE YEARS OF ACCELERATED PAVEMENT TESTING IN SOUTHERN BRAZIL; CHALLENGES, ACHIEVEMENTS AND LESSONS LEARNED

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ABSTRACT
Since 1996 a linear traffic simulator has been loading full-scale test sections in an accelerated pavement testing facility built at the Federal University of Rio Grande do Sul, in Porto Alegre, Brazil. The facility comprising the traffic simulator, test sections and a control centre, resulted from a cooperation agreement between the University and Rio Grande do Sul State Roads Department. From 1996 to 1999 researches were concentrated on unbound aggregates (conventional and alternative ones) for low volume roads. From 2000 to 2006, the studies mainly focused maintenance treatments for asphalt pavements. Special attention was given to techniques for retarding cracking reflection in overlays. The performances of test sections including paving fabrics, thin overlays (surface treatments) with polymer-modified binders and asphalt rubber overlays were compared to those of HMA with conventional binder. Besides surface measurements resulting from deflections, rutting and cracking surveys, special attention was given to pavement instrumentation (strain and stress cells). The traffic simulator was also used in a research carried out with the purpose of understanding the behaviour of PVC pipes used in culverts. All in all, more than 2,500,000 axle loads ranging from 82 kN to 130 kN were applied to fifteen different pavements. Some of the main findings are listed below.

• Load factors were computed and a design equation for low volume roads, based on APT results and reliability analysis, was proposed.
• Improvements in the specification for dry-bound macadam were suggested.
• The influence of soil suction in subgrades resilient modulus, thus in pavements performance, was quantified.
• Temperatures distribution across the thickness of asphalt mixes was modelled.
• Moduli of soil reaction for flexible pipes design were computed.
• The benefits of including paving fabrics between cracked AC pavements and new AC overlays were quantified.
• The efficiency of asphalt rubber mixes overlays was compared to that of conventional HMA (AR overlay outperformed by a factor of 4.84).

This paper summarizes achievements and lessons learned in twelve years of activities carried out by University researchers and technicians of the State Roads Department, with financial help of Brazilian agencies for research support and private companies.

Keywords: APT – unbound aggregates – asphalt mixes – maintenance treatment – modelling – pavement performance
INTRODUCTION
On May 10th, almost five months before APT2008 opening, some academic researchers and pavement engineers celebrated twelve years of accelerated pavement testing in Southern Brazil.

We start this article in an unusual way, somehow defying the principles of scientific publication. This has been intentionally done and here the reader will not only find some relevant technical and scientific discoveries but also perceive the enjoyment of those who have succeeded keeping the facility working and, perhaps, at the end of the presentation, he will find himself sharing their concerns and hopes.

Forgive us the lack of formality; we believe that this 3rd Conference on APT is a moment to celebrate achievements, to share lessons learned and gain strength to face the challenges that wait for us.

In this article we take a look at the activities carried out at the Pavement Testing Facility on the Federal University of Rio Grande do Sul (UFRGS) Campus, in Porto Allegre (Rio Grande do Sul capital), since the very beginning to present days. The main results of some studies on thin pavements and on strategies to delay crack reflection are summarized. The importance of including reliability concepts and pavement instrumentation when analysing APT results is emphasized. Some parallel studies, such as the effects of soil suction on in situ subgrades resilient modulus, the modelling of temperatures distribution across the thickness of asphalt layers and the computation of soil reaction for flexible pipes design, are reviewed. Last, but not least, we propose a discussion on some challenges that ours and other APT facilities will have to face in order to keep helping pavement technology advancing.

PREVIOUS ACTIVITIES ON APT IN BRAZIL
In Brazil, APT activities have been carried out since 1982, when a radial traffic simulator started loading test sections, in a facility run by the Pavement Research Institute (IPR) of the late National Roads Department (DNER), in Rio de Janeiro. Bernucci et al. (2007) overviewed the main studies carried out in that facility. [1]

THE BIRTH AND FIRST YEARS OF THE APT FACILITY IN UFRGS CAMPUS
In the early 90s, the Federal University of Rio Grande do Sul and the Roads Department (DAER/RS) of that Brazilian state celebrated a cooperation agreement with the purpose of studying weathered volcanic rock as materials for low-volume roads paving.

From 1992 to 2001, a comprehensive study was carried out, including:

a) Geological-geotechnical characterization and laboratory studies of ten deposits of weathered rocks located in different regions of the state;

b) The development of a qualifying criterion for weathered volcanic rocks;

c) The design and construction of a linear traffic simulator;

d) The development of a design equation for thin pavements built with weathered basalts, based on accelerated pavement testing; and

e) The construction and monitoring of an in-service road section, considering APT findings.
The UFRGS-DAER Traffic Simulator
The UFRGS-DAER/RS Traffic Simulator shown in figure 1 was designed by researchers of the UFRGS Mechanical Engineering Department, and constructed, from 1992 to 1994, by DAER/RS personnel. The equipment, 15.0 m long, 4.3 m high and 2.5 m wide, weighs around 220 kN. Bias ply tires, size designation 9.00 x 20.00, are mounted on a dual wheel on the loading carriage. The system is arranged with computer control of travel speed and indexing (where the precise path of the tires can be varied as much as ± 0.4 m from centreline for any given number of passes). Loads ranging from 41 kN (half of Brazilian standard axle load) to 65 kN are hydraulically applied. Tire pressures vary from 0.56 MPa to 0.73 MPa, accordingly. Wheel load and tire pressure are controlled to a tolerance ±2%. The test wheels travel at 6 km/h over 7 m of pavement. Loads are applied in one direction and normally distributed about a wheel path.

Figure 1 The UFRGS/DAER traffic simulator

The Research on Thin Pavements with Weathered Basalts
In the first research, carried out in cooperation with DAER/RS from May 1996 to July 1999, the traffic simulator loaded eight test sections, 20 m long and 3.5 m wide, with weathered basalt bases and/or sub-bases and 25 mm thick two-layer surface treatment with sealing coat. Lateritic red clay, similar to basalt soils, classified as A-7-6(7), according to AASHTO system, was used as subgrade in all sections. Table 1 presents the nature and thickness of the test sections granular layers.
Table 1 Test sections granular layers

<table>
<thead>
<tr>
<th>Test section</th>
<th>Weathered basalt site</th>
<th>W. basalt thickness (cm)</th>
<th>Sound rock thickness (cm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>01</td>
<td>F.1</td>
<td>16</td>
<td>-</td>
</tr>
<tr>
<td>02</td>
<td>E.1</td>
<td>16</td>
<td>-</td>
</tr>
<tr>
<td>03</td>
<td>F.1</td>
<td>21</td>
<td>-</td>
</tr>
<tr>
<td>04</td>
<td>E.1</td>
<td>21</td>
<td>-</td>
</tr>
<tr>
<td>05</td>
<td>E.1</td>
<td>16</td>
<td>12</td>
</tr>
<tr>
<td>07</td>
<td>F.1</td>
<td>32</td>
<td>-</td>
</tr>
<tr>
<td>08</td>
<td>E.1</td>
<td>32</td>
<td>-</td>
</tr>
<tr>
<td>09</td>
<td>E.1</td>
<td>16</td>
<td>18</td>
</tr>
</tbody>
</table>

After extraction with earth-moving machines, the weathered basalts were crushed in order to obtain coarse aggregates (5-10 cm), fine aggregates (1.9-0.48 cm) and dust. The fine aggregates and the dust were mixed in equal proportions to constitute the filling material of the dry-bound macadam layer. Details of test sections construction were presented by Núñez et al. [2]

From May 1996 to April 1998 the traffic simulator applied more than 277,000 loading cycles to the six test sections with weathered basalt bases. Latter, some 230,000 loading cycles were applied to test sections 05 and 09, with bases of sound rock and subbases built with weathered basalts.

On each section two 75cm-wide wheelpaths (designated as sectors), separated by a 70cm-wide untrafficked lane, were tested. On each sector a given axle load (from 82 to 130kN) was applied, following an experimental design. [3] Four numbers identified sectors; two corresponding to the test section (structure) and two others indicating the axle load in metric tons.

**How pavement performed**

Rutting, which was the major failure cause in all tested pavements, grew exponentially with the repetitions of axle loads. Two distinct phases of rut developments were observed: a rapid growth at the beginning of the test, and after that a gradual rate of growth. The rate of rutting showed to depend on axle load and on pavement structure (thickness and strength of weathered basalt). In weaker pavements (sections 01, 02, 04), a third phase, where rutting growths exponentially (approaching failure), was identified.

Post-mortem evaluation in two test sections revealed that the weathered basalt dry macadam layer contributed the most (>60%) to rutting. Two distress mechanisms affected weathered basalts: crushing in weaker aggregates and post-compaction and shear displacement in the stronger ones. [4]

Sections 01, 02, 03 and 04 developed extensive longitudinal cracking initiated from the top of the surface treatment, due to structure bending because of the consolidation in the granular layers. On the other hand, in sections 07 and 08, where plastic strains were much lower than those corresponding to first crack appearance in weaker sections, no longitudinal cracks were observed in those stronger pavements.
Concerning to deflections, a common pattern was not identified. In the weaker structures high axle loads caused a sharp increase of deflections (up to 0.77 mm in sector 0113 and to 1.04 mm in sector 0413). This may be attributed to water income due to the extensive longitudinal cracking observed in both sectors, and, sometimes to aggregate crushing and consequent alteration in grain size distribution.

In the stronger structures (sections 07 and 08), pavement deflections remained stable, in spite of the different climatic conditions at which were measured (spring of 96 to winter of 97). Deflections average values oscillated around 0.50 mm (±10%), regardless pavements low structural numbers. Such a good behaviour was attributed to the subgrade, whose back-calculated modulus averaged 320 MPa (coefficient of variation of 23%). In some test sections, the subgrade was stiffer than the pavement; a fact not surprising due to the excellent elastic behaviour of tropical and sub-tropical lateritic soils.

A broad discussion on factors affecting the elastic behaviour of thin pavements was presented by Núñez et al. (2002). [5]

Lessons we have learned
Concerning dry-bound macadam construction procedures, we found out that DAER/RS specification ES-P-07/91 is globally adequate; nonetheless, the following modifications were suggested:

a. To eliminate surface wetting after the macadam layer compaction; a practice included in the specification that may be harmful to macadam layers built over clayey soils.

b. To perform compaction control of granular layers measuring Benkelman, limiting the allowable deflection (under the 82-kN axle load) to 0.8 mm. If higher deflections levels are observed, it is mandatory the use pneumatic rollers in addition to smooth vibrating one.

Besides, the performance of tested pavement sections validated the acceptance criterion for weathered volcanic rock, proposed by Arnold (1993) [6]. The point load test may be used to qualify weathered volcanic rocks for paving purposes, defining 0.7 as the minimum value for the ratio of soaked to dry strength and 3.5 MPa for the point load strength index after soaking.

Research pays off
Studies carried out by DAER/RS have shown that the cost of aggregates for bases of thin pavements may be lowered in 60% if weathered basalts in a dry macadam layer are used instead of traditional densely graded crushed sound rock. That is, 2.2 m$^3$ of dry macadam weathered basalt base costs as much as 1 m$^3$ of compacted crushed sound rock. This remarkable economy is due to the facts that no explosives are used to quarry weathered basalts, and that crushing weathered basalts is very easy.

Moreover, the analysis did not take into account transport costs. Considering that weathered volcanic rocks are found in more than 40% of Rio Grande do Sul territory, it is reasonable to assume that the global cost reduction may be even higher than 60%.

It may argued that the cost analysis is misleading once the performances of pavements with weathered basalts or densely graded crushed sound rock bases are not expected to be the same, however it is worth to compare the performances of three sections tested with the traffic simulator applying axle loads of 120 kN.
As shown in Table 1, test sections 7 and 8 presented total aggregate thickness of 32 cm, consisting exclusively of dry macadam weathered basalt from sites F.1 and E.1, respectively. Conversely, test section 5 presented a 12 cm thick densely graded crushed rock base overlying a 16 cm thick weathered basalt (site E.1) subbase. According to the cost analysis presented, aggregate cost in TS5 is 44% higher than in TS7 and TS8.

Average rut depth evolution in the test sections is presented in Figure 2. It may be noticed that:

a) rutting grew exponentially with the repetitions of axle loads. The initial rut growth is sharper in test sections 7 and 8 (6 mm after 700 loading cycles) than in test section 5, due to the difficulty in properly compacting the macadam layers.

b) the E.1 weathered basalt is less strong than that extracted from site F.1, and rutting grew much faster in TS8 than in TS7.

c) there is a remarkable similarity in rut depth evolutions of TS7 and TS5, suggesting that, if only plastic behaviour was considered, 12 cm of densely crushed rock base would be equivalent to 16 cm of dry macadam weathered basalt (site F.1).

![Figure 2 Rut depth evolution in thin pavements (axle load = 120 kN)](image)

A quite different conclusion arises when elastic behaviour is analysed. Figure 3 shows the evolution of characteristic deflection (average plus one standard deviation) in the three test sections. Deflections were measured with Benkelman beam, applying Brazilian standard axle load (82 kN).

It may be seen that:

a) while in pavements with weathered basalt bases deflections scarcely varied 10% during trafficking, in TS5 deflections suffered a sharp increase (nearly 50%) just after the test had begun;

b) though in both TS7 and TS5 characteristic deflections before trafficking were equal (0.44 mm), after 42,000 passes of the 120 kN axle load, those values were 0.43 mm and 0.65 mm, respectively.

c) deflections development in both sections with weathered basalt bases is not very different, but it must be observed that only a few measurements of deflections were done while trafficking TS8.

Thus, it may be concluded that dry macadam weathered basalt bases presented a much better elastic behaviour than the traditional densely graded crushed sound rock base. This may be attributed to the fact that macadam bases work as draining layers,
while densely graded crushed rock tends to hold water that, by any means, gets into the pavement.

![Figure 3 Deflection evolutions in thin pavements (measured under 82 kN axle load)](image)

Strong importance is given to aggregates elastic behaviour since most of Brazilian pavements fail because of fatigue cracking of asphalt wearing courses. Fatigue cracks were not observed in the considered test sections, partly because deflections values were not too high and partly because of the wearing course nature (surface treatment).

So, if care is taken during site characterization, rock sampling and testing and pavement construction, weathered basalts may be used as alternative aggregates for low-volume roads bases with remarkable economical advantages.

**Optimising APT Results Using Reliability Concepts**

Although rut depth of 25 mm was adopted as terminal criterion, traffic on each sector was meant to proceed until an average rut depth not less than 20 mm had developed. The rationale for terminating the testing at that point was that (a) DAER/RS judged that an in-service pavement with similar distress type and extent would be repaired; (b) in the thicker pavements (sections 07 and 08) the rate of rutting depth had decreased dramatically; and (3) it was decided that it would better to apply the repetitions to another test section, in order to achieve the desired results in a shorter period.

Reliability concepts and Survival Analysis were used in order to estimate the numbers of loading cycles that would cause pavement failure (25 mm).

For pavement purpose, reliability may be then defined to be the probability that the pavement system will perform its intended function over its design life and under the conditions encountered during its operation. The term “reliability” is defined as the probability that something will not fail. [7]

The main purpose of reliability in any probability-based design is to make sure that an engineering system will not fail before a specified time. Failure occurs when an engineering system deteriorates to such an extent that it cannot accomplish its intended functions.

Defining \( R(N) \) as the reliability for a number \( N \) of loading cycles, \( F(N) \) is the cumulative probability function, given by

\[
F(N) = 1 - R(N)
\]

(1)
By deriving equation (1) we obtain the probability density function, \( f(N) \), given by

\[
f(N) = -\frac{dR}{dN}
\]  

(2)

The hazard rate is defined as the probability per number of loading cycles that a case that has survived to the beginning of the respective interval will fail in that interval. Thus, the hazard rate is given by

\[
h = \frac{R(N_1) - R(N_2)}{(N_2 - N_1)R(N_1)}
\]  

(3)

Survival analysis is a statistical method for estimating the distribution of lives, as well as the life expectancy of mean life, of subjects in experiment. [8]

One clear finding from observations of survival analysis is that even for a given type of pavement and specific structural design, there is a large amount of performance variability. These highly variable results indicate the need for a safety factor in design to ensure that a design will carry a certain amount of equivalent single axle loads (ESALs) at a high reliability level. Thus, the need for design reliability is obvious if the designer wants a high probability of success for any given design. Traditional design methods (such as the CBR equation) are considered to have a reliability of 50%, once no safety factors are included.

The Swedish physicist Waloddi Weibull introduced the Weibull distribution in 1939. It was demonstrated that when a pavement is modelled as a very large number of small discrete and independent “elements” which fail at a certain moment in the time, the Weibull distribution is theoretically the best function to describe the failure of pavement sections. [9]

The probability density function of a random variable \( N \) (number of load repetitions) that follows a Weibull distribution is of the form

\[
f(N) = \gamma (N - \delta)^{\gamma - 1} \exp\left(-\left(\frac{N - \delta}{\theta}\right)^\gamma\right), N \geq \delta
\]  

(4)

where \( \gamma \) is the Weibull shape parameter, \( \theta \) is the Weibull scale parameter and \( \delta \) is the Weibull location parameter (beginning of damage).

By varying \( \gamma \) and \( \theta \), a number of distribution shapes can be obtained. Typical values of \( \gamma \) vary between 0.5 and 8. In the Weibull distribution with two parameters (\( \delta = 0 \)), \( \theta \) is designed as “characteristic life”. 63.2% of failures occurs close to “characteristic life”, disregarding \( \gamma \) value.

On integrating (4), the Weibull probability cumulative function can be expressed as follows
In a Weibull distribution, the reliability and the hazard functions are, respectively, given by

\[ F(N) = 1 - \exp \left\{ - \left[ \frac{N - \delta}{\theta} \right]^\gamma \right\} \tag{5} \]

\[ R(N) = \exp \left\{ - \left[ \frac{N - \delta}{\theta} \right]^\gamma \right\} \tag{6} \]

and

\[ h(N) = \frac{\gamma(N - \delta)^{\gamma-1}}{\theta^\gamma} \tag{7} \]

It is possible to find distribution parameters for each test section by identifying two points on the curve: the origin where the traffic and the distress magnitudes are negligible, and the actual measured point. Based on this, the pavement designer can determine a relationship that can be utilized to determine traffic at some preset value of rutting, and subsequently a corresponding maximum rut depth that the designer considers acceptable. Also the maximum level of rutting will lead to the determination of maximum load cycles to failure for that section. [10]

Therefore, the loads to failure \((N)\) can be determined for all the pavement sections considered. The value of \(N\) is a unique property of each of the test sections and can be used as a basis for comparison because it represents a number of load applications at which a standard condition of rutting for each section was reached. The scale and shape parameters represent design calibration terms.

Figure 4 present the graph of Weibull reliability, \(R(N)\), corresponding to sector 0712. It may be seen that up to nearly 40,000 axle loads (120 kN) repetitions the reliability level is 100%. On the other hand, after 100,000 axle loads repetitions, the reliability level is 0.

As expected, as the reliability level increases, the number of axle loads to cause the pavement failure (25 mm rut depth) decreases.

Once calculated the number of loading cycles \(N\) of axle load \(L\) that would cause the failure of every sector, a multiple regression analysis was carried out, aiming at relating \(N\) to \(L\), to pavement thickness, \(T\) (surface treatment + weathered basalt layer) and to basalt point load strength index after soaking, \(IS\). The following model was proposed [4].

\[ N = 10^{\beta_1} \cdot L^{\beta_2} \cdot T^{\beta_3} \cdot IS^{\beta_4} \tag{8} \]
In model (8), $N$ is the number of axle loads corresponding to 100% degradation, $L$ is axle loads in kN ($82 \leq L \leq 130$); $T$ is pavement thickness in cm ($16 \leq T \leq 36$) and $IS$ is the weathered basalt point load strength index after 7-days soaking, in MPa ($3.8 \leq IS \leq 6.85$).

Since $N$ depends on the reliability level, values of $\beta_i$ also do. Table 2 synthesizes the multiple regression analysis for reliability levels of 50%, 80%, 90% and 95%. $R^2$ is the coefficient of determination and SEE is the standard error of estimate.

![Weibull cumulative probability function (sector 0712)](image)

**Figure 4** Weibull cumulative probability function (sector 0712)

**Table 2  Synthesis of the multiple regression analysis**

<table>
<thead>
<tr>
<th>R (%)</th>
<th>$\beta_1$</th>
<th>$\beta_2$</th>
<th>$\beta_3$</th>
<th>$\beta_4$</th>
<th>$R^2$</th>
<th>SEE</th>
<th>F</th>
</tr>
</thead>
<tbody>
<tr>
<td>50</td>
<td>6.26</td>
<td>-4.51</td>
<td>4.16</td>
<td>2.17</td>
<td>0.93</td>
<td>0.17</td>
<td>31.02</td>
</tr>
<tr>
<td>80</td>
<td>5.25</td>
<td>-3.97</td>
<td>4.12</td>
<td>1.98</td>
<td>0.93</td>
<td>0.17</td>
<td>28.49</td>
</tr>
<tr>
<td>90</td>
<td>4.41</td>
<td>-3.72</td>
<td>4.27</td>
<td>2.08</td>
<td>0.94</td>
<td>0.17</td>
<td>30.66</td>
</tr>
<tr>
<td>95</td>
<td>4.12</td>
<td>-3.61</td>
<td>4.29</td>
<td>2.07</td>
<td>0.94</td>
<td>0.17</td>
<td>30.94</td>
</tr>
</tbody>
</table>

Based on the statistical parameters $R^2$ and F, it is evident that $N$ is significantly ($\alpha=5\%$) related to the explanatory variables selected.

The AASHTO Guide for Design of Pavement Structures [11] suggests levels of reliability from 50% to 80% for low-traffic roads (equivalent to local rural roads). Considering $R=80\%$ the following model was obtained:

$$N = 10^{5.25 \cdot L^{-3.97 \cdot T^{4.12 \cdot IS^{1.98}}}$$

(9)
Model (9) shows that the number of loading cycles to cause a 25 mm rut depth:

a) Decreases exponentially (near 4\textsuperscript{th} power) as axle load increases;

b) Increases with the 4\textsuperscript{th} power of pavement thickness; and

c) Increases with the square of weathered basalt point load index.

Adopting in model (9) the Brazilian standard axle load, $L = 82$ kN, the following design equation was obtained [4]:

$$
\log N_{82} = -2.35 + 4.12 \log T + 1.98 \log IS
$$

In equation (10), $N_{82}$ is the number of cycles of the standard axle loads to cause pavement failure (rut depth of 25 mm).

Considering values of weathered basalt point load strength index after 7-days soaking, $IS$, design curves were obtained [12]. By using equation (10), it was concluded that a pavement with total thickness of 345 mm (including 25 mm of surface treatment), compacted on a subgrade with characteristics similar to that of test sections loaded by the traffic simulator, might withstand traffic volumes of $1.37 \times 10^5$ and $4.39 \times 10^5$ ESALs, if constructed with weathered basalt from sites E.1 and F.1 ($IS$ values of 3.80 MPa and 6.85, respectively).

**From APT to Practice**

In 1998 the results obtained in APT began to be put into practice. 21 km of a low-volume road were paved using the technology developed along the study. At that time the daily traffic volume was 244 vehicles, including 103 trucks and 5 buses. Considering a growing rate of 3\% per year, a traffic design (ten years) of $5.8 \times 10^5$ ESALs was computed.

The in-service test section, shown in Figure 5, is on State Road RS/132. This road crosses a hilly region (500 to 700 meters-high).

![Figure 5](image_url)
The pavement structure is similar to APT sections; 25 mm thick double surface treatment with sealing coat over dry-bound macadam (320 mm) base and subbase built with weathered volcanic rock. Subgrade consists of a residual soil with a CBR of 13%.

The weathered volcanic rock presented IS of soaked specimens of 10 MPa and Soaked to air-dried strength of 0.91.

From the opening to traffic in 1999 to October 2002, almost $3 \times 10^5$ ESALs trafficked that road. Rutting and cracking surveyed data show good correlation between APT and RLT results. Conversely, deflections in the in-service were higher (ranging from 0.38 mm to 1.07 mm) than in similar APT sections, probably due to a weaker subgrade and probably to deficiency of compaction in the macadam layer. [13]

The fact that the bearing capacity of the in-service road pavement might have been reduced due to construction problems displays the importance of the interactivity between researchers and practitioners for a successful technologic transfer.

_A Parallel Study – The Influence of Soils Suction on Subgrades Resilient Modulus_

In the last decades, great interest has been placed in evaluating subgrade soils deformability by means of laboratory or field tests. This is particularly justified in thin asphalt pavements, whose bearing capacity and elastic behaviour are strongly influenced by the subgrade soil underneath. Besides, all over the world, the use of pavement mechanistic design methods has become a definite trend. In such methods, subgrade resilient modulus is a fundamental input parameter.

Compacted subgrade soils are unsaturated soils, where suction plays a definite role in strength and deformability. Resilient modulus is sensitive to the stress state within the subgrade, and soil suction controls the stress state in unsaturated soils. Thus, it is important to quantify the influence of suction on resilient modulus in the design of new pavements and rehabilitation of existing pavements.

In an effort to obtain reliable resilient modulus of typical subgrade soils for design purpose, a parallel study [14], including laboratory and in situ tests (APT test sections), was carried out, with the goals of: a) quantifying the effects of moisture content and soil suction on soils resilient modulus; b) analysing the effects of drying and wetting cycles and compaction method on soils resilient modulus; c) measuring subgrade soils suction in situ; and d) comparing laboratory and in situ resilient modulus.

The subgrade of APT sections is a lateritic soil originated by chemical weathering of shale. Gravel, sand, silt and clay contents are 1%, 37%, 14% and 48%, respectively. Plasticity tests indicated a liquid limit of 44% and plasticity index of 21%. According to AASHTO, the soil was classified as A-7-6(7). Laboratory testing gave a standard Proctor maximum dry unit weight of 16.7 kN/m$^3$, at optimum moisture content of nearly 21%. CBR values (standard Proctor energy) ranged from 17% (for moisture content of 20.1%) to 3% (for moisture content of 26%, corresponding to 100% saturation degree).

The subgrade soil was compacted in three layers 20 cm thick. In situ density testing gave an average compaction degree of 103%. Moisture content for compaction ranged from 19% to 22% (average 20.4%), ensuring a minimum CBR value of 10%.

Laboratory experimental procedures consisted of: a) specimens preparation and compaction; b) establishing the suction-moisture content relationship (characteristic curve) by the filter paper technique; and c) resilient modulus testing on specimens submitted to drying, wetting, or wetting-after-drying paths, with suction measurement.
In situ measurements of soil suction and deflections (for resilient modulus backcalculation) were carried out at the APT facility in UFRGS Campus. Data analysed were obtained as part of the study research on weathered volcanic rocks.

Three jet-fill tensiometers installed along the test sections, with porous tips were embedded 30 cm below the top of the subgrade, measured subgrade soil suction during loading and deflection surveys.

During traffic periods, soil suction was never higher than 20 kPa, suggesting that, due to drainage deficiencies, the subgrade was almost saturated. In fact, such low suction values correspond to moisture contents higher than 25%.

Though in situ suction range was rather narrow (from 0 to 14 kPa), the effect on subgrade resilient modulus is highly significant, as shown in Figure 6. It is important to observe that the effective geostatic stress level acting on then top of the subgrade of thin pavements is generally lower than 10 kPa. Therefore, even suction values as low as 14 kPa represent significant increases to be taken into account when analysing soil elastic deformability. Those increases are sharply lowered by rising saturation degree, explaining the remarkable influence of soil suction on resilient modulus.

![Figure 6](image.png)

Figure 6  Influence of soil suction on soil backcalculated resilient modulus

Considering data presented in Figure 6, the relationship between in situ resilient modulus and matric suction was modelled:

\[
RM = 142 + 16.9 (\mu_a - \mu_w) \quad (11)
\]

In Model (11) RM is the back-calculated Resilient Modulus in MPa, and \((\mu_a - \mu_w)\) is the matric suction, in kPa. The regression is statistically significant. However, it must be stated that the model is true for suction values between 0 e 14 kPa. It is also noticed that while suction was measured along pavement edges, the deflection basins were measured along the trafficked lanes, thus representing a somehow different situation.
Comparing laboratory and in situ results it was observed that backcalculated moduli matched rather well laboratory moduli of kneading compacted specimens, but were substantially lower than those of specimens obtained by static or dynamic compaction.

Also, backcalculated moduli better agreed with laboratory results of specimens submitted to drying or wetting paths. Wetting-after-drying cycles resulted in laboratory moduli considerably lower than in situ values.

Lessons we have learned
The importance of providing a well-designed and conserved drainage system was demonstrated. Soil suction notably affects soils deformability. A simulation showed that the design traffic of a typical Brazilian pavement might be reduced to 50% if subgrade soil is excessively wet for long periods. [14]

Achievements
The first years of our APT facility were times of concerns, enthusiasm and discoveries. We were especially involved in improving and keeping the traffic simulator working and spent much time advertising how useful an APT facility might be to academic researchers and practitioners. Those were days of true partnership among UFRGS researchers, DAER/RS engineers and paving enterprises that provided resources to keep the installation working.

The objectives of the research on thin pavements with weathered basalts were fully accomplished. The innovative study on the effects of soil suction in subgrade soils resilient modulus helped to understand the complex elastic behaviour of unsaturated compacted soils.

We were proud to be visited by renowned colleagues from all over the world who wholeheartedly encouraged us to walk on, even if, sometimes, we questioned the course we were following.

The high point of those first years was the 1st APT Conference, held in Reno in 1999, when we felt kindly received by experienced engineers and academics in the worldwide APT community.

GOING ASPHALT
As the new millennium approached the research main subject moved to heavy traffic asphalt pavements, including the study of distress mechanisms and rehabilitation strategies. It had become clear that the installation could not longer count exclusively on UFRGS and DAER/RS resources and the incorporation of new partners was mandatory to keep the facility working.

The Study on Paving Fabric and AC Overlay over Existing Asphalt Pavements
In 2000, a long-term research, supported by a geotextile manufacturing enterprise, began with the purpose of comparatively evaluating the performances of asphalt overlays on cracked pavements with and without using paving fabrics.

Two kinds of geotextiles (named as G 150 and G 150-TF) were used as intermediate layers in a test section loaded by the traffic simulator and in an in-service real time loading road section.
In order to quantify the efficiency of paving fabrics, a test section (20 m long and 3.5 m wide) was built at the Pavement Testing Facility. The pavement structure was composed by a top subgrade lateritic soil, dry-bound macadam subbase, densely graded rock base, cracked asphalt concrete (4 cm thick) and AC overlay (5 cm).

On top of the granular base a 4 cm thick HMA layer was compacted. In order to represent an old asphalt layer already failed, cracks were sawn forming 50 cm x 50 cm square slabs. The cracks, all along the test section, were 4 cm deep (the old asphalt layer thickness) and 0.35 cm wide and the pattern simulated severe block cracking.

Núñez et al. (2008) characterize the materials used in the pavements layers and detail the paving fabrics installation procedures. [15]

**Paving Fabrics**

As previously stated, the test section was divided in three zones. In two of them (those at the endings) paving fabrics were installed between the cracked AC layer and the new overlay. No fabric was used in the central zone. Two different non-woven geotextiles, designed as G 150 and G 150-TF, were used one at each ending of the test section. Table 3 presents the fabrics properties.

<table>
<thead>
<tr>
<th>Property</th>
<th>Unit</th>
<th>Fabric G 150</th>
<th>Fabric G 150-TF</th>
</tr>
</thead>
<tbody>
<tr>
<td>Weight</td>
<td>g/m²</td>
<td>150</td>
<td>150</td>
</tr>
<tr>
<td>Nominal thickness</td>
<td>mm</td>
<td>1.5</td>
<td>1.0</td>
</tr>
<tr>
<td>Porosity</td>
<td>%</td>
<td>90</td>
<td>70</td>
</tr>
<tr>
<td>Asphalt retention</td>
<td>l/m²</td>
<td>1.2</td>
<td>1.1</td>
</tr>
<tr>
<td>Grab tensile strength</td>
<td>kN/m</td>
<td>8</td>
<td>9</td>
</tr>
<tr>
<td>Elongation</td>
<td>%</td>
<td>60</td>
<td>60</td>
</tr>
<tr>
<td>Normal permeability</td>
<td>cm/s</td>
<td>4.0 x 10⁻¹</td>
<td>2.7 x 10⁻¹</td>
</tr>
</tbody>
</table>

Among the above properties, grab tensile strength and asphalt retention are the most critical ones. Grab tensile strength indicates the fabric strength when it is pulled between the jaws of a testing machine until it ruptures. Asphalt retention is an indication of how much oil is necessary to saturate the fabric and make a bond.

**How pavements performed**

From May 2000 to March 2001 the traffic simulator applied 345,540 loadings (100kN axle-load) on the AC overlay. Traffic was reassumed in July 2005 and until December 2006 the traffic simulator applied more 636,660 axle-loads. All in all 982,200 cycles of the 100 kN axle-loads were applied.

Periodically, pavement sections and their environment were subjected to measurements of three types, namely: environmental; structural response; and performance.

Rainfall, air temperature and solar irradiation were continuously recorded in a meteorological station built in the Pavement Test Facility. Pavement temperatures were also recorded every time deflections and strains were measured. Annual average rainfall in Porto Alegre is 1,310 mm. Rainfalls are well distributed along 135 days of the year. There is not a rainy season, although summers tend to be drier. During testing air
temperatures ranged from 0 °C to 37 °C and pavement temperatures, measured in the interface between the cracked asphalt layer and the overlay, ranged from 4 °C to 56 °C.

During the first APT stage, from May 2000 to March 2001, when the traffic simulator applied 345,850 cycles of the 100 kN axle load, no cracks were observed on the AC overlay surface. In fact, the first crack appeared in the second APT stage, after 435,015 loading cycles. As expected cracks were first seen in the zone where the AC overlay was placed directly over the old cracked underlying AC layer, with no intermediate fabric.

The benefit of including paving fabric between the cracked AC pavement and the new AC overlay may be clearly seen in Figure 7. The first cracks appeared after 435,015 loading cycles in the overlay with no fabric and after 982,200 cycles the cracked length (summation of every crack length) was 413 cm. On the other hand, cracks on the AC overlays with intermediate paving fabrics just appeared after 982,200, totalling 80 cm in the zone with geotextile G 150 and 162 cm in the zone with G 150-TF fabric.

It is very important to notice that the cracked length in the AC overlay with G 150 fabric at the end of the testing (80 cm) was less than the cracked length measured on the AC with no geotextile after 474,395 loading cycles (88 cm). In a first approach it might be concluded that the usage of geotextile G 150 would significantly delay crack reflection and, thus, increase the life of the AC overlay by a factor of 2.

Conversely, the performance of the other paving fabric, geotextile G 150-TF, was not so good and at the end of the testing the AC overlay had accumulated a crack length of 162 cm, twice as much as the cracking length in the AC overlay with G 150 fabric, but only 40% of the cracking observed in the overlay with no fabric. So, it may be concluded that both geotextiles effectively retarded reflection cracking, deviating crack propagation, as shown in Figure 8.
Lessons we have learned
APT results showed the benefit of including paving fabric between cracked AC pavements and new AC overlay.

The system composed by AC overlay and geotextile G 150 remarkably delayed cracking reflection and outperformed the system of AC overlay with no fabric by a factor of two. The system composed by AC overlay and geotextile G 150-TF did not achieve such a good performance. Nevertheless, at the end of APT reflection cracking on the AC over G 150-TF was significantly lower (60%) than on the AC with no fabric.

The results obtained in the in-service road were not so conclusive. There, AC overlays with or without paving fabric performed similarly. Some fabric installation problems might have affected the performance of a given zone with geotextile G 150. And that reinforced the idea that paving fabrics installation procedures are critical.

A Parallel Study - Temperatures distribution across the thickness of asphalt layers
As an attempt to contribute to studies on rutting of asphalt pavements, on which the temperature has very high influence, a parallel study was carried out with the purpose of collecting temperature data in several depths in asphalt layers in order to model how the temperature distribution occurs. To make it possible, a weather station provided air data and solar irradiation and the local temperatures measurements provided the asphalt layers temperature. Measurements were continuously made along 18 months and models relating pavement temperature at depths up to 9 cm to air temperature and solar irradiation were proposed. [16]

Figure 9 exemplifies temperatures distribution across 9cm-thick of asphalt layers in a sunny summer day. Air temperatures were measured 3 cm above the pavement surface.
The influence of solar irradiation is clearly observed. At 14 h, surface pavement temperature (56°C) almost doubles air temperature (31°C).

**Lessons we have learned**

The knowledge of the distribution of temperatures across asphalt layers along the year helped to understand the severe rutting observed in bus lanes, on which the temperature has very high influence due to the heavy traffic and concentrated loads.

We learned that in summer days pavement temperatures frequently exceed the softening point of the asphalt binder commonly used in Southern Brazil and because of that rutting hazard is continuously increasing.

**Pavement Rehabilitation with Asphalt Rubber Overlays**

Accelerated pavement testing has proved to be a useful tool, providing a basis for understanding and counteracting distress mechanisms as crack reflection, as well as confirming and validating laboratory tests procedures and results.

Several APT studies focused the effects on performance of modified binders (including Asphalt Rubber) in Asphalt Concrete surface layers. In the CAL/APT program it was found that a gap-graded crumb-rubber-modified overlay outperformed a dense-graded mixture with conventional binder in terms of permanent deformation, and that it also exhibited substantial resistance to reflective cracking from the underlying failed AC layer [17]. The South-African HVS program also investigated the use of asphalt-rubber in a relatively thin, open-graded AC overlay of a cracked concrete pavement [18]. This overlay exhibited good performance in terms of reflection cracking when compared with many other rehabilitation overlays, including those with different types of intermediate layer.
Crack reflection is one of the most common failure mechanisms in asphalt pavements. The delay of cracks reflection in pavements overlays has stimulated the development of new techniques and materials. In Southern Brazil, asphalt rubber (AR) was firstly used with that purpose in the late 1990s. Several cracked pavements have had their performance improved by AR overlays. However, most reports focus binders and mixtures characteristics or construction procedures. Due to the short time elapsed and difficulties of traffic monitoring, there was a lack of data allowing comparing the efficiency of AR and asphalt concrete (AC) overlays.

In order to quantify crack reflection evolution in AR and AC overlays, a research was carried out by UFRGS and DAER/RS, supported by an asphalt industry and roads concessionaries.

Two test sections with identical structures (4 cm thick severely cracked AC wearing-course and granular layers 30 cm thick) were topped by either AC or AR overlays. Figure 10 shows the pavement structure with AR overlay.

![Figure 10 Pavement structure with AR overlay](image)

On top of the granular bases a 4 cm thick HMA layer was compacted. In order to represent an old asphalt layer already failed, cracks 4 cm deep (the old asphalt layer thickness) and 0.5 cm wide were sawn in a pattern simulating severe fatigue (alligator) cracking. In each test section, four cracked areas 1.20 m long and 0.80 m wide, alternating with uncracked areas, were sawn. Cracks were filled with clayey soil in order to prevent crack sealing by the tack coat applied before overlay setting. Further details on materials characteristics and construction procedures are presented elsewhere [19].

From July 2003 to February 2005, the traffic simulator applied 100 kN axle loads on two test sections pavements. In the AC overlay structure traffic was stopped at nearly 100,000 loading cycles, due to the extremely severe cracking accumulated. On the other hand, the traffic on the AR structure continued up to approximately 500,000 loadings.

Throughout pavements loading, cracks reflection was recorded, and deflections, strains and rut depths were measured.

Surface deflections and basins were measured with a road surface deflectometer. Tensile strains in asphalt layers were measured, using locally assembled Kyowa™ KM-120 strain gauges.
Rutting was measured using a profilograph. To eliminate initial surface irregularities from the rut depth data, profiles obtained before trafficking were used as references.

A manual procedure was used to record and measure cracking. Different colour sprays were used each time a crack survey was performed, as shown in Figure 11. Then, cracks were mapped using a square metal grid, 1.0 m x 1.0 m, with a mesh opening of 0.10 m. Afterwards, a cracking severity index, given by the ratio of cracking total length to trafficked area, was computed.

![Figure 11 Transversal cracks painted in different colours according to their appearance](image)

**How pavements performed**

Figure 12 shows cracking evolution in both structures [19]. It is clearly seen that the asphalt rubber overlay outperformed the conventional asphalt concrete one delaying crack reflection.

The first cracks appeared in the conventional AC overlay after no more than 14,000 cycles of the 100 kN axle-load. Cracks sawn in the ancient asphalt layer underneath propagated through the asphalt concrete, and after some 98,000 loadings the overlay showed a cracking density of 220 cm/m². On the other hand, the first crack appeared in the AR overlay after some 123,000 cycles of the 100 kN axle-load.

Comparing cracks evolution, it may be seen that the cracking severity index of:

- a) 50 cm/m² was attained after some 53,000 loading cycles in the AC overlay, and only after 294,000 cycles in the AR overlay;
- b) 100 cm/m² was attained after some 66,000 loading cycles in the AC overlay, and only after 339,000 cycles in the AR overlay, and
- c) 150 cm/m² was achieved after some 76,000 loading cycles in the AC overlay, and only after 455,000 cycles in the AR overlay
All in all, cracks reflected 5 to 6 times (average 5.55) slower in asphalt rubber than in conventional asphalt concrete. This may be attributed to the AR higher elasticity. Although both materials present basically the same tensile strength, the AR resilient modulus is nearly 27% lower than the AC stiffness. [19]

However, the results shown in Figure 12 had to be corrected, considering daily average temperatures during traffic on both structures. While the AC overlay was tested only in the 2003 Brazilian winter, the AR structure was trafficked during two springs, two summers and one winter. Average daily air temperatures during traffic on the AR overlay was 19.1°C, quite higher than 13.6°C, the corresponding value during traffic on the conventional AC overlay.

A remarkable fact was that surface cracks appeared not only in areas of the overlays where the underlying asphalt layer had been artificially cracked, but also in areas overlying intact asphalt layer. It was concluded that new fatigue cracks had appeared in the bottom of the lower AC layer, latter reflecting in the overlays.

Therefore, a supplementary laboratory study was carried out to quantify temperature effect on AC and AR resilient modulus, latter used in a fatigue analysis. Combining APT and fatigue analysis data, it was possible to ascertain that if both overlays were tested at identical pavement temperatures, the efficiency of the AR overlay delaying crack reflection would be 4.84 greater than that of the AC overlay. [20]

Lessons we have learned

APT results have clearly shown that asphalt rubber (AR) outperformed conventional asphalt concrete (AC) as overlay material. All in all APT results validated the results of laboratory studies reported elsewhere. Moreover, the research here reported allowed, for the first time in Brazil, to quantify the efficiency of asphalt rubber overlays compared to conventional asphalt concrete.
A similar study on surface treatments with polymer-modified binder as stress absorbing membranes (SAM) was carried out in 2003. Though those materials were less effective than AR overlays delaying severe cracking reflection, they proved to remarkably extend the residual life of pavements with low severity cracking. [21]

**Pavement Instrumentation; a Powerful Tool for APT Results Analysis**

Since the very first times of our APT Facility we were aware of the importance of pavement instrumentation as a powerful tool for analyzing results obtained in test sections loaded by the traffic simulator. The first experience with pavement instrumentation was a study on the performance of thin asphalt pavements with HMA wearing courses with conventional and modified asphalt binders [22]. Pressure cells and strain gauges were installed during pavements construction, and allowed measuring vertical and horizontal stresses and strains at several depths inside the structure loaded by the traffic simulator. Some results of that important study are shown in Figures 13 and 14.

![Figure 13 Typical patterns of longitudinal, transversal and at 45° measured at the bottom of a 4 cm thick asphalt-wearing course. [22]](image-url)
Lessons learned

Pavement instrumentation helped to understand the results of the study on the efficiency of AC and AR overlays on cracked pavements, previously discussed. Multidirectional strain gages as those shown in Figure 13 were positioned in the interfaces between the ancient cracked layer and the overlays. Periodically, longitudinal, transversal and 45° strains induced by different axle-loads (82 kN; 100 kN & 120 kN) were recorded. Subsequently, principal strains $\varepsilon_1$ and $\varepsilon_2$ were computed.

Principal strain $\varepsilon_1$ increased with traffic, ranging from 169 $\mu$strains to 386 $\mu$strains (values corresponding to the 100-kN axle-load). Considering that the strain behaviour of asphalt mixes is strongly influenced by temperatures, a statistical analysis was carried out [19], resulting in model (12).

\[
\varepsilon_1 = 0.001 N + 14.18 T – 99.25 \tag{12}
\]

In model (12) $\varepsilon_1$ is the higher principal strain (tensile); $N$ is the number of 100 kN axle-loads and $T$ is the interface temperature in °C. The model is statistically significant ($R^2 =0.94$) and shows that $\varepsilon_1$ increased with traffic (up to 98,597 cycles) and pavement temperatures (ranging from 16.4 °C to 28.8 °C).

Conversely, strains evolution in the interface between the AR overlay and the ancient asphalt layer was somehow different. Due to instrumentation failure, after 160,000 cycles was no longer possible to measure strains. Pavement temperatures did not statistically affect the strain behaviour; probably due to the scarce variation (19.5°C to 27.5°C) during the period strains were measured. Model (13) reasonably ($R^2 =0.51$) fits strains evolutions

\[
\varepsilon_1 = 126.5 + 0.0008 N \tag{13}
\]

Considering traffic of 100,000 repetitions of 100 kN axle-load and pavement temperature of 23.5° C (average value in both intervals), models (12) and (13) yield principal strains of 334 $\mu$strains and 207 $\mu$strains, respectively. Although this is just an exercise to
compare strains evolution, it seems that the AR overlay better distributes stresses, thus resulting in lower tensile strains and longer fatigue life.

Figure 15 helps to compare the evolution of Principal strain $\varepsilon_1$ in both structures, justifying the outstanding differences between cracking evolutions shown in Figure 12.

![Figure 15 3-D representations of models (12) and (13), from left to right. [19]](image)

**Achievements**

We succeeded to locally assemble strain gauges, not only for measuring horizontal strains, but also gauges for measuring vertical strains in soils and unbound aggregates. Graduate students keenly worked developing data acquisition & storage systems and measuring strains and stresses.

Pavement instrumentation remarkably enriched the analysis of the structural behaviour of test sections loaded by the traffic simulator. However, we confess that most of data collected and stored are still to be analysed.

**Parallel study - Field Investigation of the Structural Behaviour of Ribbed PVC Flexible Pipes Enveloped by Sand**

The structural mechanism of a flexible pipe-soil system is based on the interaction between the pipe and the surrounding material. The low stiffness of a flexible pipe allows high displacements under applied vertical loads, with consequent vertical diameter reduction and horizontal diameter increase. The horizontal displacements mobilize the passive thrust of the envelope soil.

In 1958, Watkins [23] proposed an equation for computing the soil reaction modulus $E'$, known as the Modified Iowa Equation (14).

$$\Delta y = \frac{D_l \cdot K \cdot (p + q)}{8 \cdot RA + 0.061 \cdot E'}$$

(14)
The soil reaction modulus E’ is the most difficult factor to determine in equation (14). It expresses the pipe-soil interaction and cannot be directly measured. The determination depends on tests in reduced scale or on field tests, with measurement of vertical diametrical displacements of a buried pipe under applied loads; the E’ value is evaluated by means of back-analysis using the Modified Iowa equation. Further information about other parameters in equation (14) is provided by Schmitz et al. [24].

The experimental study carried out in our APT Facility in 2001-2002 started by digging four ditches, where 800mm-diameter ribbed plastic pipes were installed. Two of the ditches were shallow, with the pipe next to the surface; and two were deep, for which the influence of the wheel load is less significant.

For each pair of ditches at the same depth, one had a strongly compacted envelope material (uniform sand), and in the other the envelope material had been simply dumped. In spite of the fact that envelope materials should be well compacted around flexible pipes, Brazilian construction practice consists of dumping backfill into the trenches, once that only rigid pipes have been employed. Ditches width and length were 1.4 and 5.0 meters, respectively. The backfill thickness on top of the pipe was merely 0.3 meters in the shallow ditches and 1.5 meters in the deep ones.

After backfill compaction, the ditches were covered by PCC pavers.

Loads were applied by the traffic simulator at increasing stages, ranging from 30 to 60 kN. At first, cyclic loads, at an average speed of 6 km/h, were applied. Then, the same axle load was statically applied, keeping the wheel over the ditch surface for five minutes.

Pipe vertical diametrical displacements were recorded during the loading by means of two LVDTs installed inside the pipes, with strokes of ± 50 mm.

*How pipe-soil systems performed*

In the deep ditches, measurements showed that the pipes did not suffer any elastic vertical displacement under loading, even for the dumped envelope material. This is due to the very low stresses due to wheel load acting on the top of the pipe, because of the thick backfill over the pipe. Nevertheless, high elastic vertical displacements under wheel load were measured in shallow ditches, even where the backfill material was well compacted. An example of recoverable displacements recorded under the 60 kN loading stage of the pipe installed in the compacted shallow ditch is presented in Figure 16.

When the uniform sand was well-compacted, the soil reaction modulus, E’, varied from 3 to 4 MPa, even in the shallow ditch. However, in the ditch where the sand was just dumped, E’ ranged from 0.5 to 2.5 MPa, in shallow and deep ditches, respectively.

Although compaction and depth contributed to increasing E’ values, it should be noticed that E’ values of uniform sand are very low in comparison to values of crushed stone or well-graded granular materials, which can be up to 14 MPa. Another problem related to uniform sand is the extreme variability of E’ (from 0.5 to 4 MPa) due to construction and geometric characteristics.

The research has confirmed that shallow ditches and low relative compaction of envelope material result in a high-risk combination.
How pavements performed
Rut depths measured on the surface of the ditches, under cyclic and static loads, ranged from 10 to 50 mm. The only acceptable values are those measured on top of the deep ditch with compacted envelope material.

Dynamic elastic displacements of the pipe are directly related to deflections of the pavement structure. The values corresponding to shallow ditches varied from 1.7 (compacted backfill) to 8.3 mm (dumped backfill), therefore, dumped uniform sand is unacceptable as envelope material, because of the extremely high level of deflections. Compacted uniform sand in shallow ditches also presents high deflections that could cause premature failure of the pavement structure, despite being totally acceptable in soil-pipe systems.

The only acceptable system regarding pavement performance is a deep ditch with compacted uniform sand as envelope material.

Lessons we learned
Data resulting from the experimental ditches testing confirmed the importance of compaction. Uniform sand $E'$ values are much lower than those of well-graded granular materials, making clear the importance of gradation in granular materials behaviour.

In order to provide acceptable performance of soil-flexible pipe system and pavement structure, it is mandatory that the pipes be installed in deep ditches with well-compacted envelope material.

The most important contribution of this study was emphasizing the importance of computing elastic displacements under loads. Most researches consider $E'$ values resulting from backanalysis of displacements measured under static loads, ignoring the effects of those elastic displacements under moving loads on pavement structures.
CHALLENGES WE WILL HAVE TO FACE
We are no longer beginners; still we have a long way to go.

In the years to come we will have to face new and old challenges.

We must intensely work to enhance data interpretation. Traffic simulators accelerate pavement distress by applying high axle loads at high loading frequency. The effects of high axle-load are generally computed using loading factors, as those derived from the results of AASHO Road Test. However, linear traffic simulators apply loads to pavements at very low speeds (generally lower than 10 km/h) and this also accelerates pavements degradation, especially when they include thick asphalt mixes.

Loading speed has a strong effect on asphalt layers strains, and therefore on asphalt mixes fatigue life. In order to apply APT results to real pavements on which trucks apply loads at speeds many times higher than linear traffic simulators do, we will have to advance in the field of viscoelastic behaviour of asphalt binders and mixes. Meanwhile, we propose a simplified approach in another paper [25] addressed to this Conference.

It is highly desirable to more thoroughly integrate climate effects in the analysis of pavements performances under accelerated traffic. We already know the influence of soil suction in the resilient modulus of our APT facility subgrade and have succeeded to model temperature propagation across the thickness of asphalt layers. But we still have to develop a reliable model for correcting deflections according to pavements temperatures.

We still make assumptions that we know are not completely true. In spite of the efforts carried out by our colleagues from South Africa and California, we still assume that wheel loads are uniformly distributed on circular surfaces. We need to improve pavement instrumentation and develop software to overcome gaps like that.

More than anything else we have to strive to make TT more than a meaningless abbreviation. We sadly admit that many Brazilian pavement engineers and roads authorities still see APT as a worthless sophistication. Notwithstanding, in recent years two mobile traffic simulators were designed and built, based in our old good traffic simulator. One of them has continuously tested in-service pavements in Rio de Janeiro and São Paulo states since 2003. In National Conferences on Pavements held in the last twelve years, new results of APT researches have been reported. The idea that APT is essential when developing innovative pavement materials, such as high modulus asphalt mixes [26], is growing.

We must think of technologic transfer as a mantra we should recite at every moment. If we succeed shortening the distance between APT results and paving practices, more than improving the interactivity between researchers and practitioners, we will bring new partners to support our Facility, to provide funds to Graduate students and to help us advertising what such a facility could make for better pavements.

As anticipated in the beginning of this paper we have intentionally defied the principles of scientific publication to let the reader know how happy we are for having a history, perhaps neither long nor bright, to tell; a history of concerns and hopes of people who dared trying to follow the steps of notable Pavements engineers and researchers of the most developed countries.

We truly thank the commitment of our fellows engineers and Academy researches and to all those who helped us to keep the facility working. Finally we thank the generous invitation to write a paper for a Plenary Session of this 3rd APT Conference.
ACKNOWLEDGEMENT
We would like to express our sincere appreciation to our colleague and dear friend Lélio Antônio Brito for valuable suggestions that improved the paper redaction. Many thanks to former Graduate students, who provided data and performed some analysis here presented. Running an APT Facility would not be possible without the financial help of Brazilian Agencies for Research (CNPq, CAPES and FAPERGS) and private companies, to whom we are recognized. Last, but not less important; thanks to the UFRGS-DAER/RS brave team for conducting tests, collecting data and friendly supporting us.

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