Predicting Roughness Progression Of Asphalt Pavements By Empirical-Mechanistic Model

D'Apuzzo, M.
University of Cassino , Italy

Nicolosi V.
Civil Engineering Department, University of Rome "Tor Vergata", Italy

Mattarocci, M.
Transportation Department, University of Naples "Federico II" , Italy

SYNOPSIS

The evaluation of road roughness progression represents one of the most important issue of modern pavement design and management. The roughness deterioration models developed so far, are often based on an empirical approach. Usually, an extensive long-term measurement campaign on a wide range of pavement sections is needed to calibrate such complex regression models and additional difficulties occur in collecting data on overlay roughness evolution. Furthermore, empirical models are unable to take into account new pavement materials performance. Empirical-mechanistic approach could partly solve these problems allowing the development of deterioration models by numerical simulations.

In this paper an empirical-mechanistic model, implemented in a computer software, is described in order to simulate the progression of pavement roughness as function of loads, climate, structural configuration, overall construction quality and material properties. The analysis is performed by dividing the road section into 0.25÷0.33 m long sub-sections. Material properties and layer thickness are assigned to the sub-section through a autoregressive process and non linear elastic behaviour of granular materials is taken into account. The simulation is performed through an iterative process according to which, within each computational step, the program calculates pavement stress-strain level and permanent deformation induced by dynamic loads, for 16 different types of axes and 8 daily climatic condition in each sub-section. At the end of step the total rutting is computed in each sub-section, the profile is updated and the IRI index of the road section is evaluated. The model has been used to simulate roughness progression and a preliminary parametric analysis has been performed whose results are briefly illustrated.

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INTRODUCTION

Prediction model for pavement performance or deterioration have become an essential part of modern pavement design and management technology. As matter of fact the accurate prediction of pavement performance is important for efficient management of the road infrastructure, as it can produce a significant budget saving through an accurate planning of the interventions. Suitable models for pavement management system (PMS) should be easy to be developed and implemented, and as accurate as possible. Prediction models for pavement performance are also used in road pricing and regulation studies aimed at detecting restrictions to heavy vehicle traffic and setting criteria for road pricing, where both the deterioration of the pavement over time and the relative contribution of the various factor to deterioration are important issues.

Performance prediction models are often developed by empirical approach, using regression, where the dependent variable of observed or measured structural deterioration is related to one or more independent variables, such as ESAL repetition, pavement and subgrade strength (e.g. SN), environmental factors, etc.. As matter of fact, empirical models are often regarded as the only ones which are carefully able to predict pavement roughness progression of in-service pavement section, since they have been extensively and successfully employed in the past. However, empirical models may suffer from several limitations: problems caused by data shortage in the implementation phase of the PMS, problems caused by the presence of multi-collinearity between explanatory variables (field data), biases caused by the use of endogenous explanatory variables (field data) or by inaccurate representation of the true deterioration mechanism (accelerated loading pavement tests), etc. Furthermore they are unable to take into account new pavement materials performance and vehicle innovation which are relevant in regulation studies.

Empirical-mechanistic pavement prediction models seem to be able tackle the above mentioned problems, as suggested by some authors (Ullidtz and Larsen, 1983, Collop and Cebon, 1995). On the other hand the development of empirical-mechanistic models are computationally complex and they require a careful material characterization. Recent researches deeply dealt with the latter problem, and more detailed information are now available on resilient and non-resilient (permanent) behavior of bituminous concrete and granular materials, used in flexible pavement (e.g. COST 337 - 2000, COURAGE - 1999, NHCRP 1-37A – 2003, SHRP). At the same time the computational tools are making giant strides and actually the time is probably ripe for the development of nearly mechanistic pavement deterioration models.

The objective of this paper is to demonstrate the feasibility of the development of a empirical-mechanistic pavement roughness models by numerical simulation. A mathematical model has been developed for simulating the deterioration process of flexible road pavements, by taking into account pavement structure, traffic loads, vehicle characteristics, material properties and climatic conditions. The model that was implemented in a computer program, allows to develop deterioration models, for a specific road section. An empirical-mechanistic approach was followed in the development of the deterioration process, but an effort has attempted to make all required input data as simple as possible, in order to make it applicable in most of the cases.

MODEL DESCRIPTION

The framework of the model is shown in Fig. 1, it follows a pattern similar to that developed by Ullidtz, Cebon and Collop (Ullidtz and Larsen, 1983, Collop and Cebon, 1995, Cebon 1999), but differ in the way climatic conditions, traffic distribution, material characteristics and initial roughness are considered.

According to the approach followed, the road length is discretised in a number of 0.3÷0.25 m long sub-sections. Different layer thickness and mix properties are assigned to each section by means of autoregressive time series. Furthermore, dynamic loads are assumed to be applied to the surface at the middle point of each sub–section. Primary response due to dynamic traffic loads and the subsequent permanent deformations of pavement layers and subgrade are evaluated for each section and for each calculation step. The road pavement profile are updated basing on total permanent displacement of each sub-section and the process goes on until the end of the analysis period is reached.

The resilient characteristics of granular layers (i.e. subbase, subgrade and eventually base) are updated at the end of each year basing on the stress state evaluated (see unbound granular materials characteristic and equation 23), in order to take into account their non linear behaviour.

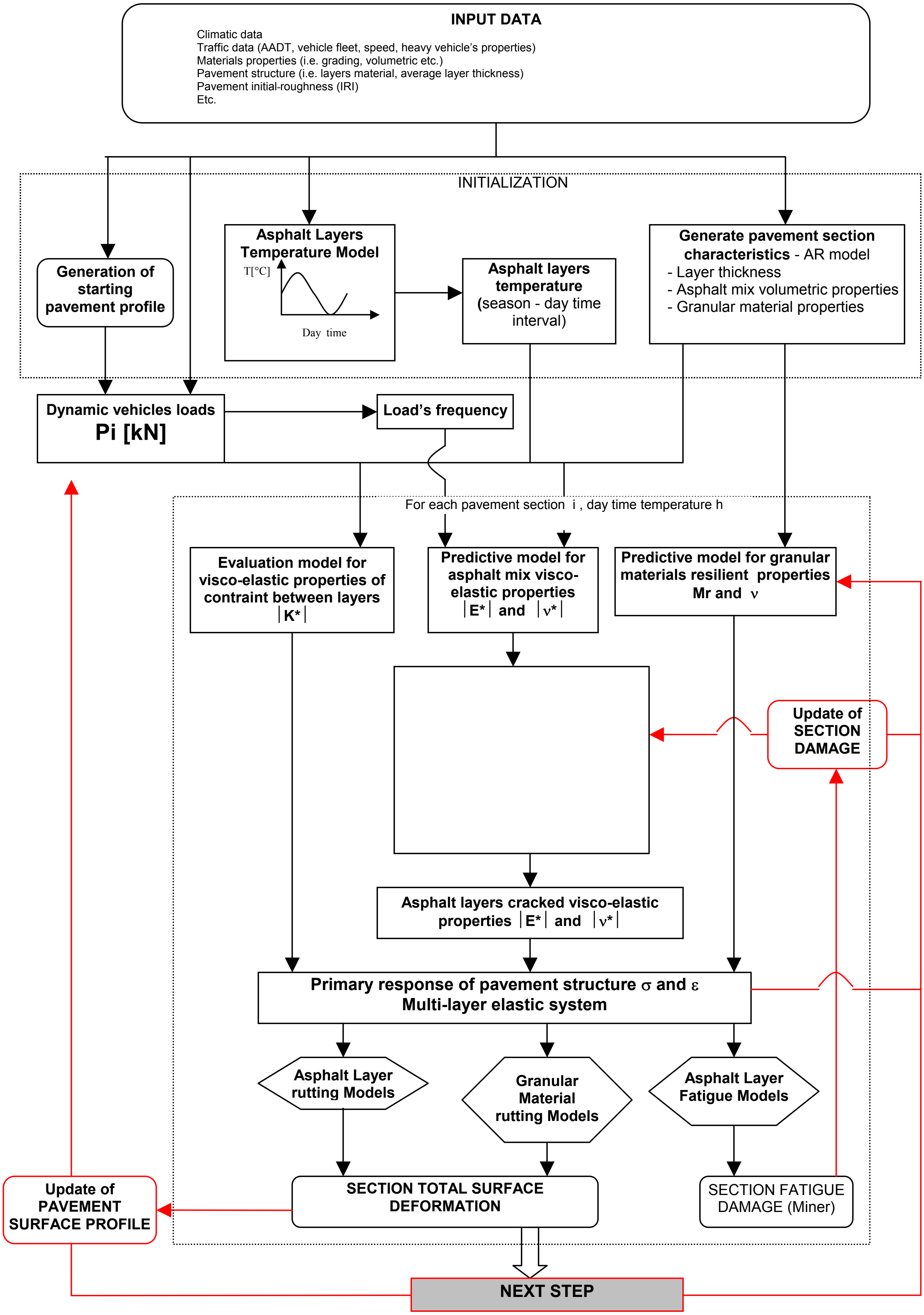

Figure 1 - Framework of the simulation model

Climatic Factors

Climate greatly affects the performance of flexible pavement in several way. In the model the environmental effects considered are: temperature and ageing of the asphalt layers. The effects of the moisture content in the granular material (sub-base and subgrade layers) could be introduced provided that the relevant parameters in the resilient modulus and permanent deformation laws are derived by cyclic tri-axial tests performed in laboratory. The effect of frost was not considered as it is negligible in most of Italy.
Average seasonal (or monthly) climatic conditions were characterized through the follows data: average air temperature, T_m, average daily variation in the air temperature A_g , average daily radiation I, average wind speed, v (see Table 1).

Table 1 : Sample of climatic data for central Italy (Di Mascio and Domenichini 1995).

SEASON	Average air temperature T_m [°C]	Average daily variation in the air temperature A_g [°C]	Average daily radiation I [kCal / gg]	Average wind speed v [m/sec]
Winter	3÷5	5÷7	2718	13÷18
Spring	11÷13	8÷10	5785	12÷19
Summer	22÷24	10÷12	6507	9÷15
Autumn	13÷15	7÷9	3547	11÷17

The model proposed by Barber was used to evaluate the temperature at mean depth of the asphalt layers of the pavement hour by hour, during the standard day of each season, or month (Barber, 1957):

$$T_{pav}(z,t) = T_m + R + (A_g/2 + 3\times R) \times F \times \exp(-C\times z) \times \sin [0.262\times t - C\times z - \mathrm{arctg}(C/(H+C))] \qquad (1)$$

where:
$T_{pav}(z,t)$ is 24 hour periodic temperature of the road pavement in [°C] at time t (from 1 to 24) and at depth z [m],
Tm , Ag , I and v are defined in Table 1,
$R=2/3 \times (b\times I) / (24\times h_c)$ [°C],
$H= h_c/K$ [1/m],
$h_c=4.882\times (1.3+0.4332\times v^{3/4})$ [kcal/hour m^2 °C],
$C=[(0.131\times s\times w) / K]^{1/2}$ [hour$^{0.5}$/m],
$F=H / [(H+C)^2 + C^2]^{1/2}$,
b is the absorptivity of surface to solar radiation (e.g. 0.6 ÷ 0.95),
K is the conductivity [kcal/m hour °C] (e.g. 1.04 ÷ 1.96),
s is the specific heat [kcal/kg °C] (e.g. 0.193 ÷ 0.22),
w is the density [kg/ m^2].
The sinusoidal variation in temperature suggested by Barber's model, was discretised in three hour periods, as illustrated in **Figure 2**. During each day time interval the response of the pavement to dynamic loads was calculated.

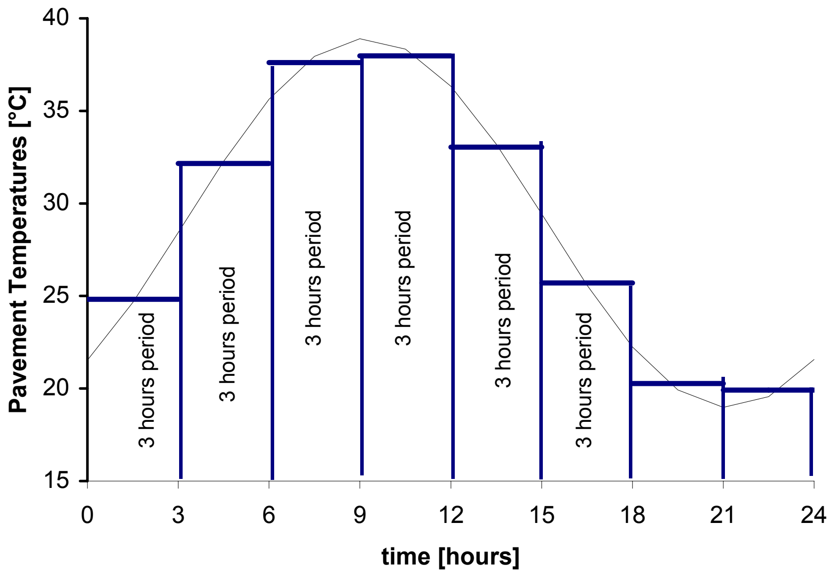

Figure 2 – Three hours period mean temperature of the pavement layers calculated by Barber's model.

Pavement surface profile

The smoothness of a newly laid pavement is a function of the quality of construction operations and, therefore, the initial road profile should be a known input parameter. If data on initial road profile are not available, two main approaches can be suggested:

- selection of a road profile from databases available in literature, such as the Long Term Pavement Performance (LTPP) Database;
- generation of an artificial road profile from the stochastic representation in terms of Power Spectral Density Function of Vertical Displacement according to ISO classification of road surface roughness.

In the former case, the initial road profile selected from LTPP database is characterized by a defined value of IRI, therefore, if different assumptions on quality construction have to be made basing on specific site conditions, initial IRI has to be modified accordingly. This can be achieved by simply scaling the initial road surface profile since IRI can be regarded as a linear operator. Therefore by multiplying profile elevation values by a suitable coefficient, that can be expressed through the ratio between the target IRI initial value and the actual IRI initial value of LTPP pavement, a road profile complying with on-site quality construction level can be derived.
In the latter case, it is worth to recall some concepts on stochastic description of road surface profile. Since early seventies, several authors (ISO 1995, Cebon 1999) have highlighted that longitudinal road surface profile can be regarded as a stationary and ergodic stochastic process. Therefore, it is possible to describe the road profile by means of a Power Spectral Density Function (PSD) of Vertical Displacement of road profile that is obtained through the Fourier Transform of the Auto-correlation function of the stochastic process describing the road profile. Starting from a continuous road profile, for a defined value of spatial frequency, *n*, centered within a frequency band Δn, the value of the Power Spectral Density Function for the assigned frequency, *n*, is defined through the following expression (Bendat et al. 1986, Diana et al. 1997):

$$G_d(n) = \lim_{\Delta n \to 0} \frac{\Psi_x^2(n, \Delta n)}{\Delta n} \qquad (2)$$

where Ψ_x^2 is the mean square value of the component of the signal for the spatial frequency n, within the frequency band Δn. However, the road profile signal is not measured continuously but it is conveniently discretised and therefore it is described as a sequence of elevation points uniformly spaced. If the length of road profile is *L* and the sampling interval is *B*, the maximum theoretical sampling spatial frequency is n_{max} = 1/*B* whereas the maximum effective sampling spatial frequency, according to Nyquist theorem, is $n_{eff} = n_{max}/2$ and, within the frequency domain, discretised spatial frequency values, n_i, are equally spaced with an interval of $\Delta n = 1/L$. Therefore, the generic spatial frequency value, n_i, can be regarded as $i \cdot \Delta n$ and the (2) can be re-written in the discrete form:

$$G_d(n_i) = \frac{\Psi_x^2(n_i, \Delta n)}{\Delta n} = \frac{\Psi_x^2(i \cdot \Delta n, \Delta n)}{\Delta n} \qquad (3)$$

with i varying from 0 to $n_{max}/\Delta n = N$.
If the road profile can be described through a simple harmonic function according to the following expression:

$$y(x) = A_i \cos(2\pi \cdot n_i \cdot x + \varphi) = A_i \cos(2\pi \cdot i \cdot \Delta n \cdot x + \varphi) \qquad (4)$$

where A_i is the amplitude, n_i is the spatial frequency and φ is the phase angle, it is possible to demonstrate that the mean square value of this harmonic signal is:

$$\Psi_x^2 = \frac{A_i^2}{2} \qquad (5)$$

Therefore from (5) it can be easily derived that the relationship between the spectral content, obtained by means of a Fourier Transform, and the PSD of a discretised road profile can be defined through the following expression:

$$G_d(n_i) = \frac{\Psi_x^2(n_i)}{\Delta n} = \frac{A_i^2}{2 \cdot \Delta n} \qquad (6)$$

However, it has to be underlined that the PSD value of vertical displacement of a road profile for a spatial frequency, n_i, is not directly linked to the Fourier Amplitude, A_i associated to the same spatial frequency by means of the expression (6) but, on a more rigorous point of view, it has to be regarded as the mean value derived from a statistically significant population of road profile measurements on which a Fourier Transform has been performed.
Several authors (Cebon 1999, Diana et al. 1997, Cole et al. 2004) have shown that, once the PSD function of vertical displacement is known, it is possible to generate an artificial road profile by means of the relationship between the spectral content and the PSD function of a road profile. As a matter of fact, by making use of expression (6) and by assuming a random phase angle, φ_i, following an uniform probabilistic distribution within the $0 \div 2\pi$ range, the artificial profile can be described through the following expression:

$$y(x) = \sum_{i=0}^{\frac{n_{\max}}{\Delta n}} A_i \cos\left(2\pi \cdot n_i \cdot x + \varphi_i\right) = \sum_{i=0}^{\frac{n_{\max}}{\Delta n}} \sqrt[2]{2 \cdot \Delta n \cdot G_d(i \cdot \Delta n)} \cdot \cos\left(2\pi \cdot i \cdot \Delta n \cdot x + \varphi_i\right) \qquad (7)$$

Recalling the ISO classification of road roughness, the PSD function of vertical displacement can be represented by means of the following equation:

$$G_d(n) = G(n_0) \cdot \left(\frac{n}{n_0}\right)^{-2} \qquad (8)$$

where n_0 is 0.1 cycles/m and $G(n_0)$ is reported in the following table (Table 2):

Table 2: $G(n_0)$ values for road roughness ISO Classification

Road Class	**$G(n_0) \cdot 10^{-6}$ [m^3]**
A Upper limit – B Lower limit	32
B Upper limit – C Lower limit	128
C Upper limit – D Lower limit	512
D Upper limit – E Lower limit	2048
E Upper limit – F Lower limit	8192
F Upper limit – G Lower limit	32768
G Upper limit – H Lower limit	131072

Substituting the (8) in the (7) an artificial road profile from ISO classification can be generated through the following equation:

$$y(x) = \sum_{i=0}^{\frac{n_{\max}}{\Delta n}} \sqrt[2]{\Delta n} \cdot 2^k \cdot 10^{-3} \cdot \left(\frac{n_0}{i \cdot \Delta n}\right) \cdot \cos\left(2\pi \cdot i \cdot \Delta n \cdot x + \varphi_i\right) \qquad (9)$$

where k is a function of ISO road class as reported in Table 3.

Table 3: *k* values for artificial road profile generation according to ISO Road Class.

Road Class	***k***
A Upper limit – B Lower limit	3
B Upper limit – C Lower limit	4
C Upper limit – D Lower limit	5
D Upper limit – E Lower limit	6
E Upper limit – F Lower limit	7
F Upper limit – G Lower limit	8
G Upper limit – H Lower limit	9

In the following figure (Figure 3) an artificially generated profile derived from a PSD function of vertical displacement dividing the A from B road class is reported. The sampling interval assumed is 0.25 m whereas the overall road profile length is 100 m. Furthermore, the minimum frequency interval, Δn, is 0.01 cycles/m , whereas the maximum theoretical spatial frequency, n_{max}, is 4 cycles/m and maximum effective spatial frequency, n_{eff} is 2 cycles/m, therefore the signal is composed with 400 harmonic functions. In the figure is also reported the same artificial road profile generated by taking into account the first 200 harmonic functions till the maximum effective spatial frequency, n_{eff}. The mean difference in elevation between the twp road profiles is below 10 % whereas the difference between the corresponding values of IRI (evaluated at a travel speed of 80 Km/h according to (Sayers, 1995)) is below 2 %.

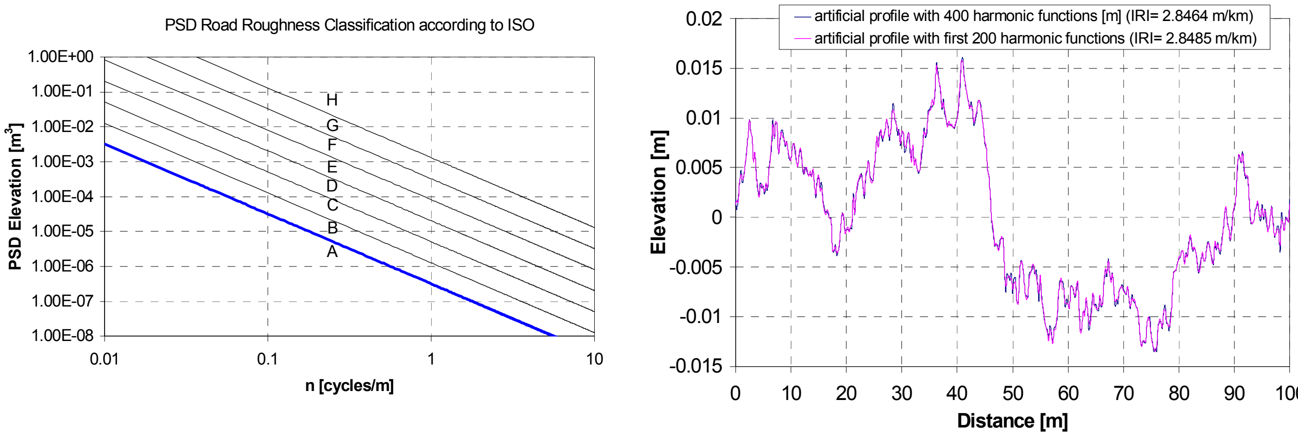

Figure 3 - ISO road roughness classification (on the left); artificial road profile generated from ISO road class A upper limit PSD vertical displacement function (on the right).

Traffic loads

The performance of the pavement is affected by heavy vehicle traffic and, in particular, by cumulative traffic, vehicle type, weight and speed. The annual average daily traffic (AADT), and the heavy vehicle traffic spectrum were considered as input data for the simulation model. Fifteen types of heavy vehicles were used to describe traffic flow, as suggested by Italian Catalogue of pavement structures (C.N.R. 1995) (see Table 4).

In order to evaluate the dynamic tyre force, a simple linear "quarter car" vehicle model was used; the model parameters, for the heavy vehicles considered, are summarized in Table 4. The dynamic tyre forces generated by each vehicle model were obtained by numerically integrating the equation of motion using the pavement surface profile displacement as input to the tyre (load was set to zero when wheel leave the ground in very rough profile). Although a more refined and non linear vehicle model could be employed, however this simple vehicle model has proved to yield a reliable estimation of vertical dynamic forces transmitted by heavy vehicles to the pavements, as highlighted by several authors (Cebon, 1993, Cebon, 1999). Preliminary results have yielded an average Dynamic Load Coefficient of 0.035 for the initial profile (IRI ≈ 1 m/Km, V=80-90 Km/h) and of 0.07 for the final profile (IRI ≈ 3 m/Km, V=80-90 Km/h) that, although fairly moderate, seem to agree with values available in literature (Cebon, 1993, Cebon, 1999).

However, in future it is planned to improve the vehicle model, by increasing the degrees-of-freedom in order to describe dual or triaxle suspensions and by introducing non-linear springs and dashpots.

Table 4: Axle type and load, as well as suspension characteristics of vehicle classes introduced in the simulation model (Lu Sun, 2001 and Cebon, 1993) .

Vehicle type	Axle type	Axles load [kN]	Ms [kg]	Ku/Ms	Ks/Ms	C/Ms
1) Light truck	S + S	↓10 ↓20	2898	280.70	201.10	0.5
2) " "	S + S	↓15 ↓30	4348	280.70	201.10	0.5
3) Light and medium truck	S + S	↓40 ↓80	11232	333.23	210.78	2.5
4) " " "	S + S	↓50 ↓110	14978	333.23	210.78	2.5
5) Heavy truck	S + T	↓40 80↓ ↓80	18138	385.80	220.40	4.5
6) " "	S + T	↓60 100↓ ↓100	23580	385.80	220.40	4.5
7) Truck with trailer and Articulated truck	S + S + S + S S + S + T	↓40 ↓90 ↓80 ↓80 ↓40 ↓90 80↓ ↓80	26300	385.80	220.40	4.5
8) " "	S + S + S + S S + T + T	↓60 ↓100 ↓100 ↓100 ↓60 ↓100 100↓ ↓100	32648	385.80	220.40	4.5
9) " "	S + T + S +S S + T + T	↓40 80↓ ↓80 80↓ ↓80 ↓40 80↓ ↓80 ↓80 ↓80	32648	385.80	220.40	4.5
10) " "	S + T + S +S S + T + T	↓60 90↓ ↓90 ↓100 ↓100 ↓60 90↓ ↓90 100↓ ↓100	39904	385.80	220.40	4.5
11) " "	S + S + TR	↓40 ↓100 ↓80 ↓80↓80	34462	385.80	220.40	4.5
12) " "	S + S + TR	↓70 ↓110 ↓90 ↓90↓90	40811	385.80	220.40	4.5
13) Dumpers	S + S + TR	↓4 0 ↓130 ↓ 130 ↓130↓130	47159	385.80	220.40	4.5
14) Bus	S + S	↓40 ↓80	10883	385.80	220.40	4.5
15) "	S + S	↓60 ↓80	12697	385.80	220.40	4.5
16) "	S + S	↓50 ↓80	11790	385.80	220.40	4.5

S=single axle, T= tandem axle, TR= tridem axle

Load frequency in asphalt layer was evaluated as suggested by Pellinen et al. (2004) but a correction was introduced to take into account the footprint area:

$$f = \frac{1}{2 \cdot \pi \cdot 10^{(50 \cdot h)} \cdot \frac{2 \cdot a}{v^{0.94}}}$$

where

f is the frequency of loading [Hz],

h is the depth from surface [m],

a is the radius of the contact area between the tire and the road surface [m],

v is the vehicle speed [m/sec].

Material properties and thickness variability

The sub-section characteristics, i.e. layers thickness and relevant bituminous mix properties, vary from point to point and they can be considered as a time series, and therefore represented through stochastic processes. In the program a second order autoregressive stochastic process was used to describe the variation of the characteristics from section to section, as suggest by Ullidtz and Larsen (1983):

$X_t = \Phi_1 * X_{t-1} + \Phi_2 * X_{t-2} + a_t$ (10)

where:

a_t is a Normal random variable having mean zero and variance σ^2_a.

The Φ_1 e Φ_2 are model parameters that can be estimate by Yule-Walker equations (Box and Jenkins, 1976):

$\Phi_1 = [(\rho_1*(1-\rho_2) / (1-\rho_1^2)]$ e $\Phi_2 = [(\rho_2-\rho_1^2) / (1-\rho_1^2)$ (11)

where ρ_1 and ρ_2 are the theoretical autocorrelation functions which can be replaced by estimated autocorrelations functions $r_k = \frac{c_k}{c_0}$ and $c_k = \frac{1}{N}\sum_{t=1}^{N-k}(x_t - \bar{x}) \cdot (x_{t-k} - \bar{x})$ k=1,2.

The variance σ^2_a, is a function of the variance of the process: σ^2_x: $\boldsymbol{\sigma^2_a = \sigma^2_x * (1-\rho_1*\Phi_1-\rho_2*\Phi_2)}$ (12)

Therefore the program generates values at each pavement section using random generator and the input data (i.e. the mean value, the standard deviation, ρ_1 and ρ_2). Little information is available on the variance σ^2_x and, above all, on autocorrelations functions of bitumen content, mix void and layer thickness. Values about standard deviation of above mentioned series trough literature data are summarized in Table 5 (Maser, 1990). Values of 0.9 and 0.7 were used respectively for ρ_1 and ρ_2 , as a large degree of autocorrelation was assumed [Ullidtz and Larsen (1983)]

Table 5: Standard deviation of thickness, bitumen content and mix void along the road pavement from literature data (BC=Bituminous Concrete , G=unbound granular material).

	Pavement Layers			
	Wear BC	**Binder BC**	**Base BC**	**Sub-base G**
σ_x of thickness [cm]	0.36	0.41	0.33	0.33
σ_x of bitumen content [%]	0.2	0.3	0.3	
σ_x of mix void [%]	0.3	0.38	0.46	

Asphalt concrete visco-elastic properties

In the simulation model, the visco-elastic properties of undamaged bituminous layers are calculated by Asphalt Institute Method (Witczak and Fonseca 1996, Clyne et al.2003):

$$Log\left|E^*\right| = \delta + \frac{\alpha}{1 + e^{[\beta + \gamma \cdot Log(f)]}} \qquad (13)$$

where

$\left|E^*\right|$ is the asphalt mix complex modulus [MPa],

$$\delta = Log(689) - 0.261 + 0.008225 \cdot p_{200} - 0.00000101 \cdot (p_{200})^2 + 0.00196 \cdot p_4 - 0.03157 \cdot V_a - 0.415 \frac{V_{beff}}{(V_{beff} + V_a)}$$

$$\alpha = Log(689) + 1.87 + 0.002808 \cdot p_4 + 0.0000404 \cdot p_{3/8} - 0.0001786 \cdot (p_{3/8})^2 + 0.0164 \cdot p_{3/4}$$

$$\beta = -0.7425 \cdot Log\left(\frac{\eta}{10^6}\right)$$

γ = - 0.716

η is the bitumen viscosity [10^6 Poise],
f is the load frequency [Hz],
Va is the air voids in the mix by volume [%],
Vbeff is the percent effective bitumen content by volume [%],
$p_{3/4}$ is the percent retained on ¾ in. sieve (19 mm) by total aggregate weight (cumulative) [%],
$p_{3/8}$ is the percent retained on 3/8 in. sieve (9.5 mm) by total aggregate weight (cumulative) [%],
p_4 is the percent retained on No.4 sieve (4.75 mm) by total aggregate weight (cumulative) [%],
p_{200} is the percent passing No.200 sieve (0.075 mm) by total aggregate weight [%],

The effect of temperature and aging on bituminous binder viscosity are considered through the relation:

$$\log\log(\eta) = A + VTS \cdot \log(1.8 \cdot T + 491.68) \qquad (14)$$

where η is the binder viscosity [centipoises], T is the temperature of the layers [°C], and A and VTS are parameters function of bitumen type and aging.

The Poisson coefficient is calculated by the following relationship (Ayres and Witczak 1998):

$|v^*| = 0.5$ for $|E^*| < 70$ [Mpa]
$|v^*| = 0.15$ for $|E^*| > 3450$ [Mpa]
$|v^*| = 1.01872 - 0.12968 \cdot \log(145 \cdot |E^*|)$ for 70 [Mpa] < $|E^*|$ < 3450 [Mpa] (15)

Degradation of asphalt concrete layers is taken into account through the criteria suggested in the “2002 Design Guide” (NCHRP Project 1-37A). Following this criteria, in damage condition, a modified value for the parameter α (i.e. α_d) have to be considered in equation (5):

$$\alpha_d = \alpha * (1-DF) \qquad (16)$$

where

DF is a damage factor represented by a sigmoidal function

$$DF = 1 - \frac{1}{\left[1+\exp(d_1 + d_2 \cdot D)\right]^{d_3}},$$

d_1, d_2, d_3, are costant (p.e. for D=1, DF=0.684, $\alpha_d = 0.316 \cdot \alpha$ and d_1=-7.15 d_2=7.93 and d_3=1.0)

D is cumulated fatigue damage, in this study it was estimated using the Miner’s hypotesis of linear accumulation $D = \sum_{i=1}^{j} \frac{N^i}{N_f^i}$,

N^i is the number of cycles of stress σ_i or strain ε_i,

N_f^i is the number of repetitions, of stress σ_i or strain ε_i, to fatigue cracking,

J is the number of different stress and strain levels (i.e. year number × season number × day time interval × axle type),

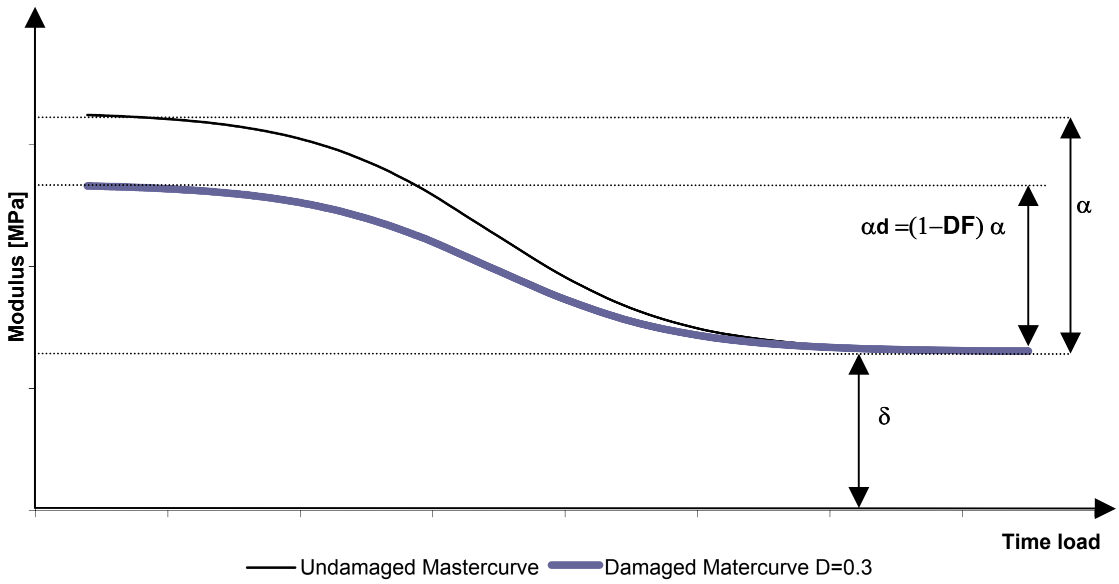

Figure 4: Example of AC Mastercurve in damaged and undamaged condition (diagram in log-log scale)

A lot of fatigue transfer function have been developed so far, which often furnish very different values, in terms of predicted fatigue life Nf. This inconsistency is due to the fact that crack propagation phenomena is difficult to define in road pavement. The fatigue transfer function used in the Italian Catalogue of road pavement was used in the simulation model (see equation 17 and Marchionna (1989)), however other functions could be easily implemented.

$$N_f = 10^{\left(6+4.7619*\left(Log\left(\frac{\Gamma * V_b}{V_b+V_v}\right)-Log\varepsilon_x\right)\right)} + 1.373 * e^{-1.098n} * h^{(-0.157+0.476n)} * \left[\left(\frac{E}{10}\right)^{\alpha'} * \left(\frac{\sigma}{10}\right)^{\beta'} * 10^{\mu'}\right] \quad (17)$$

where

N_f predicted fatigue life (10% fatigue cracking in the wheel path),
ε_x initial tensile strain at the bottom of asphalt layers,
σ_x initial tensile stress at the bottom of asphalt layers [MPa];
V_b volume of binder [%],
V_v air void content [%],
Γ binder binder correction factor (e.g. $1.25*10^{-4}$)
E mean mix stiffness dynamic (complex) modulus [MPa],
h asphalt layers thickness [cm];
n asphalt mix correction factors (e.g. 5),
α' $=\alpha(n/5)$
β' $=\beta(n/5)$
μ' $=(n/5)+0.84(1-n/5)$
α , β and μ experimentally determined coefficient.
h_{AC} is the thickness of asphalt concrete layers,
C laboratory to field adjustment factor.

Asphalt concrete non-resilient behavior (permanent deformation)

The permanent deformation in hot mix asphalt is function of material properties, stress level, number of stress repetitions and temperature. Some model have been proposed until now in order to evaluate permanent deformation in bituminous concrete layers, but few of them can be considered almost entirely mechanistic.

In this work two well known models were taken into consideration: the model suggested by Verstraeten (Verstraeten et. al. 1977) and the model reported in the “2002 Design Guide” (NCHRP 1-37A 2003). As matter of fact these models can be considered nearly mechanistic and they allows the calibration of the parameters, characterizing non resilient behavior of a specific asphalt mix, by standard laboratory test (repeated load triaxial test).

In the “2002 Design Guide” model, stress level, temperature and number of load repetitions are independent variables, while material characteristics are considered through three parameters (a_1 , a_2 and a_3):

$$\varepsilon_p = (\varepsilon_r \cdot k_1) \cdot 10^{a_1} \cdot T^{a_2} \cdot N^{a_3} \quad (18)$$

where

ε_r is the resilient strain,
T is the temperature,
N is the number of load repetitions,
k_1 is function of confining pressure,
a_1, a_2 and a_3 are material dependent .

In the model suggested by Verstraeten et. al., stress level, load time (frequency) and number of load repetitions are independent variables, while material characteristics are considered through |E*| and two parameters (A and b_{ac}) :

$$\varepsilon_{pAC}(N) = A \cdot \frac{(\sigma_1 - \sigma_3)}{|E^*|} \cdot \left(\frac{N}{1000 \cdot f}\right)^{b_{AC}} \quad (19)$$

where:

ε_{pAC} is permanent strain in the bituminous layer,
A and b_{ac} are constants of the bituminous mixture (e.g. A=57.5 and b_{ac} = 0.25 for standard bituminous concrete),
f is the frequency of the loads,
|E*| is the complex modulus of the bituminous concrete of the layers,
σ_1 and σ_3 are the vertical and radial stress.

As the parameters of the latter model are well known, in the simulation performed the Verstraeten model is used.

Unbound granular materials and soil resilient modulus

The isotropic resilient behaviour of unbound granular materials was defined by the resilient modulus "Mr" and Poisson's ratio. Previous works on granular materials behaviour are extensively reviewed in several research reports and will not be mentioned here [COST 337 (2000), Amber et al. (2002), Hornych (1997), Lekarp F. (1999) and Seyhan et al. (2002)]. Two models are suggested in COST 337 for modelling resilient behaviour: the well-known k-θ model and the expanded version of Uzan model:

k-θ model $$Mr = k_1 \cdot (\theta)^{k_2} \qquad (20)$$

expanded Uzan model $$Mr = k_1 \cdot P_a \cdot \left(\frac{\theta}{P_a}\right)^{k_2} \cdot \left(\frac{\tau}{P_a} + 1\right)^{k_3} \qquad (21)$$

where

Mr is the resilient modulus [MPa]

σ_1 is the major principal stress [kPa],

$\sigma_2 = \sigma_3$ is minor principal stress or confining pressure [kPa]

$\tau_{oct} = \frac{1}{3}\sqrt{(\sigma_1 - \sigma_2)^2 + (\sigma_1 - \sigma_3)^2 + (\sigma_2 - \sigma_3)^2}$ is the octahedral shear stress,

Pa is the atmospheric pressure (100 kPa)

k_1, k_2 and k_3 are regression constants.

Both model are implemented in the simulation program, but the expanded version of Uzan model was used in the example. As matter of fact, SHRP –LTTP study found that the Uzan model provide a very good fit to the LTTP test data and developed statistical relationship between physical properties, of granular and soil materials, and regression constants of equation [Amber et al. (2002)]. It should be noted that physical properties correlated to resilient modulus varied between the different materials and soils type. Relevant statistic parameters for coefficients of constitutive equation found in LTTP study are summarized in the following Table 6.

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Table 6: Median and mean values for each coefficient of constitutive equation (21), for base and subbase pavement materials and subgrade soils from LTTP [Amber et al. (2002)].

Coefficient		Material /Soil Group		
		Unbound Base and Subbase Materials	Coarse-Grained Soils	Fine-Grained Soils
k_1	Median	0.853	0.764	0.804
	mean	0.873	0.802	0.896
	Standard deviation	0.2726	0.2661	0.3133
k_2	median	0.628	0.446	0.243
	mean	0.626	0.452	0.282
	Standard deviation	0.1330	0.1927	0.1552
k_3	median	-0.129	-1.052	-1.399
	mean	-0.170	-1.140	-1.576
	Standard deviation	0.2148	0.7365	1.1014

The Poisson ratio of granular materials are generally taken as a constant in pavement analysis, although many researchers have shown that they are stress dependent as well. Nevertheless few stress dependent Poisson's ratio models have been developed so far. Particularly Lekarp introduced the following empirical model developed from variable confining pressure triaxial test [Lekarp (1999)]:

$$\nu_r = a \cdot \left(\frac{q}{p}\right)^2 + b \cdot \left(\frac{q}{p}\right) + c \qquad (22)$$

where

$p = (\sigma_1 + 2\sigma_3)/3$

$q = \sigma_1 - \sigma_3$

σ_1 and σ_2 see equation 21 [kPa],

a, b e c are constants function of maximum particle size (see Table 7).

Table 7: Regression constants value in equation (14) vs. maximum dimension of grain, for a crushed limestone [Lekarp (1999)].

Particle size [mm]	a	b	c	R^2
0/90	0.22	-0.57	0.52	0.97
0/63	0.20	-0.50	0.50	0.93
0/32	0.20	-0.47	0.51	0.96
0/16	0.18	-0.43	0.52	0.96

The above mentioned relationship was used in the simulation process even if a limit value of 0.5 was assumed as materials were considered isotropic.

In the degradation process the resilient modulus and the Poisson's ratio of granular layers and soil support was update every year basing on the stress state of the previous year, according to the following expression:

$$Mr_{y+1,v,s,h} = k_1 \cdot P_a \cdot \left(\frac{\theta_{y,v,s,h}}{P_a}\right)^{k_2} \cdot \left(\frac{\tau_{y,v,s,h}}{P_a} + 1\right)^{k_3} \quad (23)$$

where

$Mr_{y,v,s,h}$, $\theta_{y,v,s,h}$ and $\tau_{y,v,s,h}$ are resilient modulus and stress state parameter in the year y for load v, season s and day time interval h.

As a matter of fact, it has been observed that the stress-strain level variation in the granular layers year by year is extremely low and therefore referring the resilient properties to the previous year stress level does not affect the accuracy of results.

Unbound granular materials and soil permanent deformation

Many researchers showed that accumulated permanent strain, in granular materials, tends to become almost constant at low levels of repeated load ratio σ_1/σ_3 (i.e. no further increase in permanent strain with increasing number of loads), whereas increases rapidly at higher levels of stress ratio. This has raised the hypothesis of the existence of a critical stress level between stable and unstable condition "shakedown limit". According to this hypothesis four categories of granular material response under repeated loading have been introduced: a) purely elastic, b) elastic shakedown, c) plastic creep and d) incremental collapse [Werkmeister (2001)]. In the elastic shakedown the material response is plastic for a finite number of load applications (post-compaction period), after this phase it becomes entirely resilient and no further permanent strains occurs (i.e. permanent strain tends to an asymptotic final value). Materials in plastic creep response show a rapid decrease of plastic strain rate during the first load cycle (like in elastic shakedown post-compaction period) but a small residual incremental plastic strain still remain even after a lot of cycle (i.e. 10^5 cycle). Therefore in range c) material achieves a steady state response after a finite number of stress/strain excursion, and it shows an almost constant level of permanent strain rate (i.e. a near linear rise of permanent strains). In range d) the response is always plastic and each load application result in a progressive increment of permanent strains (i.e. only a small decrease in strain rate can be observed).

Purely elastic behaviour usually does not occur in the pavement granular materials, as a result of post-compaction permanent strain, whereas incremental collapse should not be allowed to occur in a pavement; therefore the b) and c) responses are of interest in these study.

At current state of knowledge no unified constitutive model which should predict the amount of permanent strain at any stress level has been introduced (Werkmeister et al. 2003). At the same time some material laws for permanent deformation behaviour in the elastic shakedown field have been developed so far. Among these laws, "Paute" model (see equation 24) seems to describe quite satisfactory the phenomenon at low levels of shear stress (COST 2000, Lekarp and Dawson, 1998, Gidel et. al., 2001):

$$\varepsilon_p(N) = A \cdot \left(1 - \left(\frac{N}{100}\right)^{-B}\right) + \varepsilon_p(100) \quad (24)$$

where

A is function of stress state, p.e. from Gidel et. al. (2001)

$l_{\max} = \sqrt{p_{\max}^2 + q_{\max}^2}$, $p=(\sigma_1+2\sigma_3)/3$ and $q=\sigma_1-\sigma_3$

σ_1 is the max stress (vertical) in granular layer,

σ_3 is the confining stress (horizontal) in granular layer,

B, ε_{p0}, m, n and s are material constants.

Different authors introduced laws that describe the “plastic creep” response (constant level of permanent strain rate), such as the model introduced by Veverka et. al. (1977) and adopted by Larsen and Ullidtz, (1998) (see equation 25), or the model set up by Zhang W. and Macdonald R. A. (2002) (see equation 26):

$$\varepsilon_p(N) = C \cdot N^{\alpha} \cdot \varepsilon_r^{\beta} \qquad (25)$$

$$\varepsilon_p(N) = C \cdot N^{\alpha} \cdot \left(\frac{\sigma_z}{p_o}\right)^{\gamma} \cdot \varepsilon_r^{\beta} \qquad (26)$$

where

ε_r is the resilient strain,

σ_z is the vertical stress at depth z [kPa],

N is the number of load repetitions,

p_0 is a reference stress (usually taken as atmospheric pressure 100kPa)

C, α, β and γ are material constants (see Table 8).

Table 8: Values of costants C, α, β and γ in equations (16) and (17) suggested by some authors

Equ.	Material description	C	α	β	γ	References
(25)	Granular subbase	2.0	0.30	1.00	NA	Verstraeten et al. (1982)
(25)	50% sand 30% silt and 15% clay	0.0021	0.22	1.37	NA	Larsen, H. J. E. and Ullidtz, P. (1998)).
(26)	Clayey silty sand (AASHTO classification A-4(0))	0.087	0.333	0.333	1.00	Zhang W. and Macdonald R. A. (2002)

Legend: NA = not apply

However it has to be underlined that in addition to the influence of stress state, the state of density and the moisture content also significantly affect the rate of permanent strain of unbound granular pavement material, as it was highlighted in the COURAGE research project (1999) [see also Haynes J. H. and Yoder E. J., (1963) and Bonaquist R. and Witczak M. W. (1996)]. In detail, rapid permanent strain increases were observed once a moisture content of approximately 60% of optimum is reached (about 10 times increase from 60% to 80% of optimal moisture content). This phenomena could be the cause of the rapid increase of roughness, observed in experimental tests, when high level of deterioration is reached (i.e. extended cracking in bituminous layers).

Basing on such premises, it appears clear that further studies are necessary to accurately model the granular material behaviour, as regards:

- Stress boundary between the different response behaviours (b) and d)) previously reported,
- Influence of materials characteristics (i.e. aggregate type, grading etc.)
- Influence of stress on permanent deformation
- Influence of moisture content on permanent deformation.

In the simulation model, set up in this study, both equations (15) and (16) were implemented and their effects on roughness were evaluated in the case study performed.

Pavement response model

Data from climatic conditions, dynamic loads, material properties are implemented in a pavement primary response model for the evaluation of the stress and strain level. A layered linear elastic system has been assumed as pavement model, taking into account the interaction between the layers. The modulus “K” of the elastic constrain between layers was function of temperature and interface normal stress, according to Uzan et al. (1978) and Crispino et al. (1997).

CASE STUDY

A case study was analysed in order to test the simulation model developed and to evaluate the effects of different discretization of temperature periods and of different permanent deformation models.

The road pavement analysed was a flexible one, and it is recommended by Italian Catalogue (CNR 1995) for two lane highway and high traffic volumes (see Table 9). The climatic data of south Italy were used and simulations were performed considering both mean daily temperature of the layers and mean temperature during three hours periods, in order to compare the results obtained through the two approach. The road section (150 m length) was divided in about 500 sub-sections (0.30 m length) and the starting profile generated had a roughness IRI $\approx$ 1.0 m/km. In the simulation the road profile was updated every three months. The simulation is ended, in most of cases, when a nearly unit cumulated damage value according to Miner’s law is reached.

Table 9: Data considered in the test of the simulation model

	Pavement	Layers	Thickness	[mm]
Surface corse (BC)	**Binder Course (BC)**	**Base Course (BC)**	**1° Subgrade layer**	**2° Subgrade layer**
50	60	130	200	∞
	Traffic		**Data**	
Road Type	**Heavy vehicle Speed**	**AADT [veh./day]**	**Annual increase [%]**	**% of heavy vehicle**
Main Two Lane Rural Road	80 (Km/h)	15000	1.0	15

Both granular permanent deformation models (see unbound granular materials and soil permanent deformation paragraph) were used in the case study and their effects on roughness evolution were analysed. The deterioration curves of IRI, mean and standard deviation of Rut Depth (MRD, SDRD) are conveniently depicted in the following figures (Figure 5 Figure 6 and Figure 7)
The results showed that:

- IRI evolution is dramatically influenced by model used to evaluate the permanent deformation in granular materials; in detail, model expressed by eq. 24 and eq. 25 yield a slight increase of profile roughness within the service life of the pavement examined; it has to be preliminary observed that model parameters used in the simulation have been derived by granular materials with an high coarse aggregate content and therefore may not match with conventional granular materials encountered in highway construction according to common specifications; however, the terminal value of IRI that, according to the relationships reported below, may correspond, in the worst case, to a PSI value of 3.7-3.2 and therefore well above the conventional PSI terminal value, seems to indicate that, unless additional degradation phenomena (such as the change of moisture content induced by percolation of water through pavement cracks) are considered, these models may be not able to fully capture the roughness evolution that is usually experienced in pavements;
- The maximum value of IRI obtained at the end of pavement life (i.e. cumulative fatigue damage ≈1, in most of cases, see Figure 10), equal to 2.9 m/km, agree well with roughness estimated by AASHTO design method ($PSI \approx 5 \cdot e^{-0.26 \cdot IRI} = 2.35$ or $PSI \approx 5 \cdot e^{-\left(\frac{IRI}{5.5}\right)} = 2.95$ see Smith K.L et al. 1997);
- Permanent deformation in the subgrade represents about 31÷56 % of the total mean rut depth at the end of pavement life (see Figure 8 and Figure 9).

The initial and terminal surface profiles and the cumulated fatigue damage along the road section, obtained by the simulation framework according to the permanent deformation model described by equation 26 , are shown in Figure 11. As it can be noticed from this figure, maximum cumulated fatigue damage is reached where significant local deformation is experienced (sections at 4, 8, 21, 32 m).

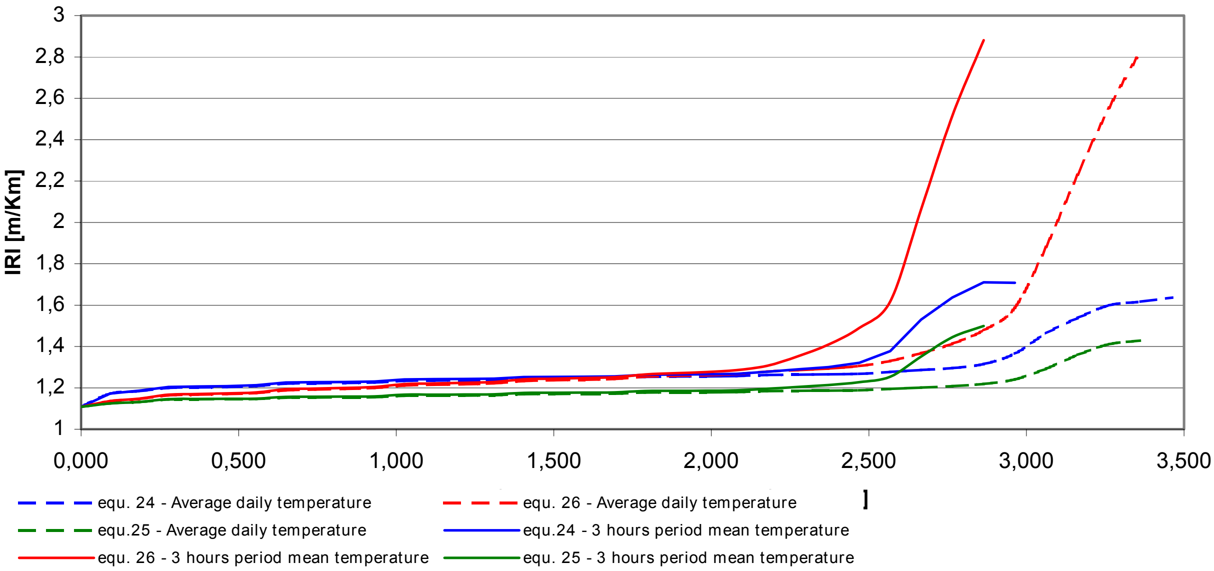

Figure 5 - IRI vs vehicle passes employing different permanent deformation models.

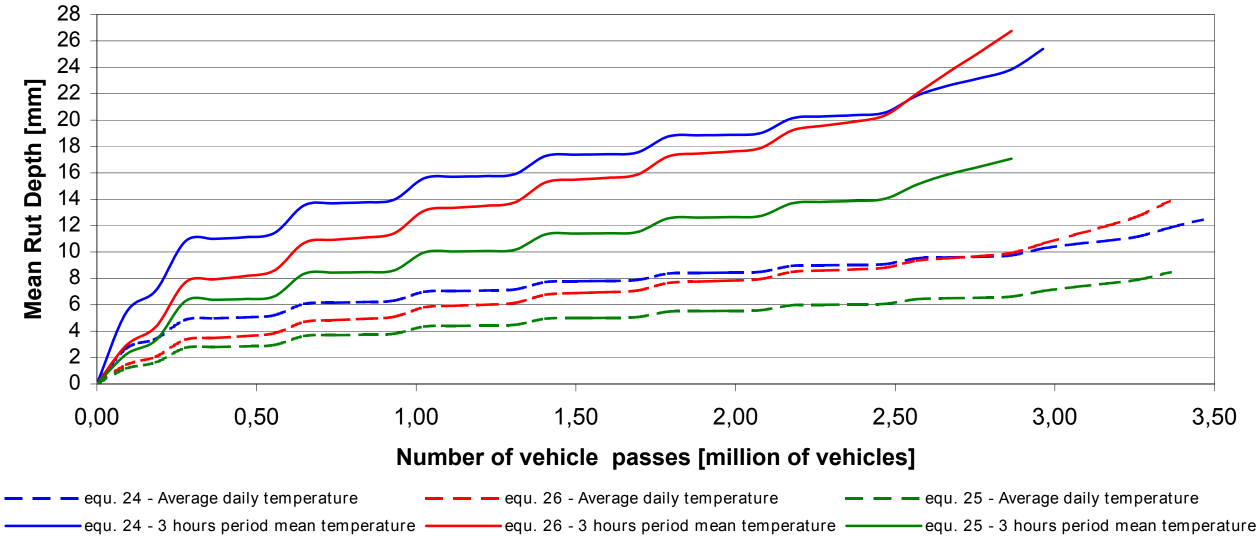

Figure 6 - RMD vs vehicle passes employing different permanent deformation models.

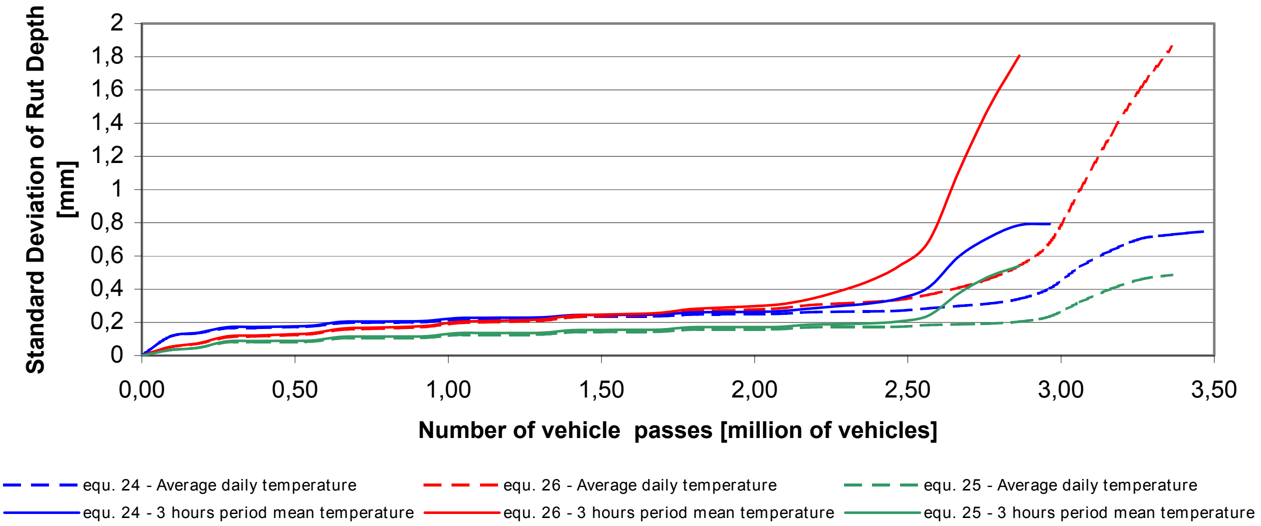

Figure 7– Standard deviation of rut depth vs vehicle passes employing different permanent deformation models.

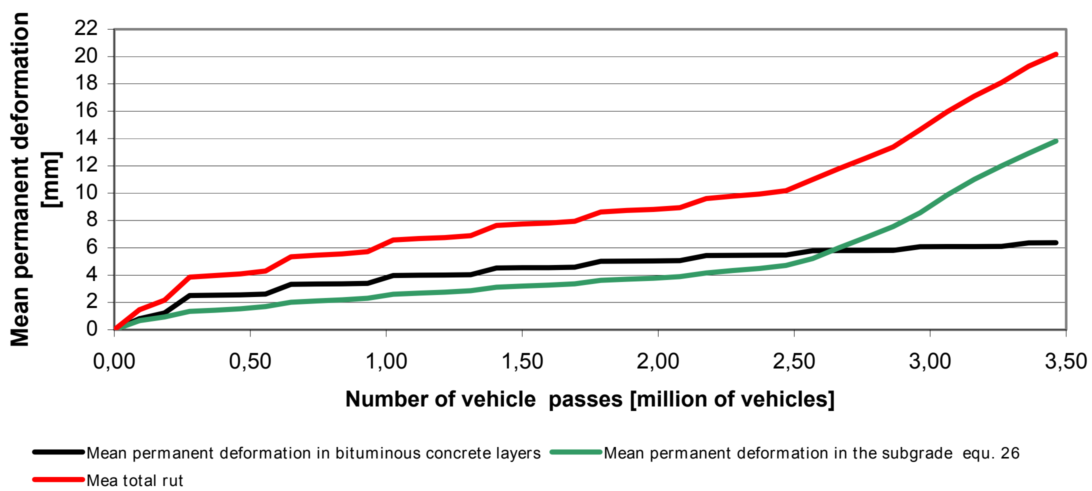

Figure 8- Mean permanent deformation in bituminous concrete layers and in the subgrade vs vehicle passes according to the permanent deformation model in equ. 26.

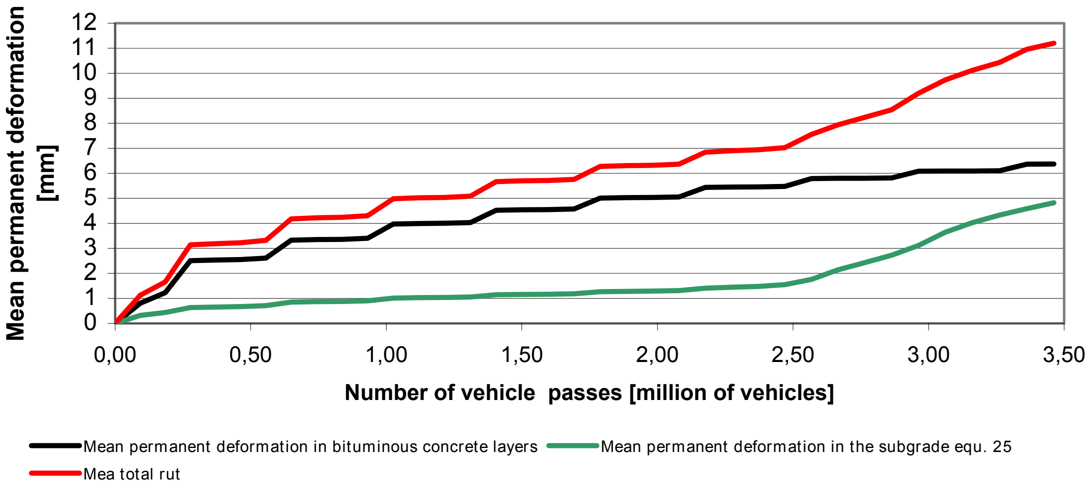

Figure 9- Mean permanent deformation in bituminous concrete layers and in the subgrade vs vehicle passes according to the permanent deformation model in equ. 25.

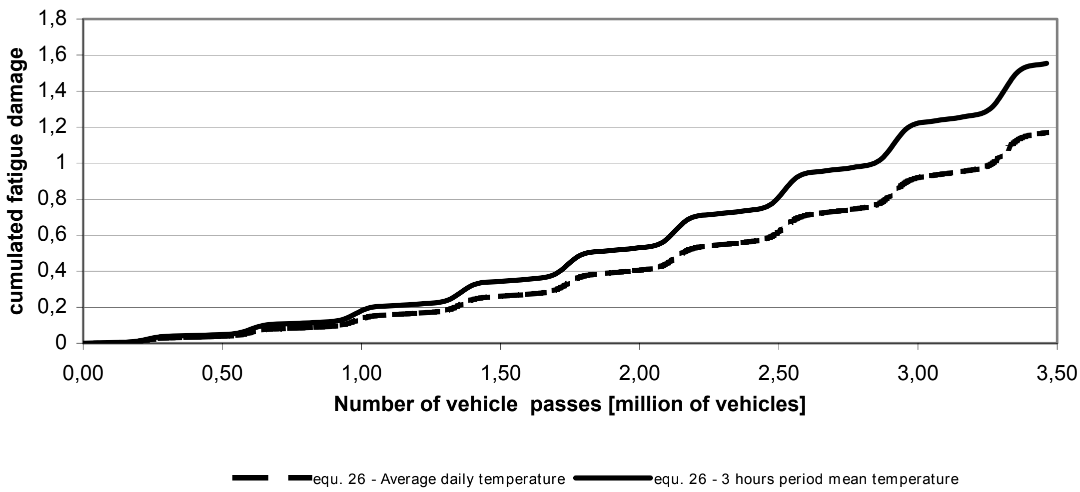

Figure 10 – Mean cumulated fatigue damage vs vehicle passes.

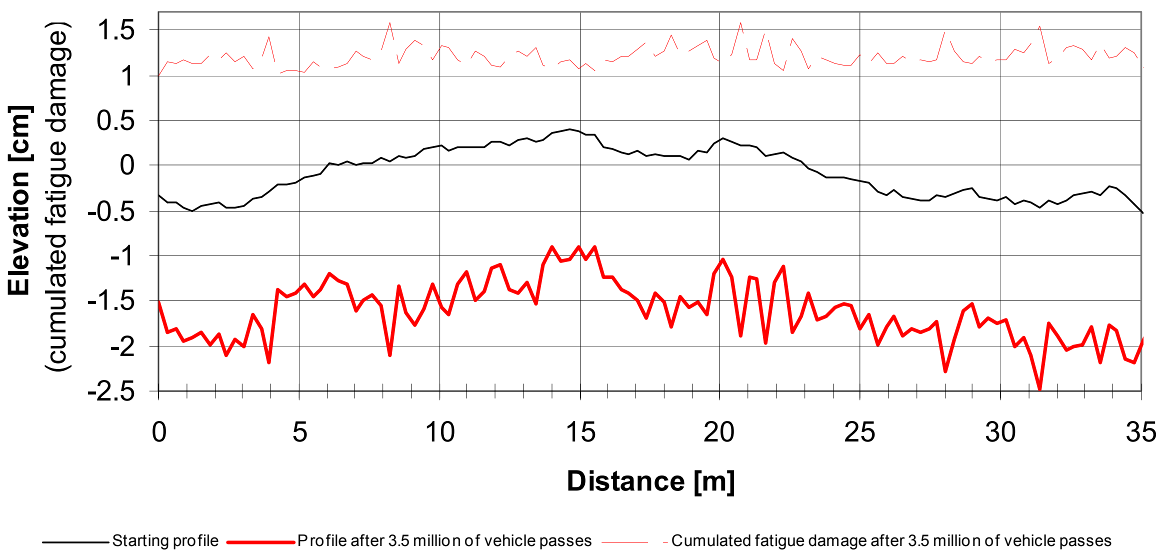

Figure 11 - Example (35 m window).of profile evolution calculated by simulation model (permanent deformation model equ.26).

CONCLUSION

The objective of this study was to show that deterioration models can be developed by a mechanistic approach. A simulation model that is able to derive simple deterioration laws of roughness and rutting for a specific road pavement structure in any climatic condition (provided that material properties and heavy vehicle traffic data are known) has been presented.

The roughness and rutting progression curves obtained could be easily implemented in the PMS instead of more generic empirical deterioration models.

Results from preliminary case studies seems to indicate that

- The daily temperature trends play an important role in the evolution surface roughness and rutting in flexible pavements;
- Permanent deformation laws in granular layers are key component to properly describe the roughness and rut evolution.

Further improvements of the model framework could be:

- Implement a more accurate model to simulate effects of variations in moisture content related to mean number of days of rainfall;
- Examine the application of the model to rehabilitated pavement (e.g. mill and replace, overlay, etc.) in order to evaluate rut and roughness evolution after maintenance works.

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