Politecnico di Torino

Master's Degree in Civil Engineering

Analysis and design of Perpetual Asphalt Pavements

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1. Introduction

Perpetual pavement is a relatively recent concept, introduced in 2000 by the Asphalt Pavement Alliance (APA). They defined a Perpetual Pavement as "an asphalt pavement designed and built to last longer than 50 years without requiring major structural rehabilitation or reconstruction and needing only periodic surface renewal in response to distresses confined to the top of the pavement" *(APA, 2002)*. However, the "idea" of a perpetual pavement is not a fully recent concept, but during the time a lot of engineers tried to design and develop a long-life pavement. In fact, starting from 1950s several full depth and deep strength asphalt pavements have been designed, with the aim to sustain for 20/25 years with minimum maintenance and rehabilitation activities. Initially, they were developed using very empirical approaches, considering the results and observations coming from the evaluation of the response of different pavements, in terms of stress, strain and displacements, under different traffic loading conditions.

So, the goal of the use of full depth and deep strength pavements was to reduce the structural damages, trying to maintain the level of the solicitations under certain limits.

This type of approach has a lot of advantages as:

- Low life-cycle cost by avoiding deep pavement repairs or reconstruction
- Low user-delay costs since minor surface rehabilitation of asphalt pavements only requires short work windows that can avoid peak traffic hours
- Low environmental impact by reducing the amount of material resources over the pavement's life and recycling any materials removed from the pavement surface

Clearly, using this type of approach the possible risk is to overdesign the pavement, because there are limits after which the increase of the thickness doesn't give any structural improvements, and this will cause an unjustified economic expenditure.
In this way, the concept of perpetual pavements was developed with the idea to keep the advantages coming from the long-life pavement, and, at the same time, improving the methodology, avoiding possible economic risks.

The perpetual pavement design is based on empirical-mechanistic approach. This allows us to merge the mechanistic approach, characterised by rigorous models, with information and results coming from experience.

The general structure consists of three HMA layers designed to resist a specific distress type. The main idea is to construct asphalt pavements with an impermeable rut and wear resistant top structural layer placed on a rut resistant and durable intermediate layer and a fatigue resistant and durable base layer (*Romanoschi et al., 2008*).

As in conventional asphalt pavement, in the design we consider two main distress phenomena: fatigue cracking and rutting. This is the reason why we are focusing on two critical strains: the maximum tensile strain at the bottom of the asphalt layer and the maximum vertical strain at the top of the subgrade. After putting various design parameters into the analytical model, outputs are than transformed into accumulate damage in terms of bottom up fatigue cracking and structural rutting.

These responses are used in transfer functions to predict the life of the pavement through theoretical horizontal strain at the bottom of the HMA and vertical compressive strain at the top of the subgrade.

If these two strain values are accurately estimated and maintained under specific strain limits, for both distresses (fatigue cracking & structural rutting), the pavement will act as a perpetual pavement, and this means that the damage under cycling load is not cumulating into the pavement, preserving in this way the structural capacity of the pavement. So, during the years no major structural damage will occur and just a periodic surface rehabilitation of asphalt pavements is required.

The purpose of this thesis work is to analyse in the first part the state of the art of Perpetual Asphalt Pavements, going to define what are the limiting responses so that we can define a pavement as perpetual, characterizing also the layered structure and the materials. The central part focuses on the design method, and in particular on the Mechanistic-Empirical design method with the presence of limiting responses. Moreover, in this part a comparison of the available software used for the design of PP has been done, which compared to the software used for the design of traditional flexible pavement allow to highlight the peculiarities of a Perpetual Pavement.
Lastly, in the third part, two different case study have been analysed; the first is a case study in Texas, which has been analysed with the aim to compare the results coming from the three different software. The second one is a case study in Italy. In this last case, a traditional flexible pavement structure was compared with different perpetual pavement solutions, thus comparing the increases / decreases in the thickness of the bituminous layers and the related structural improvements.

2. History of perpetual pavements

As already mentioned, since 1950s/1960s the first long-life asphalt pavements have been designed. Many of these pavements in the past forty years were the products of full-depth or deep-strength asphalt pavement designs, and both have design philosophies that have been shown to provide adequate strength over extended life cycles *(APA, 2002)*.

Deep strength asphalt pavement is a pavement constructed of asphalt for the surface and base, placed on a granular or stabilised subbase, instead, a full-depth asphalt is a pavement in which asphalt is used for all courses above the subgrade or improved subgrade and it is laid directly on the prepared subgrade.

It is significant that these pavements have endured an unprecedented amount of traffic growth. For instance, from 1970 to 1998, the average daily ton-miles of freight increased by 580 percent, and the average freight loading continues to increase 2.7 percent per year *(D'Angelo et al., 2004)*.

So, it is evident that at the beginning the idea of perpetual was unconsciously inside of these well-designed and well-constructed pavements, until the 2000s where there was a very huge increase of interest upon it. Starting from this point a lot of researches and tests have been performed in order to improve the design method of the perpetual pavement.

Typical examples of this kind of "actions and activities" are:

- The International Society for Asphalt Pavements dedicated a special session to Perpetual Pavements in 2002.
- Three international conferences have been held on the topic, one at Auburn University in 2004 and the others at Ohio University in 2006 and 2009.
- The Transportation Research Board held a workshop session on Perpetual Pavements in 2001
- The Federation of European Highway and Road Laboratories (FEH RL) has undertaken a series of efforts to define long-life pavements *(Ferne and Nunn, 2004; Ferne, 2006)*.
- Three major national studies on Perpetual Pavements were initiated through the National Cooperative Highway Research Program (NCHRP).
- State studies on Perpetual Pavements have been or are currently being conducted in Kansas *(Romanoschi et al., 2006)*, Ohio *(Sargand et al., 2006)*, Wisconsin *(Crovetti et al., 2008)*, Pennsylvania *(Solaimanian et al., 2006)*, Oklahoma *(Gierhart, 2008)*, Texas *(Scullion, 2006)*, Michigan *(Von Quintus, 2001b; Von Quintus and Tam, 2001)*, New Mexico *(TRB, 2009)*, Illinois *(Thomson and Carpenter, 2004)*, Washington *(Mahoney, 2001)*, and California *(Monismith et al., 2009)*.
- Perpetual Pavement design workshops have been held in Ohio, Kansas, Oregon, Colorado, Texas, Minnesota, Tennessee, Georgia, Hawaii, Wisconsin, Oklahoma, and Indiana.
- The National Center for Asphalt Technology (NCAT) Test Track has pavement test sections designed as Perpetual Pavements which are instrumented to validate the design concepts.
- Two pavement design computer programs specifically for Perpetual Pavements have been developed at Auburn University.

- The concept of the endurance limit has been incorporated in the new American Association of State Highway and Transportation Officials (AASHTO) Mechanistic-Empirical Pavement Design Guide (MEPDG) (AASHTO, 2008).

Although the Perpetual Pavement concept was first articulated in 2000, many asphalt pavements constructed long ago function as Perpetual Pavements. The Asphalt Pavement Alliance instituted the Perpetual Pavement Award program in 2001 to recognize state agencies and other owners of pavements that had the foresight to build pavements according to these principles.

Since 2001, the APA's Perpetual Pavement Award program has recognized 144 long-life pavements in 31 U.S. states and one Canadian province. These roads were all at least 35 years old when honoured and had never experienced a structural failure. To qualify, a road could not have had more than 4 inches of new material added over the previous 35 years, and it could not have been resurfaced more frequently than once every 13 years. The winning pavements range in age from 35 years to 99 years, and the average age was 45.4 years at the time the award was won (http://www.asphaltroads.org).

Here, is a list of roads that have won the Perpetual Pavement award from 2001 to 2007 is reported:

- *148-mile New Jersey Turnpike, New Jersey*
- *Interstate 40, Caddo County, Oklahoma*
- *Interstate 90, Washington*
- *Interstate 65, Marshall County, Tennessee*
- *Interstate 35, Pine County*
- *Interstate 80, Lowa*
- *Eareckson Air Station, Runway 10-28, Alaska*
- *Don Valley, Canada*
- *State Route 73, Ohio*
- *Garden State Parkway, New Jersey*
- *US 20, Holt County, Nebraska*
- *US 63, Texas County, Missouri*
- *Trunk Highway 71, Hubbard County, Minnesota*
- *Interstate 17, Milepost 256 to 261, Arizona*
- *Central Expressway, Santa Clara, California*
- *Julian Carroll-Jackson Purchase Parkway, Kentucky*
- *Interstate 26, Spartanburg County, South Carolina*
- *Interstate 180, Bureau County, Illinois*
- *Interstate 181, Mile Marker 11.56 to 19.71, Tennessee*
- *US 85, Platte River Valley, Colorado*
- *State Highway 173, Bandera County, Texas*
- *Ranch to Market 2828, Bandera County, Taxas*
- *Interstate 90, Laurel to Billings, Montana*
- *Interstate 24, Coffee County, Tennesse*

- *Interstate 20, Aiken County, South Carolina*
- *Interstate 59, Lauderdale County, Missisipi*
- *Interstate 95, Cecil County, Maryland*
- *Interstate 95 Between Greenwich and North Stonington, Connecticut*
- *San Diego Freeway, Interstate 405, Caifornia*
- *Trunk Highway 61, Milepost 53 to 61, Minnesota*
- *State Highway 35, Mile Marker 21.68 to 26.87, Nebraska*
- *State Route 14, Log Mile 2.42 to 16.54, Tennessee*
- *Interstate 81, Mile Posts 318.4 to 324.9, Virginia*
- *u.s. Highway 41, Wisconsin*
- *State Route 11, Tennessee*
- *Interstate 20, South Carolina*
- *US 30 East from Wayne/Stark County Line, Ohio*
- *Interstate 15, Montana*
- *US 78 Desoto County (Future I-22), Mississippi*
- *U.S. Route 54 Camden County, Missouri*
- *U.S. Trunk Highway 71, Minnesota*
- *Lapeer – Caro Road MS-24, Tuscola County, Michigan*
- *US 60 Ashland – Cannonsburg Road, Kentucky*
- *Route 82 Haddam County, Connecticut*

Starting from 2000s, in different countries all around the world, several projects/researches started to be developed.

Three perpetual pavement test sections, along with two control sections, were built during the summer of 2005 as part of a perpetual pavement experiment on a newly constructed expressway in Shandong Province, China. A conservative fatigue threshold of 70 με resulted in the first test section, a 20 in. (500 mm) full-depth asphalt pavement. The second test section, a 15 in (380 mm) full-depth pavement, used a less conservative threshold of 125 με. The third test section duplicated the second, but with a higher performance graded binder in the bottom 3 in. (75 mm) lift. The two remaining sections are representative of typical expressway design in China: thin HMA layers on a pozzolanic-treated base. One section has 13 in (330 mm) of HMA on 16 in. (400 mm) of limeflyash treated base, and the other section, more typical of pavements in Shandong Province, has 6 in. (150 mm) of HMA on 21.5 in. (560 mm) of lime-flyash treated base. The expressway opened to traffic in December 2005 and experiments with control vehicles and live traffic are currently underway. The testing protocol is described, and representative response data collected in December 2005 are also presented (*Yongshun Yang e. al*.,*2006*).

In 2002, Michigan decided to join a growing list of states utilizing the Perpetual Pavement concept for high-volume roadways. The Michigan Asphalt Paving Association (MAPA) approached the MDOT about constructing a Perpetual Pavement demonstration project. A section of northbound US-24 in Detroit was selected as demonstration location. The project occurred on a 1.18 mile stretch of northbound US-24, which is a seven to eight lane boulevard along this section, in Detroit, with a posted speed limit of 45 miles per hour. The pavement structure was removed down to the original subgrade and then rebuilt. The new cross-section consists of 10 inches of asphalt over 12 inches of aggregate base over 14 inches of sand subbase. A geotextile separator was included between the base and the subbase, and 6 inches undrained was trenched in for drainage. Two types of subgrade exist on the project: a sand and clay. A 492-foot (150 m) test section was set up for each of the two different subgrades. In-place soil and asphalt properties were tested in the test section for each layer (*M.J.Eacker et al.)*.

In 2003, WisDOT constructed two perpetual pavement test sections on the entrance ramp to I-94 from the Kenosha Safety and Weigh Station Facility in Southeastern WI. Test section 1 (TS1) HMA layers were constructed as follows: 2-in surface layer (PG 76-28, 6% air voids); 4.5-in middle layer (PG 70-22, 6% air voids); 4.5-in lower layer (PG 64-22, 4% air voids). Test section 2 HMA layers were constructed as follows: 2-in surface layer (PG 70-28, 6% air voids); 4.5-in middle layer (PG 70-22, 6% air voids); 4.5-in lower layer (PG 64-22, 6% air voids). The test sections were subjected to nearly 100% truck traffic with a projected 75 million ESALs over 20 years. After seven years in service, premature longitudinal and alligator cracking was present in the wheel paths of both test sections, with TS1 displaying a slightly higher level of distress. No rutting was observed in either test section. Forensic coring showed that the cracking was top-down. The early distresses were likely due to segregation and over-compaction that occurred during construction.

Strain induced by trucks with known loads was measured using strain gages installed during construction. Strain at the bottom of the HMA pavement was typically lower than 70x10-6, the currently-accepted HMA fatigue endurance limit. Strains up to 100x10-6 were measured with high axle loads (47 kips), slow travel speeds (32 mph) and high pavement temperatures (90-103°F).

The perpetual pavement performance was acceptable overall. Distresses were limited to the surface HMA layer, which can be milled and replaced without affecting the lower layers. Strains were low at the bottom of the HMA pavement, indicating that the pavement system adequately resisted fatigue damage (*Irene K. Battaglia et al., 2010*).

Since 2001, the State of Texas has been designing and constructing perpetual pavements on some of its heavily trafficked highways where the expected 20-year truck-traffic estimate of 18-kip ESALs is in excess of 30 million. To date, there are 10 in-service perpetual pavement (PP) sections, typically consisting of about 22 inches total thickness of HMA layers and supported on an 8-inch thick treated (lime or cement) base, resting on a well compacted subgrade soil.

3. Limiting Perpetual Pavement Responses

The typical approaches to Perpetual Pavement design focus on pavement responses related to structural rutting and bottom-up fatigue cracking. So, some critical thresholds need to be defined, in terms of strain, below which structural damage does not cumulate, and it can be considered equal to zero.

3.1 Fatigue cracking and Fatigue Endurance Limit (FEL)

The so-called bottom-up fatigue cracking is a typical mechanism that starting from the bottom asphalt layer it may propagate to the surface affecting all the layers of the pavement structure. The problem related to the crack propagation is the water inflow phenomena, that can travel thought the cracks until to reach the unbound material layers, changing the proprieties and causing also structural damages due to action of the temperature.

Fatigue cracking typically begins due to high repeated strains at the bottom of an asphalt layer from heavy loads (*Huang, 1993*).

In the following *figure n. 1* we can see a very schematic scheme of fatigue cracking.

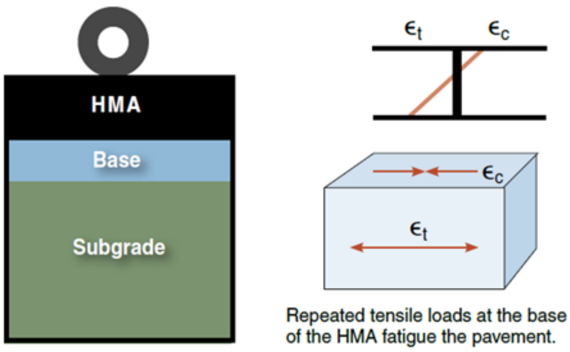


Figure n. 1 – Fatigue Cracking synthetic scheme

This failure criterion has been studied widely and there are several approaches. The typical one is to perform laboratory test using standard AASHTO four-point bending beam fatigue tests, and consider the tensile strain to predict fatigue life, as shown in *equation n.1*:

$$Nf = K1\ (1/\varepsilon)\ ^\wedge\ K2 \qquad (1)$$

- Nf is the number of load cycles to failure
- ε is the flexural tensile strain in the beam
- K1 and K2 are the fatigue coefficients (slope and intercept)

The fatigue coefficients K1 and K2 vary with changes in material property, pulse duration, rest periods between loads, and other parameters.

One way to decrease the probability of bottom-up fatigue cracking is to increase the thickness of the pavement structure. Thick pavements have been shown to limit cracking to the surface of pavements by reducing the maximum strain at the bottom of the asphalt pavement (*APA, 2002; Merrill et al., 2006; Romanoshci, 2008; Al-Qadi et al, 2008; Newcomb et al., 2000; St. Martin et al., 2001*).

However, the phenomenological nature of this model provides no unique relationship between fatigue life and tensile strain, and this is a limitations, in particular if we want to evaluate the behaviour at low strain condition, that is an important mechanism for extended life (or perpetual) hot-mix asphalt pavements. In fact, there is a limit below which asphalt mixtures tend to have almost unlimited fatigue life, the so-called Fatigue Endurance Limit (FEL). One of the firsts to propose this concept was Wöhler for metallic materials. In the experiments done by Wöhler there was a clear a tendency for material to have infinite fatigue life at the fatigue endurance limit.

Of course, this phenomenon was widely studied for what concerns the metallic material, but we cannot say the same for HAM, also because it is a very complex material, time-temperature dependent, and also complex from a chemical point of view. With the increasing emphasis on Extended Life Hot Mix Asphalt Pavements (ELHMAP), or perpetual pavements, the verification of the existence of this endurance limit, a strain below which none or very little fatigue damage develops, has become a substantial consideration in the design of these new multi-layered full depth pavements.

The criterion showed before, based on the equation, do not establish or provide support for the existence of a fatigue endurance limit, a concept that has been postulated for a considerable time.

An allowable load repetition for fatigue damage is typically defined using a fatigue failure criterion of 50% loss of the initial flexural stiffness of HMA under a cyclic loading condition *(Monismith and Deacon 1969*). However, noticeable macro damage or crack initiation during repeated loading cycles may not be clearly explained with the 50% of initial stiffness-loss criterion, because it does not include conceptual or physical parameters to explain the threshold between the elastic range without any cracking and the plastic behaviour, where irreversible cracking is initiated during a relatively small number of loading cycles.

To use the 50% stiffness reduction failure criterion is only proportionally shifting the true failure point, which results in time savings in laboratory testing. Some researchers studied a correlation between the 50% stiffness reduction failure and the true failure based on the dissipated energy analysis. There are different approaches, such as initial dissipated energy, the total dissipated energy, or simply the dissipated energy versus load cycle curve (*Carpenter et al., 2005*).

Rowe obtained good results by using the rate of change in dissipated energy to indicate fatigue failure. However, the rate of change in dissipated energy by itself does not provide for a single unified method to examine failure in different test modes. To overcome that difficulty, Carpenter and Jansen suggested using the change in dissipated energy to relate damage accumulation and fatigue life. That work was refined and expanded by Ghuzlan and Carpenter, and a detailed dissipated energy ratio analysis was further developed by Carpenter, Ghuzlan, and Shen. That approach defines the RDEC as a ratio of the change in dissipated energy between two cycles divided by the dissipated energy of the first cycle, represented as follows:

$$\mathrm{RDEC} = \frac{(\mathrm{DE}_{n+1} - \mathrm{DE}_n)}{\mathrm{DE}_n}$$

Where:

- RDEC = ratio of dissipated energy change,
- DEn = dissipated energy produced in load cycle n, and
- DEn+1 = dissipated energy produced in load cycle n + 1.

This ratio provides a true indication of the damage being done to the mixture from one cycle to another as a function of how much dissipated energy was involved in the previous cycle. By using that approach, the percent of input dissipated energy that goes into damage for a cycle can be directly determined during the fatigue test.

As introduced by Carpenter et al., the damage curve represented by RDEC versus loading cycles can be distinctively divided into three stages, as it is showed in *figure n. 2*. The portion of interest here is Stage II, the so-called Plateau Value (PV), in which the RDEC is almost constant until the dramatic increase in Stage III, which is the onset of true failure.

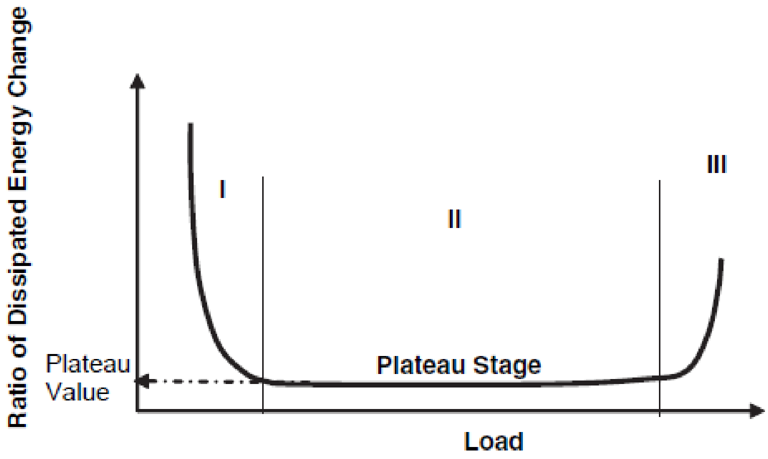


Figure n. 2 – Typical dissipated energy ratio plot with three behaviour of zones (Shen and Carpenter, 2003)

The plateau value (PV), the nearly constant value of RDEC, characterizes a period in which there is a constant percent of input energy being turned into damage. This value appears to be mixture and load strain input related. For any one mixture, PV is a function of the load inputs, and for similar load inputs, PV varies with mixture type. This value is significant because it provides a unique relationship with fatigue life for different mixtures, loading modes, and loading levels.

The dissipated energy ratio procedure provides an easy mechanistic means of examining the energy handling capability of a mixture as it relates to fatigue behaviour in a manner not possible with traditional methods, especially important when testing at low strain levels. Flexural fatigue testing at low strain levels is problematic because it is not possible to devote sufficient machine time to take every sample to failure. Thus, previous research demonstrated the existence of a fatigue endurance limit at low strains. Depending on the mixture and binder types, this low strain level can range from 70 to 500 microstrains. The fatigue life of tested samples at these low strain levels is extremely long, and the 50% stiffness reduction points are never reached in testing time as great as 48 million repetitions. Load repetitions to fatigue must be extrapolated for such long-life, low-damage fatigue testing (*Shen and Carpenter, 2003*).

As shown, the PV is a unique parameter for damage and failure regardless of test parameters such as mixture types, loading modes, frequency, and rest periods. It is unique at both normal and low strain–damage levels. The uniqueness of the PV allows it to be used to predict long fatigue life without running tests to failure. Laboratory tests and statistical analysis suggest that there is no significant difference between the PV predicted from shortened load repetitions and extended load repetitions as long as the sample has reached its plateau stage. To ensure that such a stable plateau stage is reached, 500,000-cycle load repetitions are sufficient.

At normal strain levels the change in the rate of damage accumulation is proportional to the change in the strain level. However, at low strain levels a non-linearity is introduced. At low strain levels there is a decidedly reduced amount of damage being done per cycle compared to normal strain level testing. The data support a gradual trend toward the endurance limit, and not a distinct break point. The recognition of healing and the resultant property changes it produces in a mixture can be proposed to explain this observed non-linearity, lending credence to a physical rationale for a fatigue endurance limit.

Healing is a continual process that can be thought of as a process that returns energy into the HMA, increasing the load carrying ability of the mixture, and in effect repairing a portion of the damage done by the previous loads. Healing becomes most evident when a rest period is imposed between load cycles and the healing can be seen in the increased modulus after the rest period. In actuality this repair process is continual and occurs to some extent even during load cycling. At high strain levels the amount of healing energy is relatively small in relation to the damage energy, but at low strain levels the proportion of damage energy is smaller and could approach the energy returned to the mixture by the healing process. Given that an asphalt aggregate combination produces a specific amount of healing potential, there could exist a strain level at which the damage energy was equal to the healing energy, and no damage would accumulate if the load cycles were slow enough and total healing was allowed to occur. Even if loading was continual, there would exist a point at which the kinetics of the healing process would offset the load cycle damage, and little or no damage would accumulate in the HMA, producing an extended fatigue life, a fatigue endurance limit

So, the data generated from the mechanistic analysis of damage accumulation through the dissipated energy approach clearly provides support for the existence of a fatigue endurance limit. The normal and low strain data can be considered as two distinctly different processes that can be represented by their individual fatigue curves as related to tensile strain, something that cannot be substantiated from the traditional analysis.

Although the data set is currently limited to low strain testing at 70 micro strain, the data shows that the trend is toward an extraordinarily extended fatigue life. While the change may be a continuous function rather than a precise lower limit it would appear that an asymptote is being approached at 70 micro strain. The exact limit is very likely mixture/binder specific. Whether or not the 70 micro strain level is accepted as an endurance limit it is apparent that this level is capable of providing a significantly longer fatigue life than would be predicted from normal testing. For practical design considerations this could be considered a limit beyond which life extension becomes extremely long in comparison to traditional designs and load repetitions used (*Shen and Carpenter, 2003*).

3.2 Structural rutting

The other main distress phenomenon that is considered in the design approach is the structural rutting. This is a surface depression in the wheel path. Pavement uplift (shearing) may occur along the sides of the rut. There are two basic types of rutting: mix rutting and subgrade rutting. Mix rutting occurs when the subgrade does not rut yet and the pavement surface exhibits wheel path depressions as a result of compaction/mix design problems. Subgrade rutting is a structural rutting, and it occurs when the subgrade exhibits wheel path depressions due to loading. In this case, the pavement settles into the subgrade ruts causing surface depressions in the wheel path.

The surface rutting is confined to the upper few inches of the pavement and can be remedied with removal and replacement of the pavement surface. On the other case maybe not enough a partial treatment or in the worst case we must perform structural operations or reconstructions.

The permanent deformation in any of a pavement's layers or subgrade usually is caused by consolidation or lateral movement of the materials due to traffic loading. More in specific it can be due to insufficient compaction of HMA layers during construction. If it is not compacted enough initially, HMA pavement may continue to densify under traffic loads. It can be also related to the subgrade rutting (e.g., as a result of inadequate pavement structure) or improper mix design or manufacture (e.g., excessively high asphalt content, excessive mineral filler, insufficient amount of angular aggregate particles).

As we mentioned before, in this approach for rutting we will consider the maximum vertical strain at the top of the subgrade. Also, in this case, several researches have been done during the time. One possible approach has been proposed by Harvey et al. (2004) and Walubita et al. (2008) who considered the vertical compressive strain at the top of the subgrade as the limiting design parameter. Their approach was to use a value of 200 micro strain as the limiting strain for the subgrade criterion. It was reasoned that plastic deformation in the lower layers would not occur if the compressive strain in the subgrade was kept below this value. This is achieved by increasing either the thickness of the total pavement structure or the stiffness of one or more of the pavement layers.

A different approach was proposed by researchers at the University of Illinois (*Bejarano et al., 1999; Bejarano and Thompson, 2001*). They used the ratio of the subgrade stress to the unconfined compressive strength of the soil, known as the Subgrade Stress Ratio (SSR). They noted that for clay soils in their study, the transition from a stable to an unstable condition occurred when the SSR was in the range of 0.50 to 0.60. For design purposes, they recommend using an SS R of 0.42, although they acknowledge that this rutting criterion is not well established. However, this approach allows the designer to account for the strength of the subgrade in determining the limiting response.

4. Layered structure and materials of Perpetual Pavement

4.1 General Perpetual Pavement materials and layer composition

In the design of a Perpetual Pavement, but in general in all the pavements, the selection of the materials and the knowledge of their proprieties and behaviour, in a certain environmental condition, is fundamental. So, the study and the characterization of the foundation and the asphalt layers is crucial.

A typical layer composition of perpetual pavement is made by:

1. Solid foundation
2. Flexible, fatigue-resistant HMA base layer
3. Durable, rut-resistant intermediate HMA layer.
4. Rut-resistant, renewable surface layer

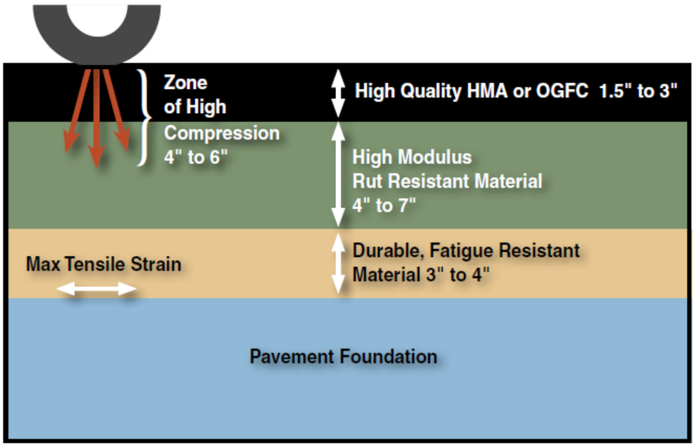


Figure n. 3 – Perpetual Pavement cross-section (Newcomb et al, 2000)

4.1.1 Foundation

The foundation is structured considering a subgrade made by soil, adequately compacted, and a sub-base course, made by unbound granular mixture.

The subgrade should provide adequate stiffness because it provides resistance to deflection allowing rollers to produce a firm compaction of all layers (*APA, 2002*). It provides also a uniform support to the entire pavement. Uniformity in an important aspect, because we should avoid that is some section, we have less support creating differential settlement and the consequently failure of the pavement. The top 15-30 cm should be adequately compacted.

Regardless of the kind of material employed, the foundation should meet some minimum requirement for stiffness throughout construction as well as during the life of the pavement *(Thomas et al., 2004).* Depending upon site conditions and pavement design, this may require the chemical or mechanical stabilization of soils or base course materials

Pratico et al. (2011) proposed that for higher traffic volume combined lime-cement treatment is good compared with lime treated subgrade soils. The changes of module in these layer and unbound materials may affect the mechanistic responses of the pavement and should consider the worst condition in order to prevent damage (*APA, 2002*). The Illinois DOT (IDOT) and Newcomb et al. (2010) proposed that the subgrade should have a California Bearing Ratio (CBR) of at least six to avoid excessive deformation during the construction and overstressing periods of pavements life.

In the Figure n. 4 we can clearly see when remedial action is required if the soil CBR is less than 6 or is optional between a CBR of 6 and 8, and it is considered unnecessary above 8. The remedial procedures provide a working platform adequate to prevent overstressing the subgrade, facilitate paving operations, and are sufficiently stable to minimize the development of surface rutting from construction traffic.

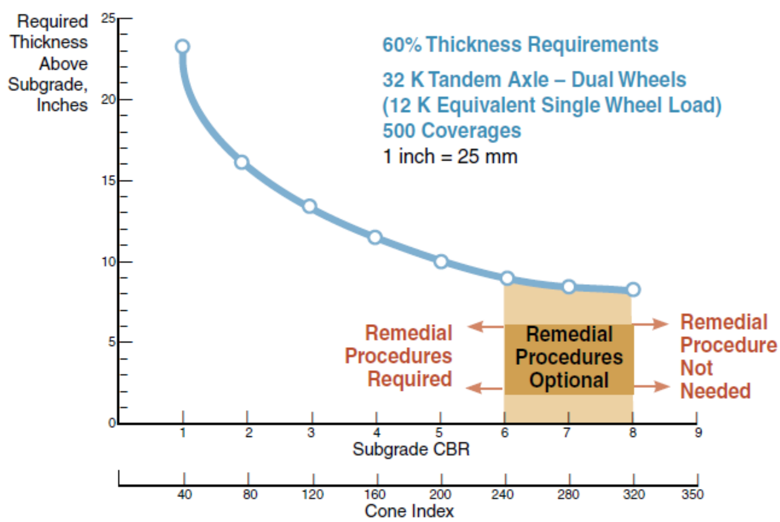


Figure n. 4 – Illinois Granular Thickness Requirement for foundation (IDOT, 1982)

Another important aspect is related to the seasonal effect of the resilient modulus of the subgrade and unbound material modulus. In time, we can have a change on the available resilient modulus, due to the fact that we have a variation of the moisture content. We can have for instance an increase of the modulus due to freezing effect, getting a very high value.

Some approaches use a seasonal modulus adjustment factors for subgrade and overlying granular materials to characterize their respective behaviours during the design life. Another possible approach is to take into account the different impact on the pavement of the changes, because unfortunately this phenomenon doesn't follow a linear relationship but it's a complex phenomenon. For this reason, it is possible to consider the concept of relative damages, in which we don't do a simple mean of the modulus in the different months but for each E we will consider the relative damage. Then, we will calculate the average of the damage coefficient and considering the corresponding resilient modulus associated to this value.

The British *(Nunn et al., 1997)* formulated an end-result specification founded on nuclear density tests and surface stiffness as measured by a portable dynamic plate bearing test. The foundation design practice in the UK is shown in Table n.1. The CBR of the subgrade dictates the thickness of the overlying granular layers called the capping and subbase layers. For a subgrade CBR of less than 15, a minimum six-inch thickness of subbase is required. Capping material may be considered similar in quality to a lower quality base course material in the U.S., and the subbase may be considered a high-quality base material. TRL set end-result requirements for the pavement foundation, both during and after its construction. Under a falling weight deflectometer (FWD) load of 9000 lb, a stiffness of 5800 psi was required on top of the subgrade and 9500 psi was required at the top of the subbase.

Table 3. Transport Research Laboratory Foundation Requirements
(Nunn et al., 1997)

Subgrade CBR	**< 2**	**2 - 5**	**> 5**
Subbase Thickness, in.	6	6	9
Capping Thickness, in.	24	14	—

Table n. 1 – Transport research Laboratory Foundation Requirements (Nunn et al.,1997)

So, at the end, we can say that the foundation is crucial for the construction and performance of a Perpetual Pavement.

4.1.2 Flexible, fatigue-resistant HMA base layer

As it was mentioned in the previous paragraphs, one of the two failure criteria were the bottom up fatigue cracking. It was also mentioned the existence of Fatigue Endurance Limit, under which the damage is not cumulating. So, the HMA base layer has the main function to resist to the repeated loading condition and to avoid the formation of cracks which can be propagating, giving the possibility to the water to go inside, reducing the structural function of the pavement.

Generally, two basic mix designs used to improve the fatigue life: Softer binder and higher binder content. The most common practice for improving the fatigue life is by incorporating a higher asphalt content in the mix design (*Romanoschi et al., 2008; Newcomb et al., 2010*).

One of the main mixture characteristics that can help guard against fatigue cracking is a higher designed asphalt content which accomplishes two important goals: from one side it allows the material to be compacted to a higher density, and in turn, improve its durability and fatigue resistance.

The asphalt content in the base should be defined as that which produces low air voids in place. This ensures a higher volume of binder in the voids in mineral aggregate (VMA), which is critical to durability and flexibility. This concept has been substantiated by Linden et al. (1989) in a study that related higher-than-optimum air void content to reduction in fatigue life. Fine-graded asphalt mixtures have also been noted to have improved fatigue life (*Epps and Monismith, 1972*).

As demonstrated in the study made by Elie Y. Hajj et al., Impact of Rich-Bottom Design in Asphalt Pavements, the rich-bottom mix can be defined as having a binder content that is 0.5% higher than the optimum binder content. They designed and tested different rich dense graded hot mix asphalt (HMA) mixes and evaluated them using unmodified and polymer-modified asphalt binders. An extensive laboratory evaluation was undertaken to determine the mixture's properties, such as resilient modulus, as well as its fatigue and rutting characteristics. Additionally, mechanistic analyses were conducted for a total of twenty-four pavement structures and based on the data generated from the laboratory experiment and the mechanistic analyses, they demonstrated that the rutting resistance of the rich mix is similar to its corresponding optimum mix, supporting the use of polymer-modified mixes in the top lift, and the rich-bottom design increased the fatigue life of the pavement structure when compared to the conventional pavement structure.

Another possible approach can be to design the thickness, creating a stiffer structure in a way to reduce the tensile strain that is acting on the bottom of the last HMA layer. As it is obvious that if the engineer design stiff structure, the tensile strain at the base of the HMA will be reduced. The FRL should have a half percent of asphalt binder content increase (*Carpenter & Shen, 2006*).

4.1.3 Durable, rut-resistant intermediate HMA layer

The intermediate or binder layer must combine the qualities of stability and durability. Stability in this layer can be obtained by achieving stone-on-stone contact in the coarse aggregate and using a binder with an appropriate high-temperature grading (*Newcomb et al., 2010*). This is especially crucial in the top four inches of the pavement where high stresses induced by wheel loads can cause rutting through shear failure. The internal friction provided by the aggregate can be obtained by using crushed stone or gravel and ensuring an aggregate skeleton. One option would be to use a large nominal maximum size aggregate which could reduce cost due to a lower asphalt content (*Newcomb et. al.*).

Both binder and aggregate are of importance for resisting shearing failure and formation of ruts. It is typically the thickest layer in the system expose to both tension and compression by situating on both sides of the neutral axis. The stone on stone contact in the coarse aggregate gives stability to mix however large nominal maximum aggregate size can lead to segregation and pressure of air void can expose to the intrusion of water. So, a viable option is to keep lower void content in the mix design with a high level of compaction in the field (*APA, 2010*).

Rutting can be prevented by using an appropriate high temperature grade binder. The high-temperature grade of the asphalt should be the same as the surface to resist rutting. However, the low temperature requirement could probably be relaxed one grade, since the temperature gradient in the pavement is relatively steep and the low temperature in this layer would not be as severe as the surface layer (*Newcomb et. al*., *2006*).

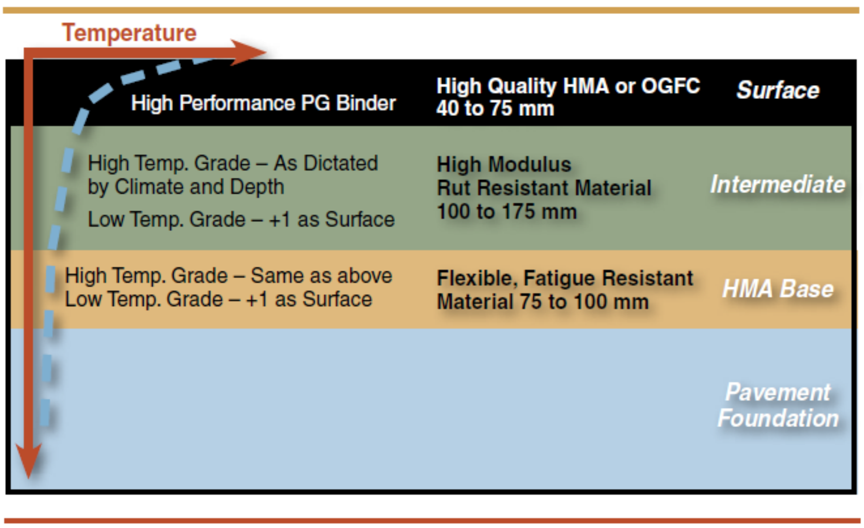


Figure n. 5 – Impact of temperature gradient on asphalt grade (Asphalt pavement alliance, IM-40)

Pavement temperature had large effect on pavement response, but it changed with depth. Surface deflection must be corrected to a reference pavement temperature. Mid depth temperature considered as the representative temperature for pavement structure. It has the best relation to measured tensile strain (*Ma & Huang, 2013*). BWLLS3 method is recommended for calculating the mid depth pavement temperature if there is no measured mid depth pavement temperature data. Also, Huber and Chen temperature correlation method is recommended for temperature correction at the network level (*Gedafa et al., 2013*).

4.1.4 Rut-resistant, renewable surface layer

The wearing surface requirements would depend on traffic conditions, environment, local experience, and economics. Performance requirements include resistance to rutting and surface cracking, good friction, mitigation of splash and spray, and minimization of tire-pavement noise. These considerations could lead to the selection of stone matrix asphalt (SMA), an appropriate Superpave dense-graded mixture, or open-graded friction course.

In some cases, the need for rutting resistance, durability, impermeability, and wear resistance would dictate the use of SMA. This might be especially true in urban areas with high truck traffic volumes. Properly designed and constructed, an SMA will provide a stone skeleton for the primary load carrying capacity and the matrix (combination of binder and filler) gives the mix additional stiffness.

The matrix in an SMA can be obtained by using polymer-modified asphalt, with fibers, or in conjunction with specific mineral fillers. Brown and Cooley (1999) concluded that the use of fibers is beneficial to preclude drain-down in SMA mixtures. They also point out the need to carefully control the aggregate gradation, especially on the 4.75 mm and 0.75 mm sieves. In instances where the overall traffic is not as high, or in cases where the truck traffic is lower, the use of a well-designed, dense-graded Superpave mixture might be more appropriate. As with the SMA, it will be necessary to design against rutting, permeability, weathering, and wear. The Asphalt Institute *(1996b)* provides guidance on the volumetric proportioning of Superpave mixtures. It is recommended that a performance test of dense-graded mixtures, whether SMA or Superpave, be done during mixture design. At a minimum, this should consist of rut testing *(Brown et al., 2001)*, but other tests such as the flow number test from the AMPT *(Dongre et al., 2009)* or the Superpave shear tester *(Sousa et al., 1994)* could be employed to estimate the performance of the material *(Newcomb et al.)*.

Open-graded friction courses (OGFC) are designed to have voids that allow water to drain from the roadway surface. These are primarily used in western and southern regions of the United States to improve wet-weather friction but may be found in northern states such as Massachusetts, New Jersey, and Wyoming also. Mixtures should be designed to have about 18 to 22% voids to provide good long-term performance *(Huber, 2000)*.

As it is subjected to highest temperature variation means inclined to experience more thermal cracking which can be prevented by a selection of low temperature grade binder. APA recommends using a performance grade one temperature higher than is typically the area (*APA, 2002; Wills & Timm,2009*).

5. Mechanistic-Empirical approach for Perpetual Pavement

The M-E approach is an iterative method, in which the pavement response in terms of stresses, strains or deflections is used to estimate the allowable number of loads to failure for a given conditions of loading and material properties (*Newcomb et al., 2010*).

The structural responses of a flexible pavement under a certain loading condition are evaluated using mathematical models. Different models can be used and the most common is the layered elastic model. The basic assumption is that each layer is homogeneous, isotropic and linearly elastic with an elastic Modulus E and a poisson ratio ν. Moreover, pavement layers extend infinitely in the horizontal direction and the bottom layer (usually the subgrade) extends infinitely downward.

A layered elastic model requires a minimum number of inputs to adequately characterize a pavement structure and its response to loading. These inputs are:

- Material properties of each layer (E, ν)
- Pavement layer thicknesses
- Loading conditions

The latter one has to be characterized by a certain magnitude and geometry. Usually specified as being a circle of a given radius (r or a), or the radius computed knowing the contact pressure of the load (p) and the magnitude of the load (P). Although most actual loads are more closely represented by an ellipse.

Figure n. 6 shows how these inputs relate to a layered elastic model of a pavement system.

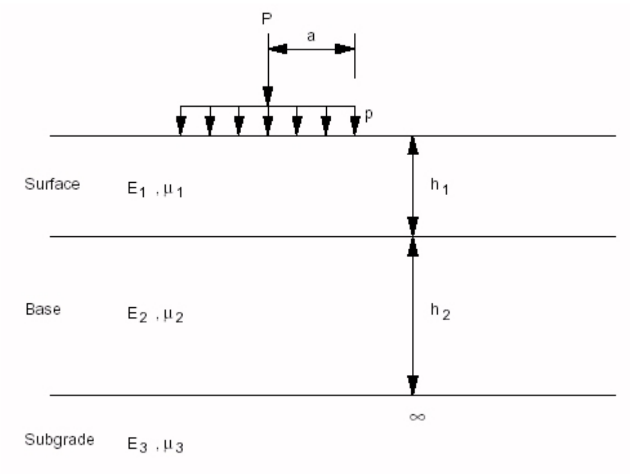


Figure n. 6 – Layered elastic model inputs

The outputs of a layered elastic model are the stresses, strains, and deflections in the pavement:

- **Stress**. The intensity of internally distributed forces experienced within the pavement structure at various points. Stress has units of force per unit area (N/m2, Pa or psi).
- **Strain**. The unit displacement due to stress, usually expressed as a ratio of the change in dimension to the original dimension (mm/mm or in/in). Since the strains in pavements are very small, they are normally expressed in terms of micro strain (10-6).
- **Deflection.** The linear change in a dimension. Deflection is expressed in units of length (mm or µm or inches or mils).

As we mentioned in the introduction, the two worst distress phenomena are fatigue cracking and rutting. This is the reason why we will focus on two critical strains: the maximum tensile strain at the bottom of the asphalt layer and the maximum vertical strain at the top of the subgrade. In this way, it is possible to estimate the allowable numbers of loads to failure (N_f), taking always into account that we are referring to a given loading condition.

Then, another important and intricated aspect is related to traffic prediction, because the allowable numbers of loads must be compared with the predicted number of loading, coming from the traffic calculations. It is possible to adopt a simple approach, referring to the Annual Average Daily Traffic (AADT), which is given by the number of vehicles passing through a single infrastructure in a single day, and we can refer to the average over the year.

Knowing the AADT, it must be multiplied for some factors as:

- Analysis period
- Directional Distribution Factor
- Percentage of traffic on the design lane
- Growth factor
- Percentage of heavy vehicles

Consequently, we have to translate the information referring to vehicles into axles, and in particular standard axles. Even if it is not the axle/wheel load but rather the damage to the pavement caused by the wheel load that is of primary concern, is not so simple to determine the number and types of wheel/axle loads that a particular pavement will be subjected to over its design life. The most common historical approach is to convert damage from wheel loads of various magnitudes and repetitions ("mixed traffic") to damage from an equivalent number of "standard" or "equivalent" loads. The most commonly used equivalent load in the U.S. is the 18,000 lb (80 kN) equivalent single axle load (normally designated ESAL).

Finally, we will compare the two predictions. The number of allowable loading must be compared with the traffic prediction and it must be higher, and to develop a correct and well design pavement it must be slightly higher, because otherwise there is the concrete risk to overdesign it.

A very schematic flowchart of the method can be showed in the following *figure n. 7*:

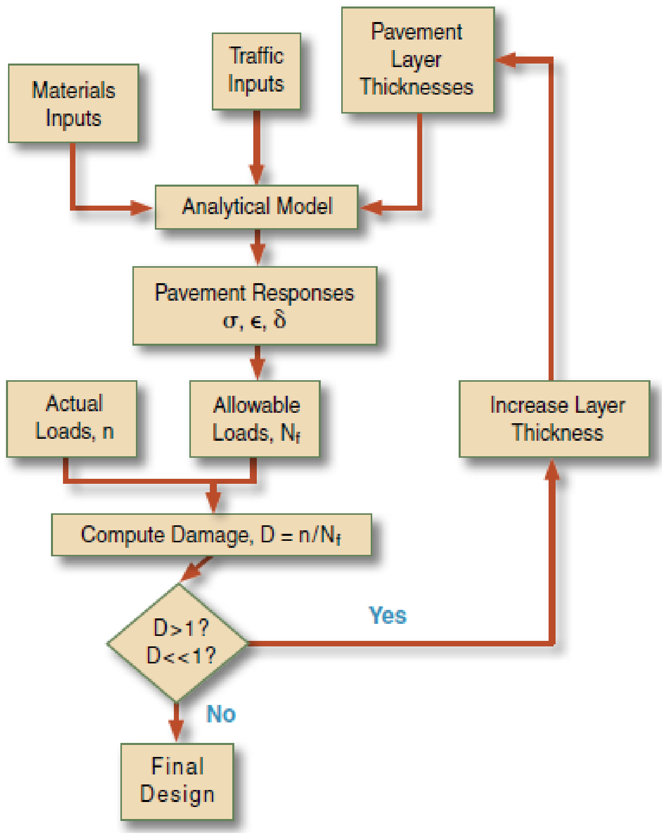


Figure n. 7 – Flowchart of Mechanistic-Empirical approach

In Perpetual Pavement design, there are limiting strains below which damage does not occur, and thus damage is not accumulated. So, the key assumptions of these design approaches are that the pavement response determined in terms of stress, strain and deflections needs to be kept under a specified limit. In this sense, the concept design can be summarized in the *figure n.8*:

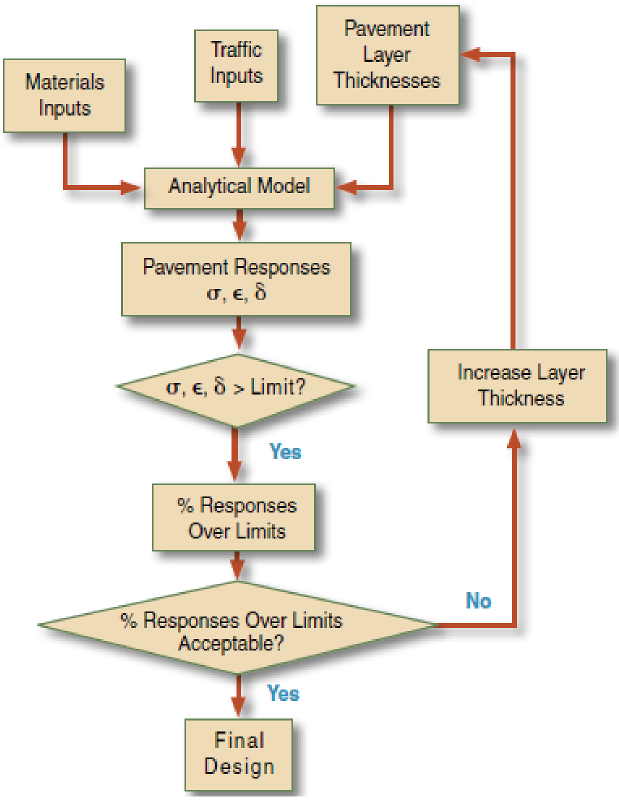


Figure n. 8 – Flowchart of Perpetual Pavement Mechanistic-Empirical approach

In this way, a key aspect is related to the setting of the threshold values. As showed in the paragraph n. 3, different researches have demonstrated that the following values of strain can be considered:

- 70 micro strain for fatigue cracking
- 200 micro strain for fatigue structural rutting

5.2 The Texas PP design concept

Texas is one of the countries in which, starting from the early 2000s, the concept of Perpetual Pavement was widely studied, with several researches and projects. Still today, there are 10 Perpetual Pavement section in service, constructed in four Texas districts:

- Fort Worth 2 sections on SH 114
- Laredo 4 sections on IH 35
- San Antonio 2 sections on IH 35
- Waco 2 sections on IH 35

The general PP design philosophy is to mitigate rutting and bottom-up fatigue cracking in the pavement structure, with a design structural life of up to 50 years. However, they are subject to periodic surface maintenance and/or renewal in response to surface distresses in the upper layers of the pavement during their service lives. Deep seated structural distresses such as fatigue cracking (bottom-up) and/or rutting should not occur or if present are very minimal (*Lubinda F. et al.*).

Considering the two main distress phenomena above mentioned, they considered as critical strain thresholds the following:

- Horizontal tensile strain at the bottom of the lowest HMA layer (ε_t): $\leq$ 70 $\mu\varepsilon$ (bottom-up fatigue cracking)
- Vertical compressive strain on the top of subgrade (ε_v): $\leq$ 200 $\mu\varepsilon$ (rutting)

So, from a conceptual point of view there is not so much difference with respect to the general PP mechanistic-empirical (M-E) design principle. The true difference with respect the other approaches is based on the materials and layered structure. In the Texas PP design concept, there is a clear and detailed description of the layers which compose the PP, as it's possible to see in the following *figure n. 9*:

	Layer Designation, Materials, and Functions				Thickness (inches)
Layer 1	PFC (SS3231)	Porous Friction Course		Sacrificial layer	1.0 – 1.5
Layer 2	HDSMA (SS3248)	Heavy-Duty SMA	1/2" Aggregate + PG 76-XX	Impermeable load carrying layer	2.0 – 3.0
Layer 3	SFHMAC (SS3249)	Stone-Filled HMAC	3/4" Aggregate + PG 76-XX	Transitional layer	2.0 – 3.0
Layer 4	SFHMAC (SS3248)	Stone-Filled HMAC	1.0-1.5" Aggregate + PG 76-XX	Stiff load carrying layer	8.0 - Variable
Layer 5	Superpave (SS3248)	Superpave (RBL)	1/2" Aggregate + PG 64-XX (Target lab density=98%)	Stress relieving impermeable layer	2.0 – 4.0
Layer 6	Stiff base or stabilized subgrade		Construction working table or compaction platform for succeeding layers		6.0-8.0
Subgrade					∞

Figure n. 9 - Typical Texas PP Structural Section (*Lubinda F. et al.*)

5.2.1 Layered structure and material characterization

As highlighted in the previous picture, with respect to the general structure, there are more than 3 HMA layers.

The first layer is a PFC (Porous Friction Course). This layer is optional, in fact, only in 4 sections over 10 is used (San Antonio–IH 35 and Waco–IH35). For this reason, this layer is not considered from a structural point of view. Porous friction courses (PFCs) are mainly recommended as surface drainage layers on high-speed road-corridors and runway pavements. These are used as surface drainage layers to improve pavement surface permeability, skid-resistance, and visibility and also to mitigate hydroplaning effect during wet-weather conditions.

The typical mix-designs and material characteristics that they used in 4 of the 10 Texas PP are:

- 6.0-6.1%PG 76-22S
- 0.0-1.0% lime
- 0.3-0.4% cellulose fibers
- igneous/limestone aggregates (19 mm NMAS open-graded) (Avg OAC = 6.0%)

The second layer is a SMA (Stone Matrix Asphalt). Stone matrix asphalt (SMA) is a gap-graded HMA (Figure 1) that is designed to maximize deformation (rutting) resistance and durability by using a structural basis of stone-on-stone contact. Because the aggregates are all in contact, rut resistance relies on aggregate properties rather than asphalt binder properties. Since aggregates do not deform as much as asphalt binder under load, this stone-on-stone contact greatly reduces rutting (*NAPA, 1999*).

The typical mix design is composed by:

- 5.9-6.8% PG 76-22S
- 5.0-11.0% mineral filler
- 0.0-1.5% lime
- 0.0-0.4% cellulose fibers
- 0.0-4.5% fly ash
- igneous/limestone aggregates (12.5 mm NMAS gap-grade) (Avg OAC = 6.0%)

Layers 1 (PFC) and 2 (SMA) are intended to improve the resistance to oxidation/weathering, thermal cracking, rutting, and permeation.

Layer 3 is a transitional load carrying layer, also composed of a SFHMAC mix with a NMAS of around ¾ inch. The mix is made by:

- 4.2-5.2% PG 76-22
- 0.0-1.5% lime
- 0.0-1.0% anti-strip
- Limestone aggregates (19 mm NMAS dense to coarse graded) (Avg OAC = 4.4%)

Layer 4 represents the main structural load-carrying and stiff rut-resistant layer with a minimum thickness of 8 inches to ensure adequate structural capacity in terms of the load spreading capability. The 1inch SFMAC was primarily designed as the main load bearing layer with an expected high resistance to rutting, and thus, the coarse aggregate gradation (*Lubinda F. et al.*).
During the construction phase they noticed some difficult in terms of workability and constructability, mainly due to the low binder content and the presence of coarse aggregate gradation with low fines content.

The mix design is made by:

- 4.0-4.5% PG 70-22
- 0.0-1.5% lime
- 0.0-0.5% anti-strip
- Limestone (25 mm NMAS coarse-graded with low fines) (Avg OAC = 4.2%)

Then, there is the last HMA layer, an RBL (Rich-Bottom Layer). As it has been mentioned, one of the main failure criteria is the bottom up fatigue cracking, and for this reason the Rich Bottom Layer" (RBL) also called "Fatigue Resistant Layer" (FRL) is used to increase the fatigue resistance. This can be done increasing the binder content. Usually, one of the possible approaches is to increase of 0.5% the optimum content. In this case the mix was done using:

- 4.2-6.1% PG 64-22
- 0.0-1.5% lime
- 0.0-0.5% anti-strip
- Limestone aggregates (12.5 mm NMAS dense-graded) (Avg OAC = 5.4%)

5.2.2 Future Texas PP Design and Recommendations

The load projections based on the actual traffic data and the measured material properties indicated that the Texas PP structures were conservatively designed. The results indicated that the total HMA thickness can be satisfactorily reduced to about 12 to 14 inches, resulting in an over 6-inch HMA cost-savings from the current 22 inches. With the currently designed greater total HMA thicknesses, the results also indicated that the RBL was structurally unnecessary. However, the RBL may still optionally be required for durability and impermeability characteristics.

So, the PP structures could be optimized down to about 12 to 14 inches in total HMA thickness, without compromising the PP structural integrity. The computed strain responses are satisfactorily within the M-E threshold, and the predicted performance life is over 20 years.

Based on the findings coming out from different researches and from an extensive computational analysis with the FPS and MEPDG software, they suggest different alternative structural designs as a function of three traffic levels:

- traffic ESALs ≤ 30 million
- 30 million < Traffic ESALs ≤ 50 million
- traffic ESALs > 50 million.

As highlighted from the figure n. 10 the proposal is for the future Texas PP design to have a structural thickness, as a minimum, of 12 inches HMA and 6 inches treated base material for sections with traffic level of 30 million ESALs or less. For traffic greater than 50 million ESALs, the minimum should be 15 inches total HMA thickness and 8 inches treated base material.

Table 9-1. Future Texas PP Design Proposals.

Layer#	Thickness (inches)	Mix Type	Designation	2004 TxDOT Spec Item	Material
(a) Traffic ESALs ≤ 30 million					
1	2	SMA	Surfacing	Item 346	PG 70-28 or better
2	2	¾-inch Superpave	Load transitional layer	Item 344	PG 70-22 or better
3	≥ 6	Type B	Main structural load-carrying rut-resistant layer	Item 341	PG 64-22 or better
4	2	Type C or ½-inch Superpave	Rich bottom fatigue-resistant layer (durability & impermeability	Item 341	PG 64-22
5	≥ 6	Base	Lime or cement treatment	Items 260, 263, 275, & 276	
Subgrade (in-situ soil material)					
Minimum PP structure thickness = 18 inches (12 inches HMA and 6 inches base)					
(b) 30 million < Traffic ESALs ≤ 50 million					
1	2	SMA	Surfacing	Item 346	PG 70-28 or better
2	3	¾-inch Superpave	Load transitional layer	Item 344	PG 70-22 or better
3	≥ 8	Type B	Main structural load-carrying rut-resistant layer	Item 341	PG 64-22 or better
4	2	Type C or ½-inch Superpave	Rich bottom fatigue-resistant layer (durability & impermeability	Item 341	PG 64-22
5	≥ 6	Base	Lime or cement treatment	Items 260, 263, 275, & 276	
Subgrade (in-situ soil material)					
Minimum PP structure thickness = 21 inches (15 inches HMA and 6 inches base)					
(c) Traffic ESALs > 50 million					
1	2-3	SMA	Surfacing	Item 346	PG 70-28 or better
2	≥ 3	¾-inch Superpave	Load transitional layer	Item 344	PG 70-22 or better
3	≥ 8	Type B	Main structural load-carrying rut-resistant layer	Item 341	PG 70-22 or better
4	2-4	Type C or ½-inch Superpave	Rich bottom fatigue-resistant layer (durability & impermeability	Item 341	PG 64-22
5	≥ 8	Base	Lime or cement treatment	Items 260, 263, 275, & 276	
Subgrade (in-situ soil material)					
Minimum PP structure thickness = 23 inches (15 inches HMA and 8 inches base)					

*On top of the SMA, a PFC (TxDOT 2004 spec item 342) can be added as an "optional" surface moisture drainage, noise-reduction, and skid-resistant layer. Preferably, the PFC layer thickness should be 1.5 inches.

Figure n. 10 - Future Texas PP Design Proposals (*Lubinda F. et al.*)

6. Perpetual Pavement design software

In order to make a comparison of the available software used for the design of a Perpetual Pavement, different software have been evaluated. These compared to those one used for the design of traditional flexible pavement, allow to highlight the peculiarities of a Perpetual Pavement.

Three software have been evaluated:

- PerRoad
- Mechanistic-Empirical Asphalt Pavement Analysis (MEAPA)
- PaveXpress

All the three software are based on the classical Mechanistic–Empirical theory, but at the same time they have different approaches and different input requirements.

6.1 PerRoad software

PerRoad is a software created with the aim to design and check Perpetual Pavements. It is based on a mechanistic-empirical approach with the use of layered-elastic theory.

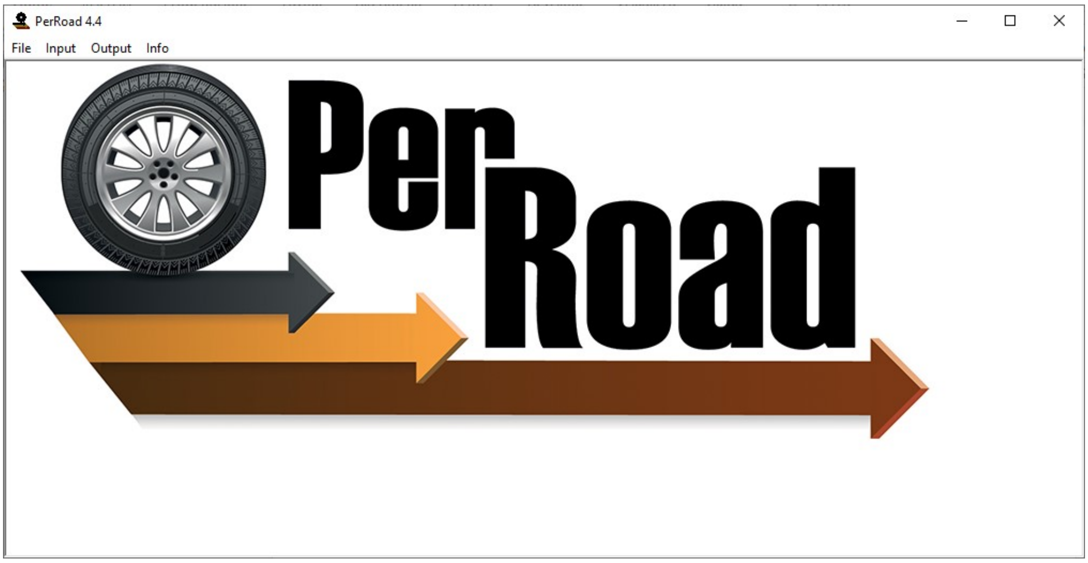


Figure n. 11 – PerRoad software main screen

The procedure was developed at the National Center for Asphalt Technology (NCAT) at Auburn University (AU) in conjunction with the Asphalt Pavement Alliance (APA), the National Asphalt Pavement Association (NAPA) and State Asphalt Pavement Associations (SAPAs) through the Pavement Economics Committee.

PerRoad uses the layered-elastic theory combined with Monte Carlo simulation. Thus, adopting a probabilistic approach for the evaluation of stresses, strains and deflections.

The software has a relatively simple approach. It's possible to distinguish two input interfaces and one of output.

6.1.1 Analysis and input Interface

The first input interface is related to the seasonal and structural information, where for structural information it refers to the layer information. In the seasonal information, you must consider each season and its relative duration in weeks (the sum must be 52). In this case the AC modulus can be automatically adjusted using the temperature correction or if it is turned off it's possible to put a referred value.

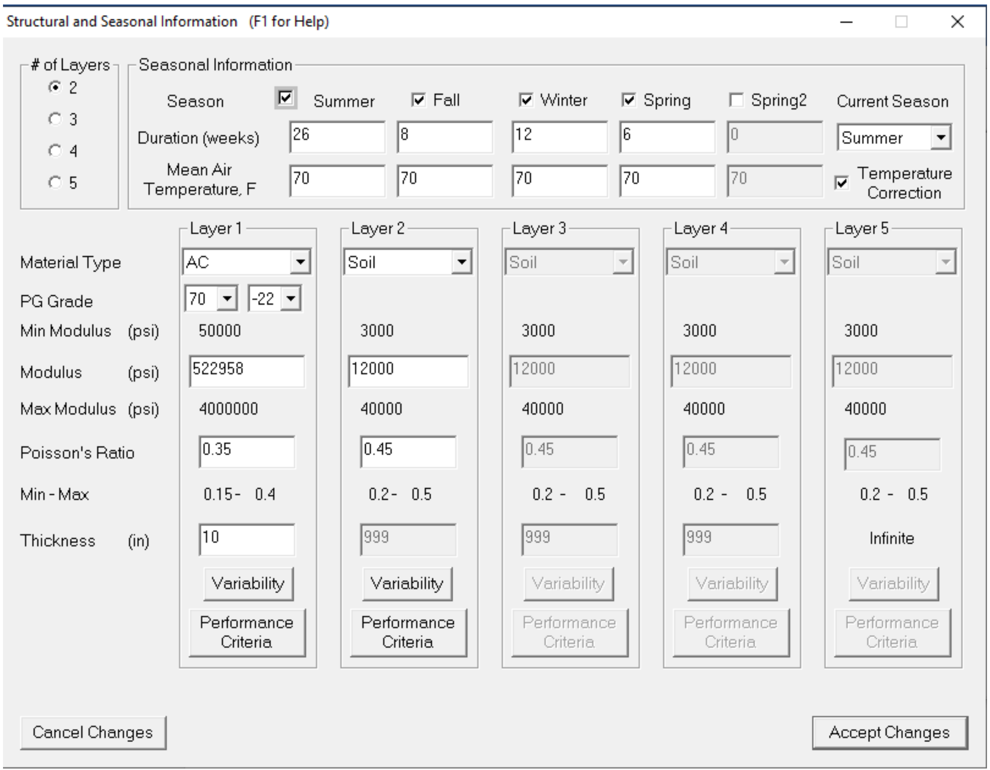


Figure n. 12 – PerRoad software input interface

Then, another aspect is related to the variability, and in particular to the modulus and thickness variability. The default values were set to be consistent with values found in the literature but can be adjusted as needed. These values are used during the Monte Carlo simulation to generate pavement response distributions and the amount of variation should increase deeper into the pavement structure.

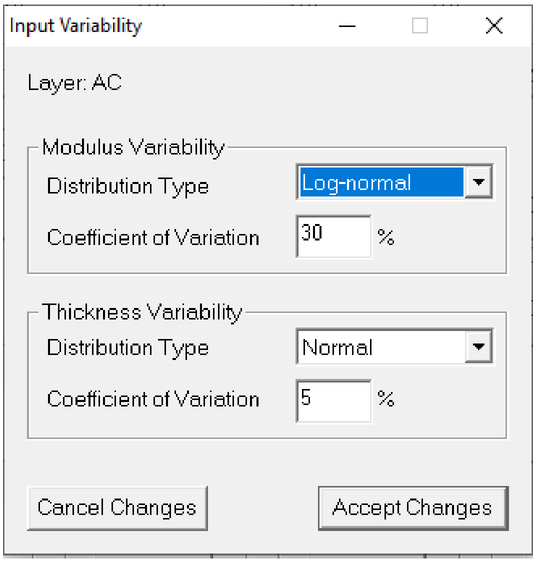


Figure n. 13 – Input variability window

Then, for the analysis it is necessary to set the design criteria for each pavement layer and to decide which locations of the layer require design criteria: top, middle and/or bottom.

As we mentioned, in case of Perpetual Pavements we referred to bottom-up fatigue cracking and rutting. The bottom-up fatigue cracking is usually controlled by monitoring horizontal tensile strain at the bottom of the lowest new asphalt concrete layer. Rutting is often controlled by monitoring vertical compressive strain at the top of the subgrade layer.

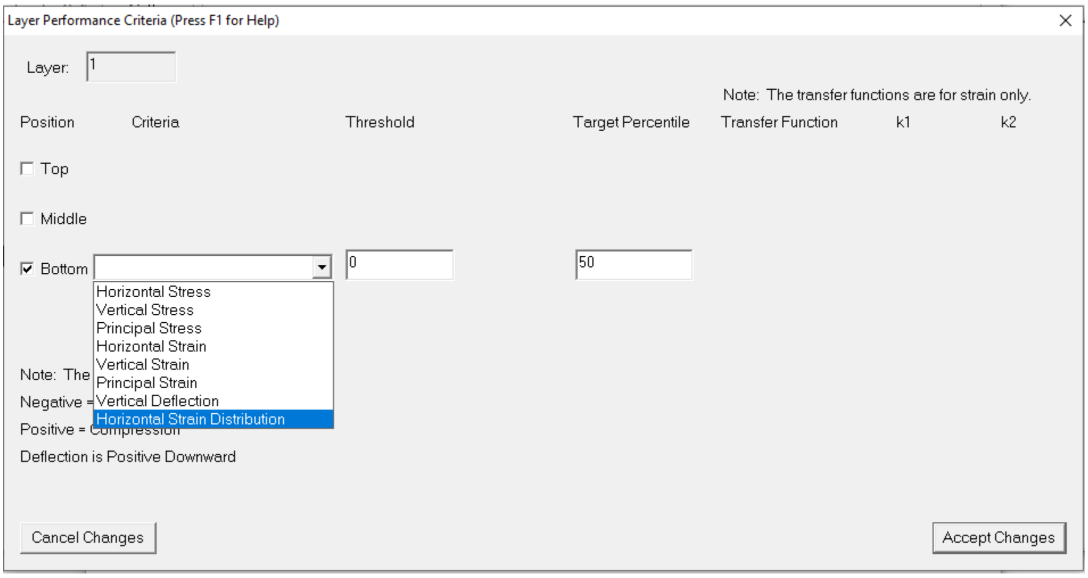


Figure n. 14 – Layer performance window

One of the advantages of this software is the possibility to adopt different approaches for the evaluation for the horizontal strain.

One possible solution is the so-called "Horizontal Strain Distribution". Recent research at the National Center for Asphalt Technology has supported the use of strain distributions for controlling bottom-up fatigue cracking. The main idea of this design approach is to control the range of strain values experienced by the pavement below a pre-defined range. The range of values are quantified by their magnitudes and corresponding percentiles. In PerRoad, the design will be controlled by the 95th, 85th, 75th, 65th and 55th percentiles, respectively. The *figure n.15* shows the results of a sample design where strain distributions were used.

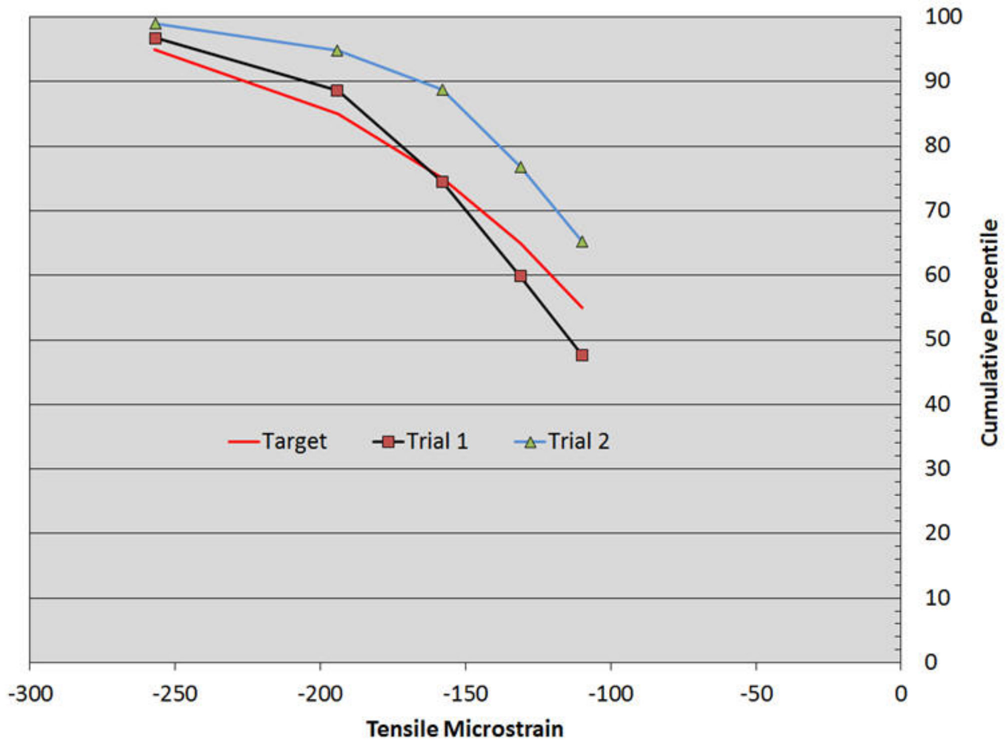


Figure n. 15 – Percentile curve control

The x-axis represents the tensile microstrain. The values are negative to indicate tension. The y-axis represents the cumulative percentile. The red line labeled "Target" indicates the control strain distribution. Points above this line indicate a greater chance that the strain levels are below the target values. Points below this line indicate a poorer chance that the strain levels are below the target values. Therefore, the goal of the design is to select thicknesses such that the resulting strain distribution percentiles fall entirely above the target line. If this is the case, then fatigue cracking should not occur because the pavement is experiencing lower strain levels than those expected to crack the pavement (http://www.asphaltroads.org/PerRoad/).

In order to set each percentile, one possible solution is to consider a "Load Default Distribution" in which automatically the software will set some strain values based directly on results from the NCAT Test Track. Of course, this is a good solution in case that the materials in this design are generally consistent with those from the NCAT Test Track, but if there aren't any better information, this is an acceptable starting point. Otherwise, it is possible to set a reference value of endurance limit, and automatically the software will set the different percentiles.

The other possibility can be to set a strain threshold without considering the distribution described above. In this case is necessary to set just one value the relative percentile control value as highlithed in *figure n. 16*. These are used in the PerRoad Monte Carlo simulation to act as a design control. Values above this target indicate a greater chance that the strain levels are below the target values.

The default value for the target percentile is the 50^{th} percentile, but if you have better information you can easily modify it.

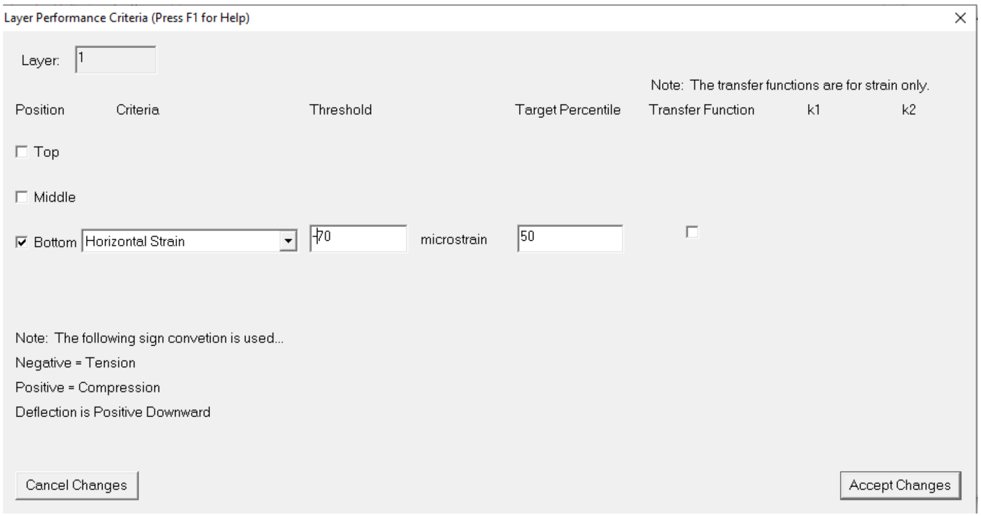


Figure n. 16 – Horizontal strain performance criteria

In case you want adopt the conventional M-E design Criteria there is a box in which it is possible to insert the transfer function coefficients. A transfer function is an equation that is used to predict pavement life in terms of a number of repetitions (or loading cycles) to failure.

The most common transfer functions relate pavement responses to either structural rutting, and fatigue cracking. The transfer function assumes a relationship between the bending strain at the bottom of an asphalt layer and the occurrence of fatigue cracking in that layer, as shown in the following equation:

$$Nf = k1\ (\varepsilon t)\ ^\wedge\ k2 \qquad (2)$$

Where:

- Nf = Number of load cycles to fatigue failure
- k1, k2 = constants
- εt = tensile strain due to bending at the bottom of the HMA.

Likewise, the transfer function for the structural rutting is related to the vertical strain at the top of the subgrade layer and the relative constants, as follows:

$$Nf = k3\ (\varepsilon v)\ ^\wedge\ k4 \qquad (3)$$

Note that the threshold strain must still be specified. When PerRoad executes the Monte Carlo simulation, strain responses less that the threshold do not contribute to damage accumulation. Strain values exceeding the threshold will be entered into the transfer function and used to compute damage over time (http://www.asphaltroads.org/PerRoad/).

The second input interface is related to the loading conditions.

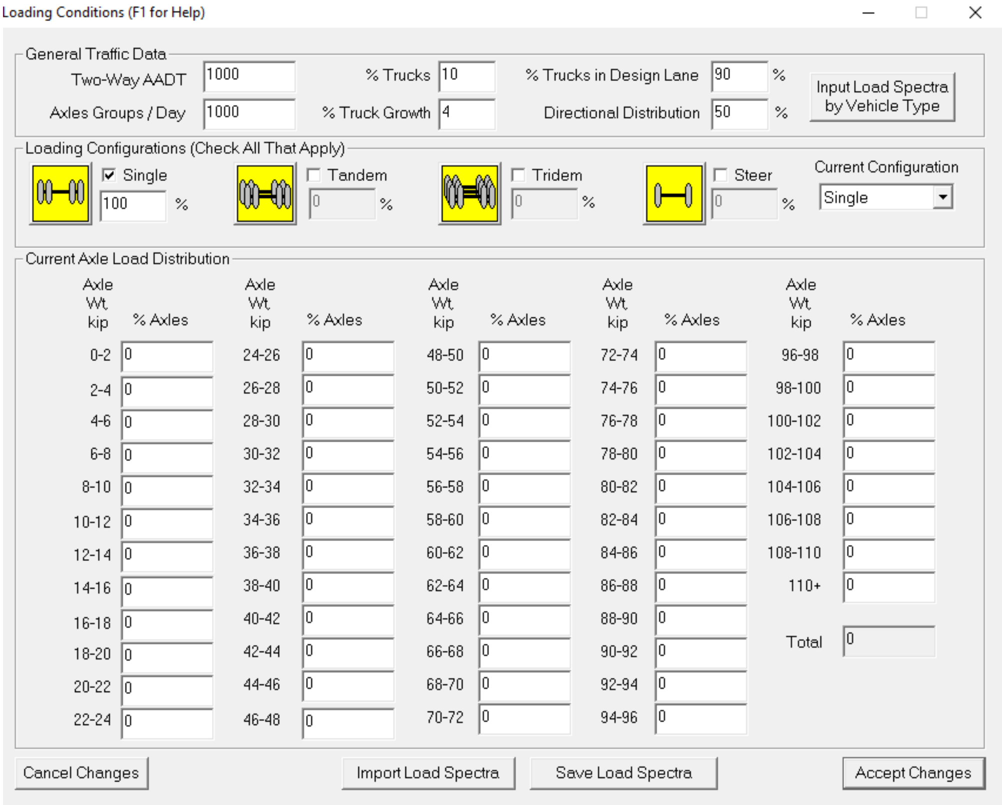


Figure n. 17 – PerRoad software loading condition interface

At this point, there are two approaches; the first one is to fill manually the boxes related to the loading conditions and current axle load distribution. You can simply enter the values of the loading configuration directly and the sum should equal 100%. These percentages will be used in the Monte Carlo simulation to represent the actual expected traffic and generate a distribution of pavement responses. Also, for the current axle distribution you can enter the values directly and the sum should equal 100% for each axle type.

The other approach is to use the Input Load Spectra by Vehicle Type. This function allows you to select the type of road considering the Roadway Functional Classification, and automatically will load a default vehicle distribution.

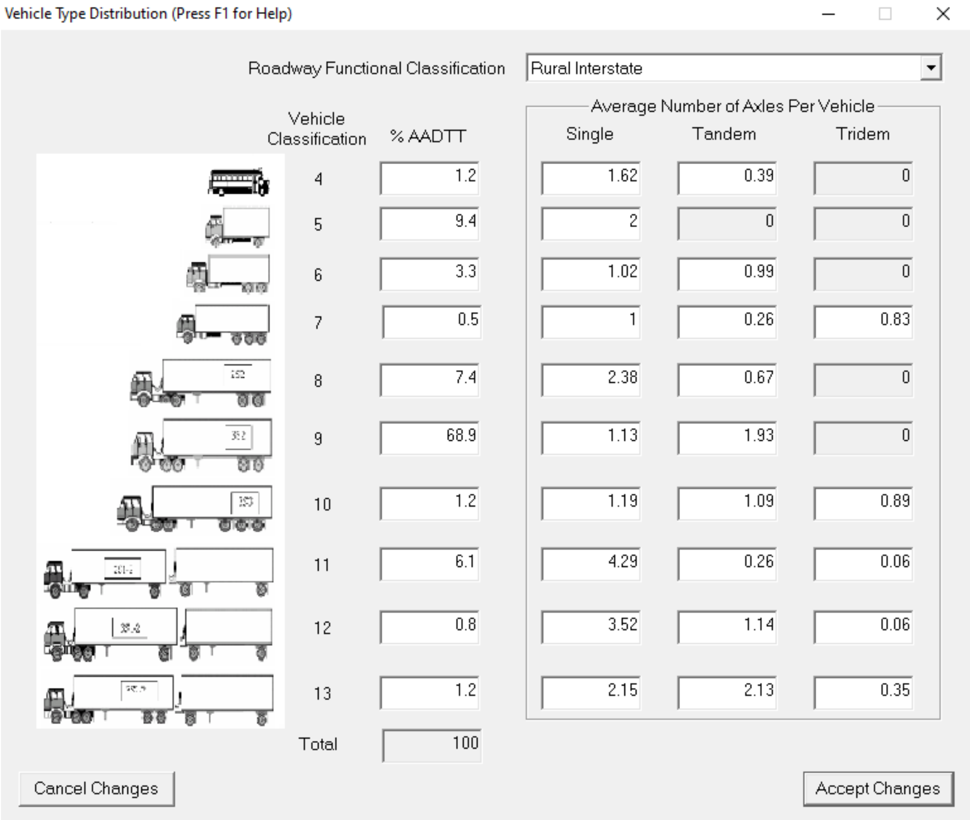


Figure n. 18 – PerRoad software vehicle type distribution interface

6.1.2 Outputs

In the *figure n. 19* it is possible to see the typical output window of PerRoad.

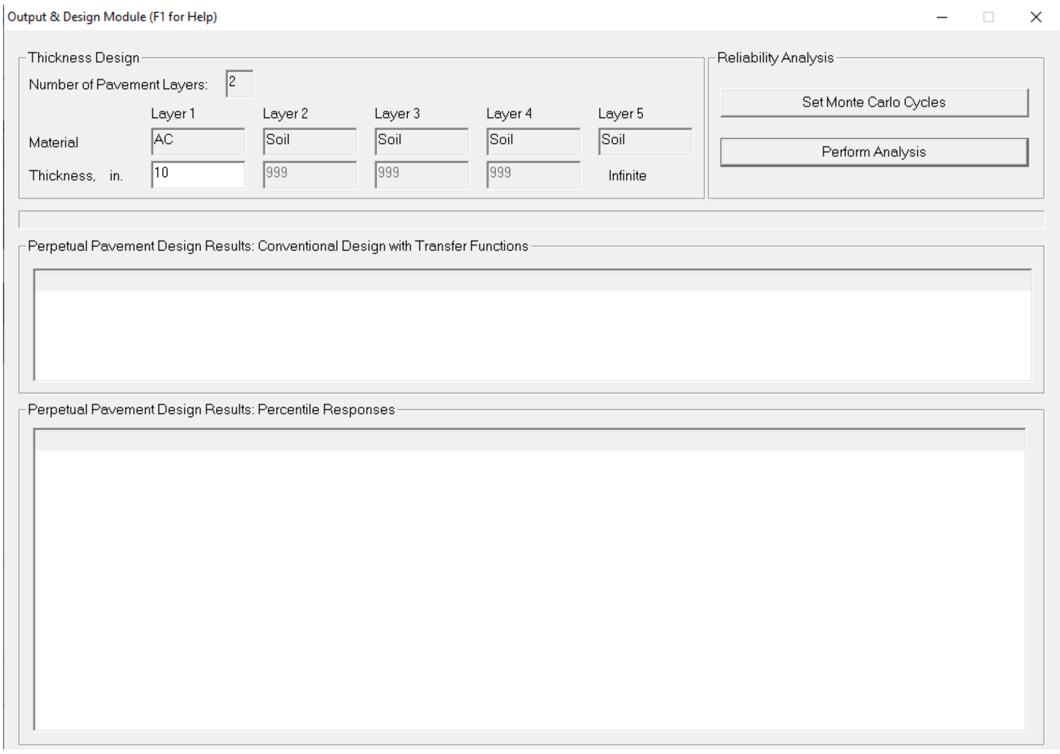


Figure n. 19 – PerRoad software general output interface

Before to perform the analysis, you can change the number of Monte Carlo simulations. The number of simulations has been set to 5,000. More may be needed to achieve a stable solution, but 5,000 is a good starting point. Whenever the designer leaves this output window and comes back, the number of Monte Carlo cycles is reset to 5,000 (http://www.asphaltroads.org/PerRoad/).

Depending on the type of failure criteria selected, you can obtain different output.

If you have selected the transfer function approach, as output you can get:

- **Percent Below Critical**: The probability that a pavement response will not exceed the threshold.
- **Damage/Million Axle:** This parameter indicates the damage accumulation rate as calculated by Miner's hypothesis. The units are damage per million axles.
- **Years to D = 0.1**: This value estimates the amount of time, given the current traffic volume, growth and damage accumulation rate, before the damage number will reach 0.1.

- **Years to D = 1.0**: This value estimates the amount of time, given the current traffic volume, growth and damage accumulation rate, before the damage number will reach 1.0. This value is consistent with traditional M-E methods that designed for a terminal level of distress.

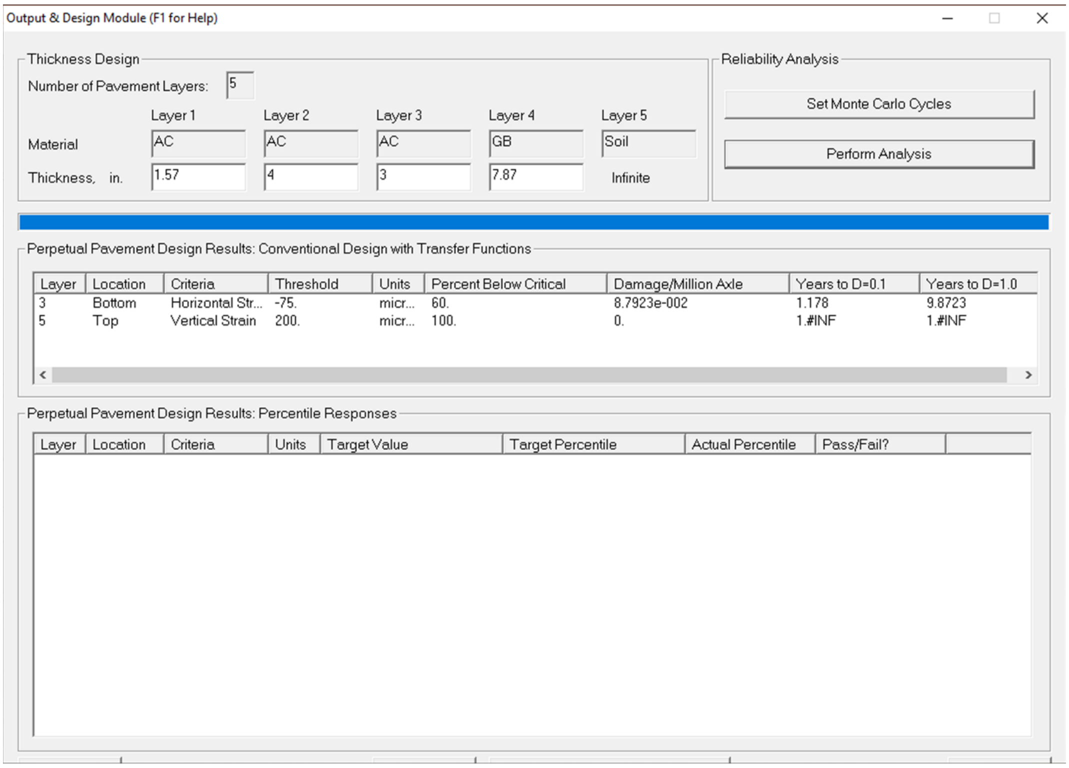


Figure n. 20 – PerRoad software output interface using transfer function

If you have selected a horizontal strain distribution, or a single threshold value with a target control percentile, the outputs will be defined as follows:

- **Target Value**: Designer-specified target pavement response value. Negative is tension, positive is compression.
- **Target Percentile**: Fixed cumulative percentage value corresponding to target response value.
- **Actual Percentile**: Predicted cumulative percentage value corresponding target response value.
- **Pass/Fail**: If the actual percentile exceeds the target percentile, this criterion passes. If not, it fails. If any percentile fails, the thickness(es) should be adjusted and the design re-executed.

For all criteria except Horizontal Strain Distribution, you may have entered a target percentile value. If the actual percentile from the simulation exceeds the target percentile, the pavement passes on that criterion. If you selected a Horizontal Strain Distribution, then there are more control points to be considered as explained below.

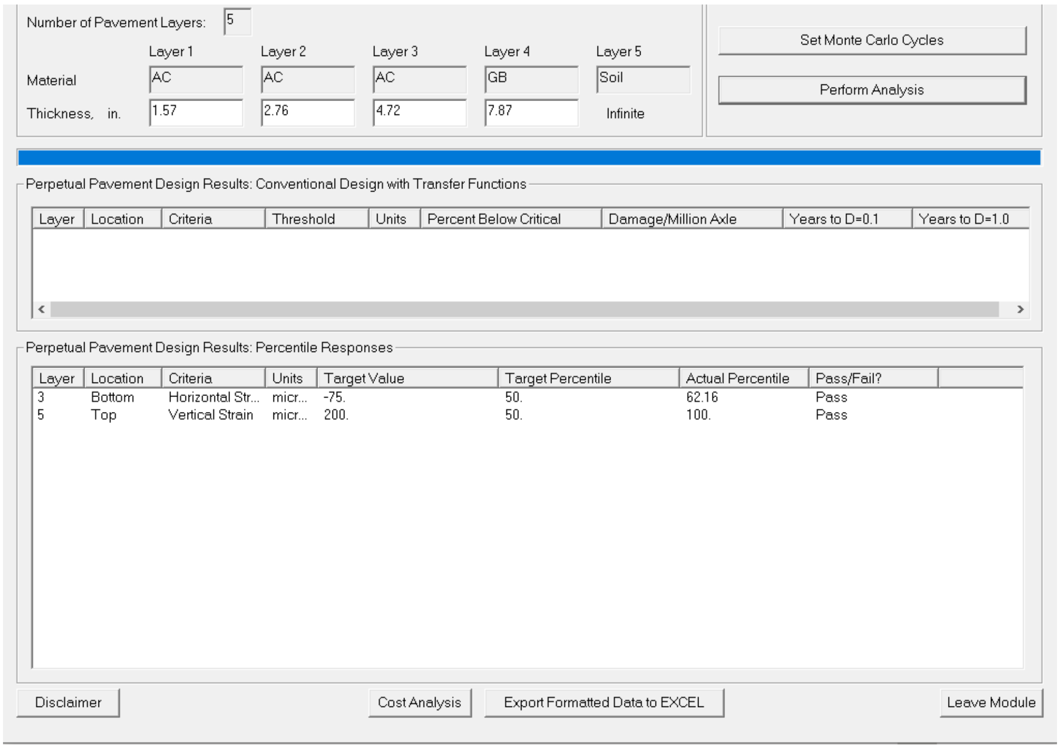


Figure n. 21 – PerRoad software output interface using percentiles

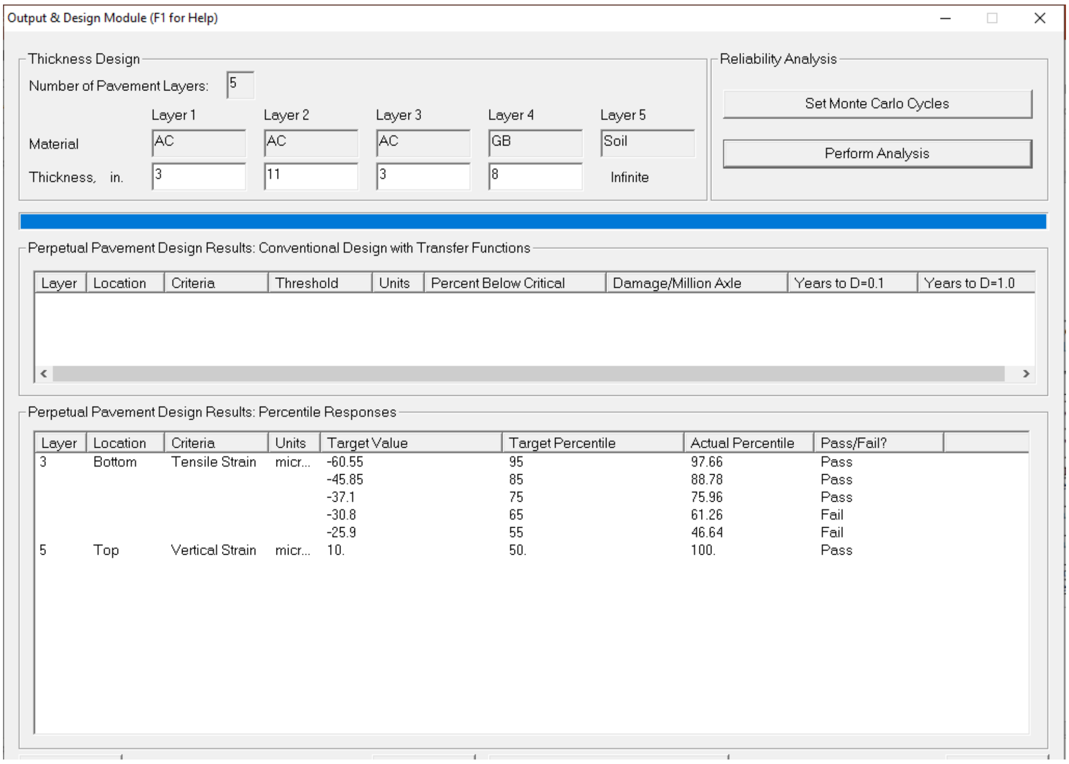


Figure n. 22 – PerRoad software output interface using horizontal strain distribution

6.2 MEAPA Software

Mechanistic-Empirical Asphalt Pavement Analysis (MEAPA) is a web pavement design application based on the traditional Mechanistic-Empirical design method, developed by the Michigan State University (MSU).

The access can be done from any browser and the data are stored in a cloud.

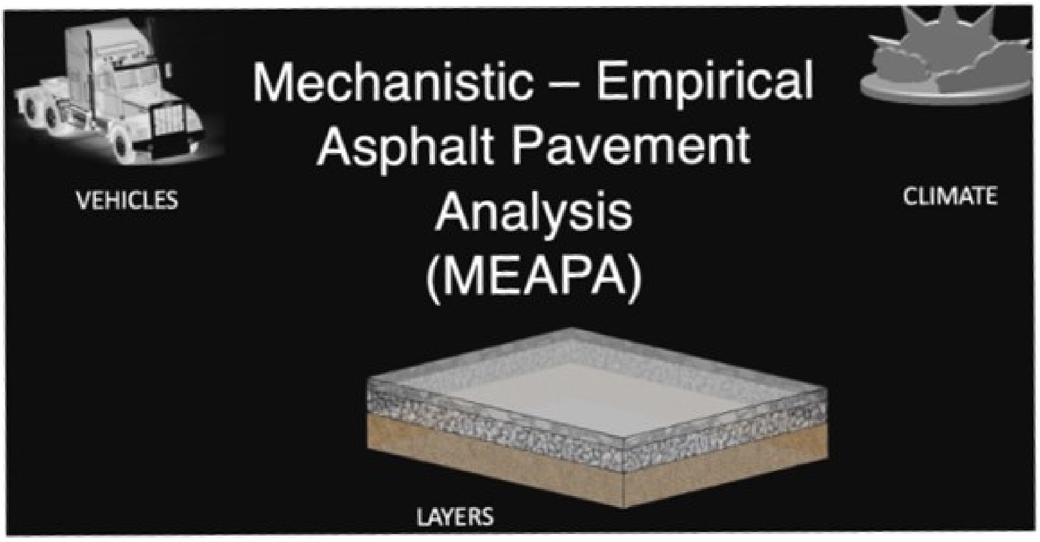


Figure n. 23 – MEAPA software (*Kutay et al.,2020*)

6.2.1 MEAPA input interface

Entering with the name of the city and selecting the state automatically the software will consider the closest climate station. In fact, the software includes some climatic stations all around the world.

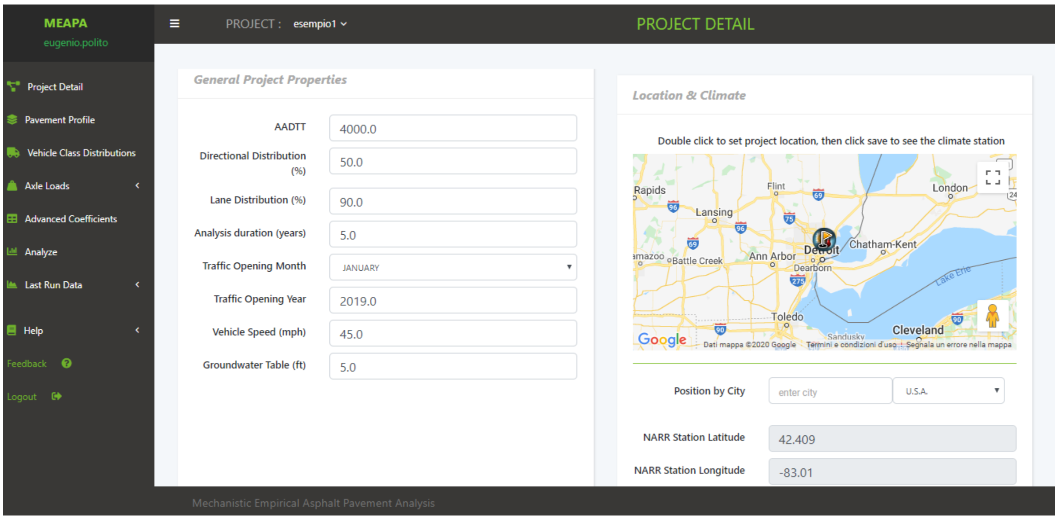


Figure n. 24 – MEAPA software project detail page

The climatic model in MEAPA is very similar to the Enhanced Integrated Climatic Model (EICM) in the MEPDG. The EICM in the MEPDG includes the following three major components:

• Prediction of temperature with depth is based on the model:

- The Climatic-Materials-Structural Model (CMS Model) developed at the University of Illinois (*Dempsey, 1969*)

• Prediction of moisture with depth is based on the model:

- The Infiltration and Drainage Model (ID Model) developed at the Texas A&M University (*R L Lytton et al., 1993*)

• Prediction of frost heave:

- The CRREL Frost Heave and Thaw Settlement Model (CRREL Model) developed at the United States Anny Cold Regions Research and Engineering Laboratory (CRREL).

Inside the pavement profile page, selecting each layer, it is possible to enter with the general information and set from one side the mixture dynamic modulus and phase angle, and on the other side the binder dynamic shear modulus and phase angle. You can also copy the table and past on excel, enter with own values or is possible to select the values from the database.

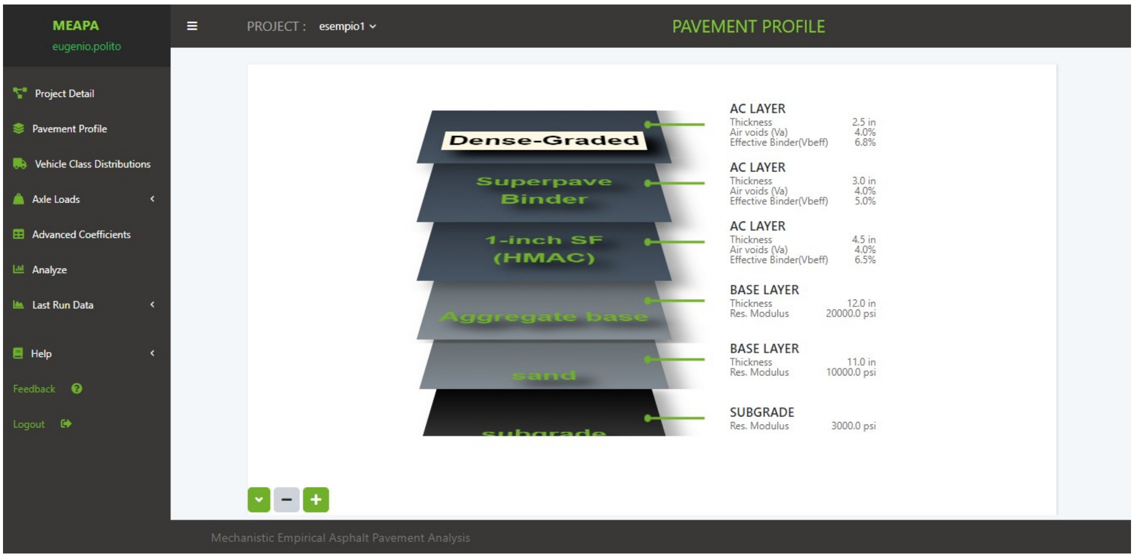


Figure n. 25 – MEAPA software pavement profile page

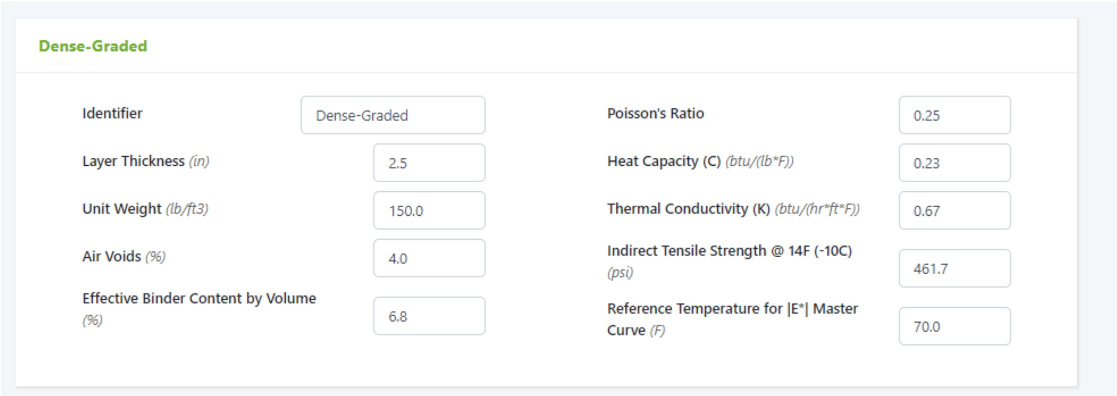
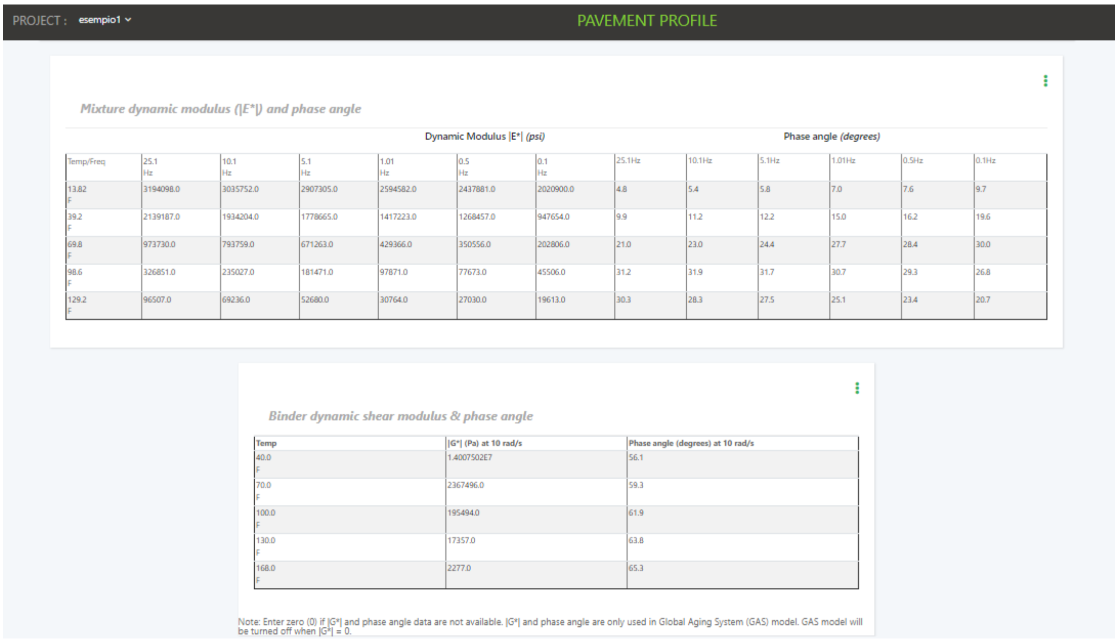


Figure n. 26 – MEAPA software layer information window



	Dynamic Modulus |E*| (psi)						Phase angle (degrees)					
Temp/Freq	25.1 Hz	10.1 Hz	5.1 Hz	1.01 Hz	0.5 Hz	0.1 Hz	25.1Hz	10.1Hz	5.1Hz	1.01Hz	0.5Hz	0.1Hz
13.82 F	3194098.0	3035752.0	2907305.0	2594582.0	2437881.0	2020900.0	4.8	5.4	5.8	7.0	7.6	9.7
39.2 F	2139187.0	1934204.0	1778665.0	1417223.0	1268457.0	947654.0	9.9	11.2	12.2	15.0	16.2	19.6
69.8 F	973730.0	793759.0	671263.0	429366.0	350556.0	202806.0	21.0	23.0	24.4	27.7	28.4	30.0
98.6 F	326851.0	235027.0	181471.0	97871.0	77673.0	45506.0	31.2	31.9	31.7	30.7	29.3	26.8
129.2 F	96507.0	69236.0	52680.0	30764.0	27030.0	19613.0	30.3	28.3	27.5	25.1	23.4	20.7

Binder dynamic shear modulus & phase angle

Temp	|G*| (Pa) at 10 rad/s	Phase angle (degrees) at 10 rad/s
40.0 F	1.4007502E7	56.1
70.0 F	2367496.0	59.3
100.0 F	195494.0	61.9
130.0 F	17357.0	63.8
168.0 F	2277.0	65.3

Note: Enter zero (0) if |G*| and phase angle data are not available. |G*| and phase angle are only used in Global Aging System (GAS) model. GAS model will be turned off when |G*| = 0.

Figure n. 27 – MEAPA software Mixture and binder dynamic modulus windows

Another aspect regards the vehicle class distribution. The classification of the vehicles is based on that one given by the Federal Highway Administration. As highlighted by the *figure n. 28* you must specify the percentage and the growth of each class.

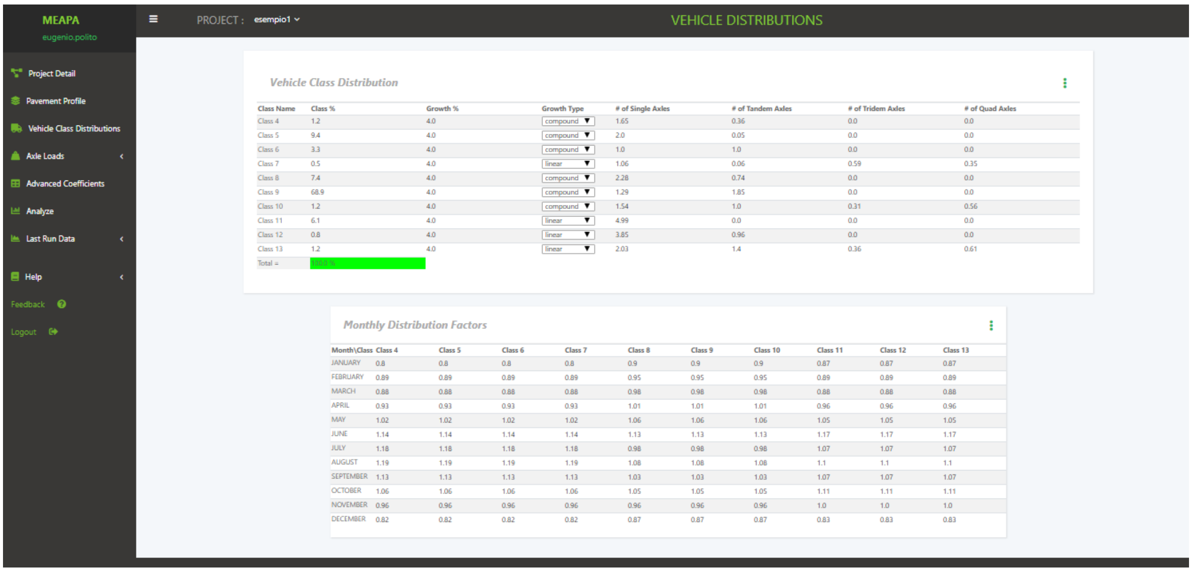


Figure n. 28 – MEAPA software vehicle class distribution page

One of the most interesting elements of MEAPA software refers to the Advanced Coefficients page.

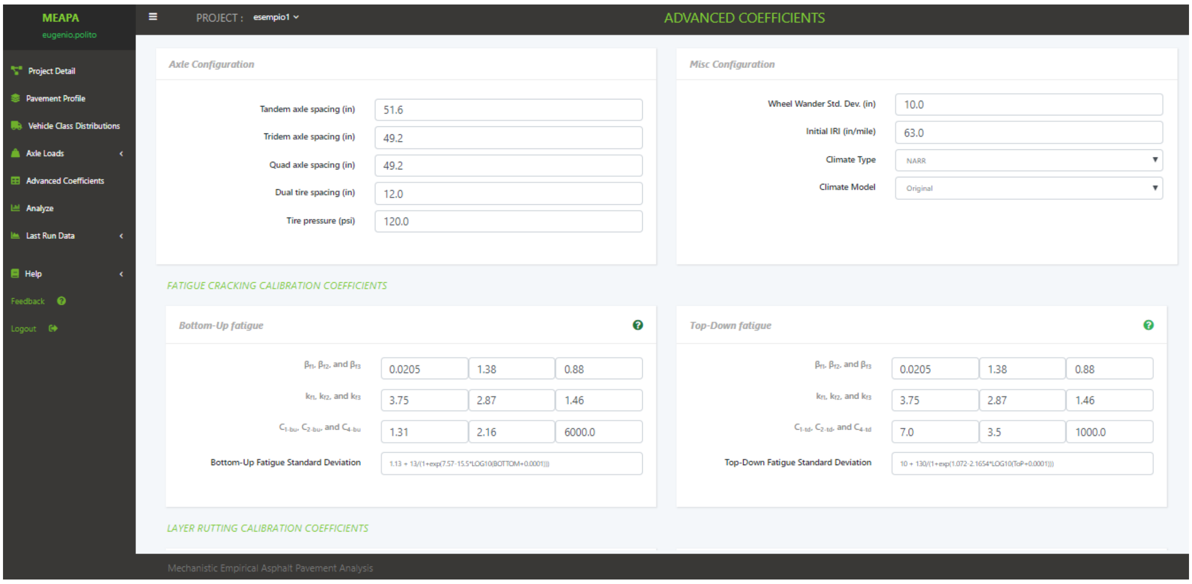


Figure n. 29 – MEAPA software advance coefficients page

In this section you can set all the coefficient for each model, such as calibration coefficients, coefficients for the transfer function, etc.

6.2.2 MEAPA analysis and output

There are five general analysis steps in MEAPA models:

1. Traffic data processing.
2. Climate data processing and running the mechanistic climatic model (MCLIM) to compute temperature with depth.
3. Perform structural analysis to compute critical strains and stresses a mechanistic procedure.
4. Use phenomenological Material Damage Models (MDMs) to compute theoretical failure condition corresponding to an analysis period for a given critical stress or strain.
5. Compute accumulation of damage.
6. Compute actual distresses using empirical transfer functions.

Steps 1 and 2 are generally common to all the pavement types. Steps 3 through 6 are implemented in different ways for different types of the pavements. Subsequent sections include the implementation details and the basic models used for each pavement type (*Kutay et al.,2020*).

Before to start the analysis, it is necessary to set:

- Distress save period (months)
- Structural response save period (months)

In other words, for instance considering a structural response save period equal to 6 months, the software will save the distresses / 3D structural response considering single axles, tandem axles, tridem axles and quad axles every 6 month. Of course, decreasing the save period will increase the amount of data

Depending on the type of structure different output are available. The following distresses are computed for the pavement type AC-GB (Asphalt Concrete over Gravel Base):

- AC top-down fatigue cracking (ft/mile)
- AC bottom-up fatigue cracking (%)
- AC thermal cracking (ft/mile)
- Rutting – AC, base, subbase, subgrade (in)
- International Roughness Index (IRI) (in/mile)

The general steps of the algorithm are as follows:

1. Development of the |E*| master curves for the AC layer(s)
2. Sub-layering of the structure
3. Calculating equivalent frequencies and load correction factors using the MEPDG procedure
4. Running the climatic model and obtaining temperature at the center of each sublayer
5. Running the Global Aging System (GAS) model
6. Calculation of the elastic moduli in five quintiles in a given month using the temperature at each quintile, frequency and the |E*| master curve coefficients.

7. Defining the critical strain locations for each type of distress
8. Running the thermal cracking model
9. Running the MatLEA structural response model at each quintile of each month, then:
    a. Compute the top-down cracking increment
    b. Compute the bottom-up cracking increment
    c. Compute the AC rutting increment
    d. Compute the base/subbase rutting (same model) increment
    e. Compute the subgrade rutting increment.
    f. Summation of the distresses computed during 5 quintiles of each month to compute the cumulative monthly distresses.
10. Compute IRI values for each month

The following is the list of distresses computed for the AC-CSM and AC-CSM-GB pavement types:

1. AC reflective cracking due to the fatigue damage in the CSM layer
2. AC top-down fatigue cracking (ft/mile)
3. AC bottom-up fatigue cracking (%)
4. AC thermal cracking (ft/mile)
5. Rutting – AC, base, subbase, subgrade (in)
6. International Roughness Index (IRI) (in/mile)

The general steps of the algorithm are as follows:

1. Development of the |E*| master curves for the AC layer(s)
2. Sublayering of the structure
3. Calculating equivalent frequencies and load correction factors using the MEPDG procedure
4. Running the climatic model and obtaining temperature at the center of each sublayer
5. Running the Global Aging System (GAS) model
6. Calculation of the elastic moduli in five quintiles in a given month using the temperature at each quintile, frequency and the |E*| master curve coefficients.
7. Defining the critical strain locations for each type of distress
8. Running the thermal cracking model
9. Running the MatLEA structural response model at each quintile of each month, then:
    a. Compute the top-down cracking increment
    b. Compute the bottom-up cracking increment
    c. Compute the AC rutting increment
    d. Compute the CSM layer damage and cracking increment
    e. Compute the base/subbase rutting (same model) increment
    f. Compute the subgrade rutting increment.
    g. Summation of the distresses computed during 5 quintiles of each month to compute the cumulative monthly distresses.
10. Compute IRI values for each month

In the analysis the pavement layers are sub-layered into several layers. This is needed for:

- Calculation of temperature, frequency and then the moduli of each AC sublayer using the |E* |master curve coefficients

- Calculation of the rutting at the centre of each sublayer in all pavement layers
- Calculation of thermal stresses at the centre of each sublayer in thermal cracking model.

The sub-layering is done using the following rules:

- Top layer:

  - If the thickness is greater than 1.5”, subdivide into layers with 0.5”, 0.5”, 1” layers and the remaining thickness. For example, if the thickness is 1.75”, the sublayers are 0.5”, 0.5” and 0.75”.
  - If the thickness is 4.25”, the sublayers are 0.5”, 0.5”, 0.25”, 1”, 1” and 1”. - If the thickness is less than 1.5”, there is no sub-layering. Entire layer is treated as one sublayer.

- Subsequent layers:

  - If the thickness is greater than 2”, subdivide into multiple 2” sublayers and remaining thickness. One exception is that if the remaining thickness is between 2” and 4”, entire remaining thickness is treated as one sublayer.
  - If the thickness is less than 2”, there is no sublayering. Entire layer is treated as one sublayer.

Then, you can choose to download a PDF report in which are summarized the main information, run input and output about the pavement, or if you want to have a look in more detailed way to the data, is possible to download the raw input/output data

7. PaveXpress Software

PaveXpress is a free, online tool to help you create and evaluate pavement design and overlays using key engineering inputs, based on AASHTO 1993 and 1998 supplement pavement design process.

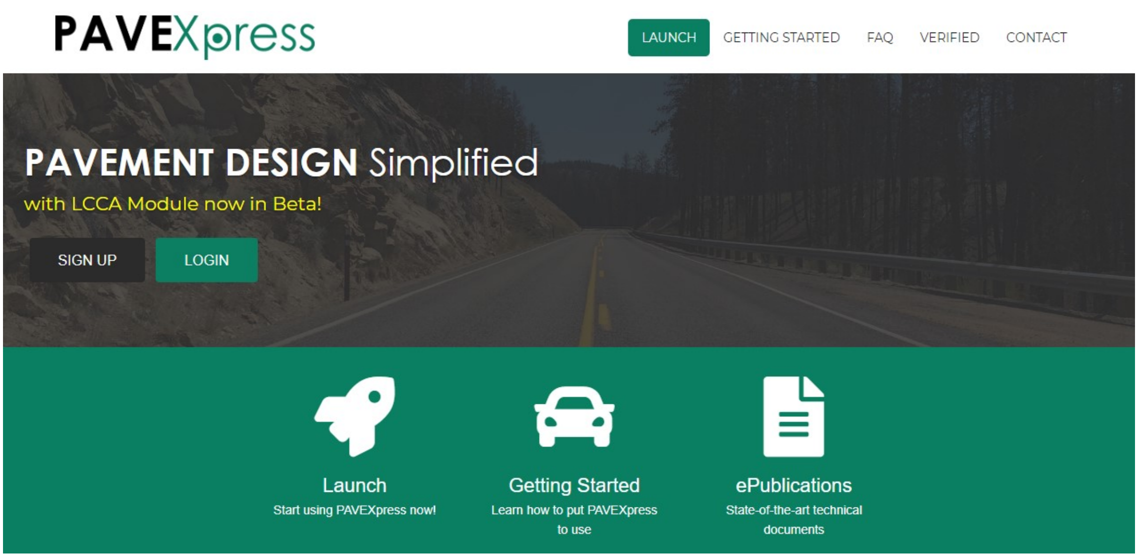


Figure n. 30 – PaveXpress software home page

7.1 PaveXpress analysis and input interface

In PaveXpress there four possible scenarios:

- Determine Pavement Structure
- Analyse Pavement Structure
- Estimate Material Cost
- Calculate Life-Cycle cost

For the purposes of this thesis we will focus mainly on the first two.

For what concern the determination of a pavement structure, there are different step that you must follow, starting from the general information about the project, such as the type of road, the type of pavement, etc., and adding the classical input design parameters, as showed in the following *figure n. 31*:

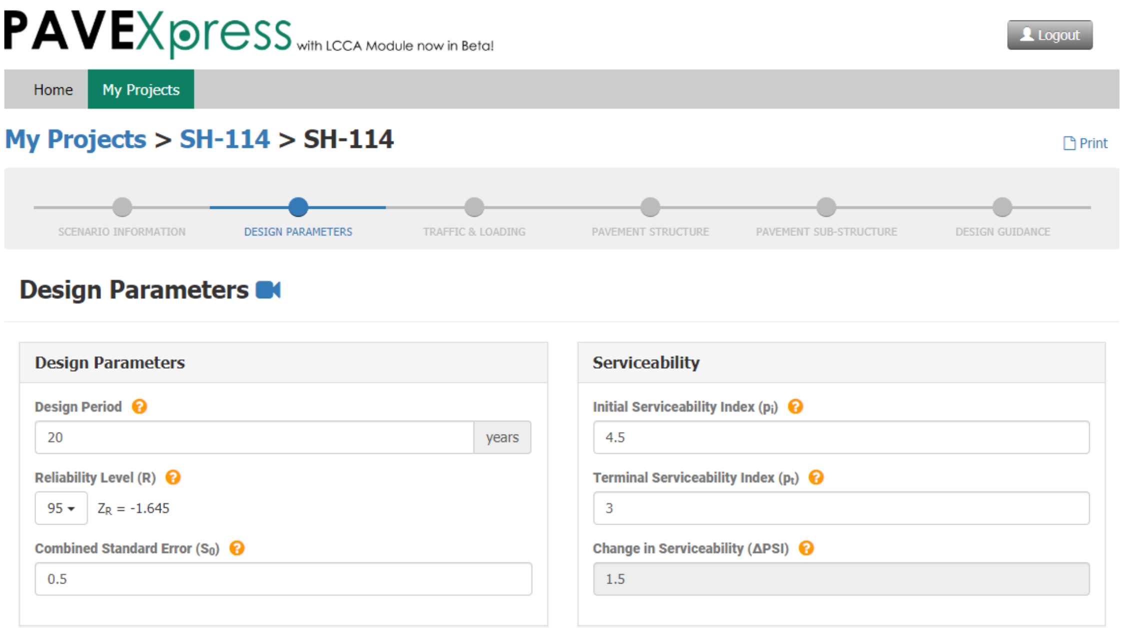


Figure n. 31 – PaveXpress software design parameters page

For the traffic condition you can directly enter with the value of ESAL, otherwise putting the general inputs used to calculate the ESAL, the software will calculate it automatically.

Instead, for the asphalt layers you must specify the layer coefficient, the drainage and the typical thickness. In case of the base, the thickness is the really unknown, which is that one that we need to satisfy the traffic over the analysis period.

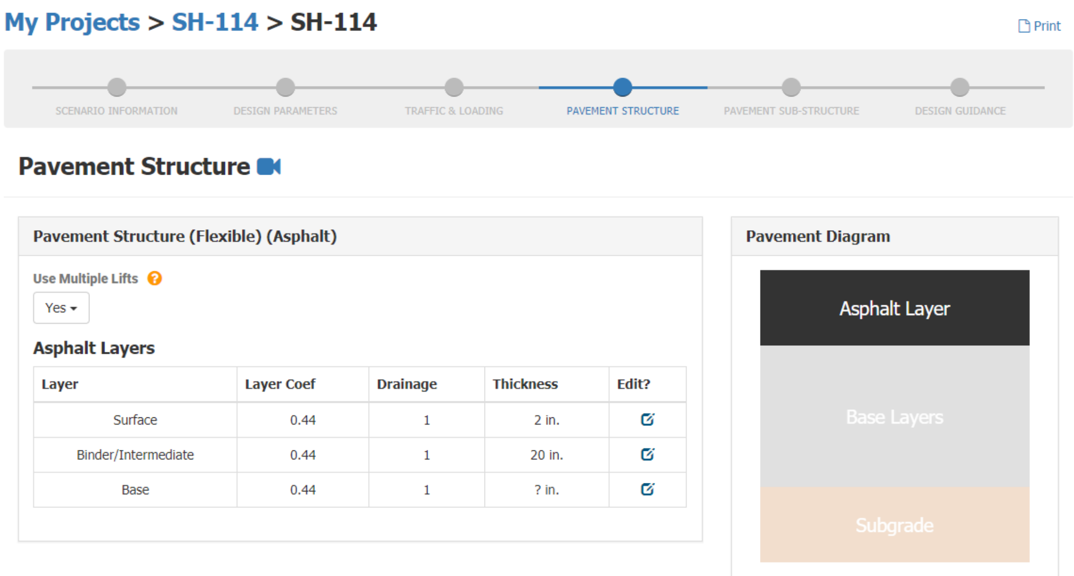


Figure n. 32 – PaveXpress software asphalt pavement structure page

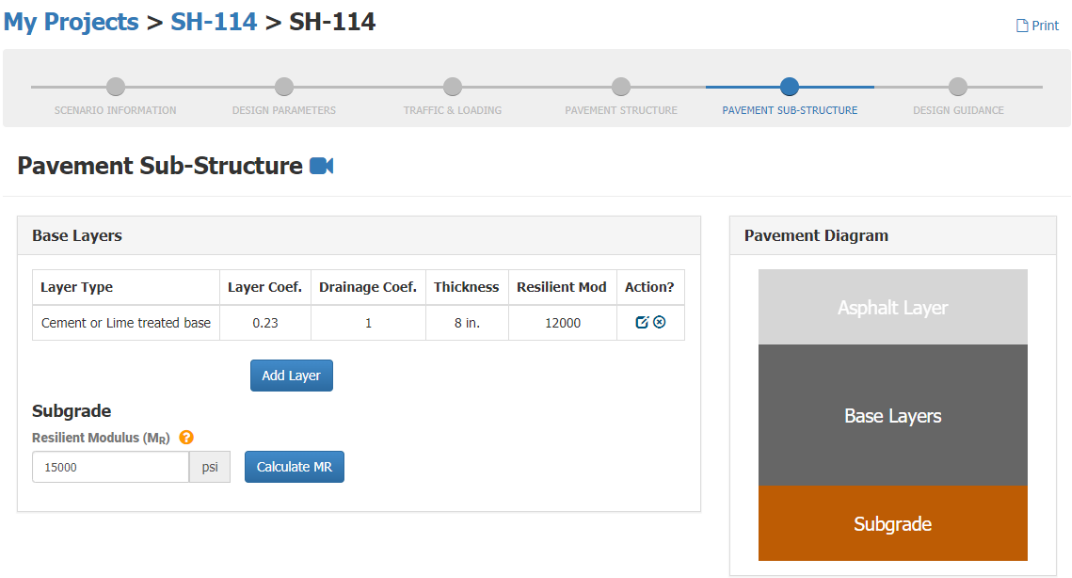


Figure n. 33 – PaveXpress software pavement sub-structure page

Then, based on the subgrade strength and traffic over time, the software evaluates a minimum value of structural number. In this way, PaveXpress will adjust the pavement thickness and in particular the base, in a way to satisfy the required minimum design SN.

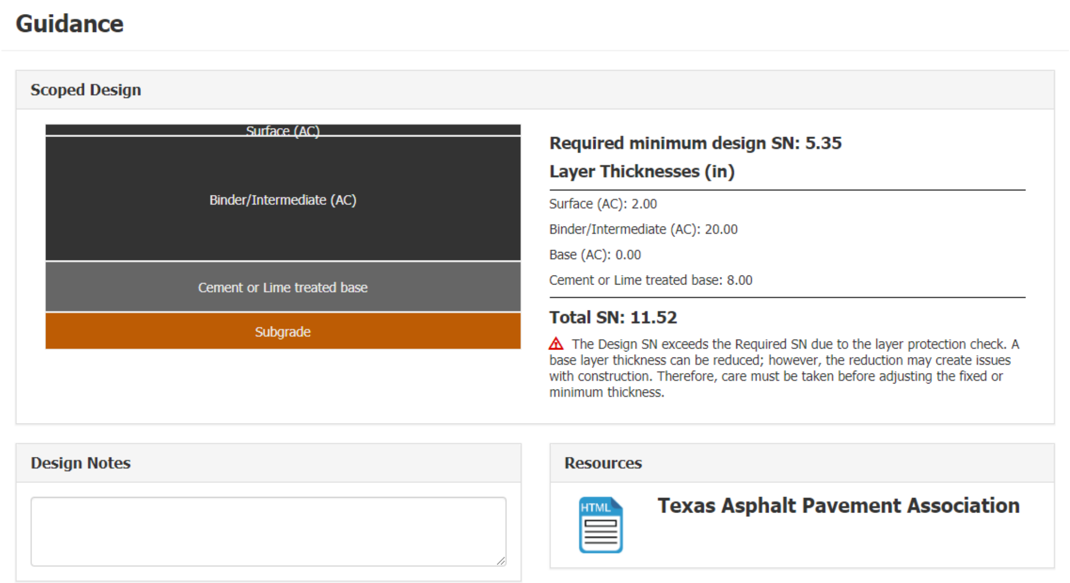


Figure n. 34 – PaveXpress software guidance page

The other possible scenario is the analysis of a pavement structure. You have to enter with all the general information about the pavement structure.

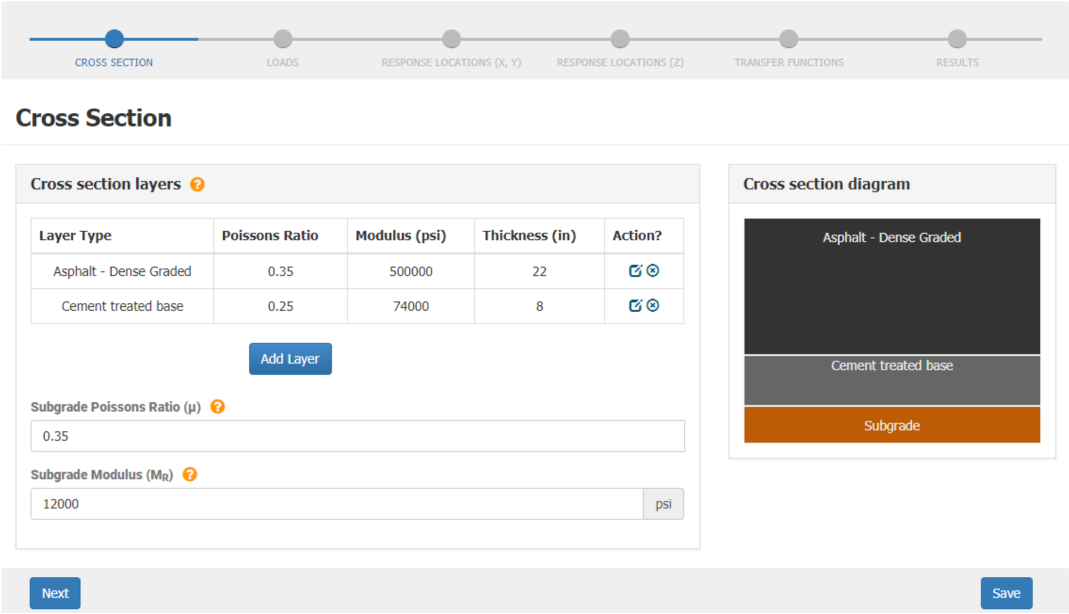


Figure n. 35 – PaveXpress software cross section page

For the analysis it is necessary to select the type of loads and the response location. In PaveXpress there are 6 load configurations:

- 3S2
- Typical Steer Axle
- Typical Single Axle with Dual Tires
- Typical Tandem with Single Tires
- Typical Tandem with Dual Tires
- Typical Tridem with Dual Tires

Moreover, there is the possibility to select the exact location first in the 2D space with respect to the loading points and then along the z direction.

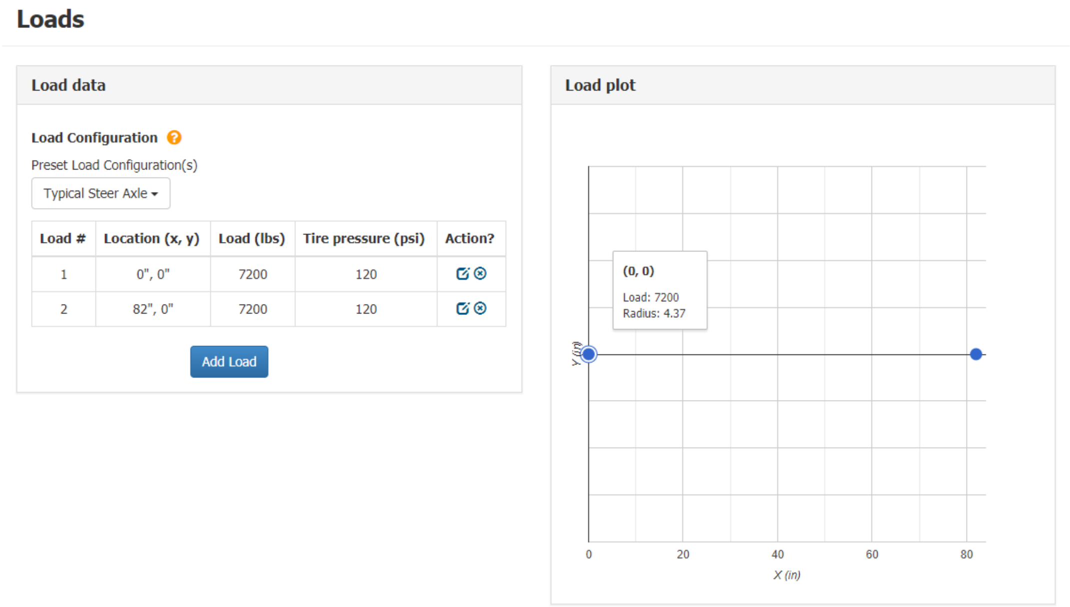


Figure n. 36 – PaveXpress software loads page

Another important aspect regards the failure criteria, in particular for the fatigue and structural rutting. In PaveXpress there is the possibility to select different models. For the fatigue you can select:

- Minnesota Do/OT model
- Per Road model
- Generalized model

The general equation is here reported:

$$Nf = (a \times 10\text{-}6)\ (1/\varepsilon t)\ ^\wedge\ b \qquad (4)$$

Each model has a different coefficient:

- Minnesota Do/OT model: (a = 2,83; b = 3,206)
- Per Road model: (a = 2,83; b = 3,148)

Otherwise, selecting the generalized model, you can set custom values.

For the rutting, there are two possible rutting models to choose from:

- AI model, published in 1982 by the Asphalt Institute
- Generalized model

The general equation is:

$$Nr = (a)\ (10\text{-}6/\varepsilon v)\ ^\wedge\ b \qquad (5)$$

Where, for the Asphalt Institute model the coefficients are:

- a = 1,077 E^18
- b = 4,4843

Of course, in case you are performing a Perpetual Pavement analysis, you can set an endurance limit.

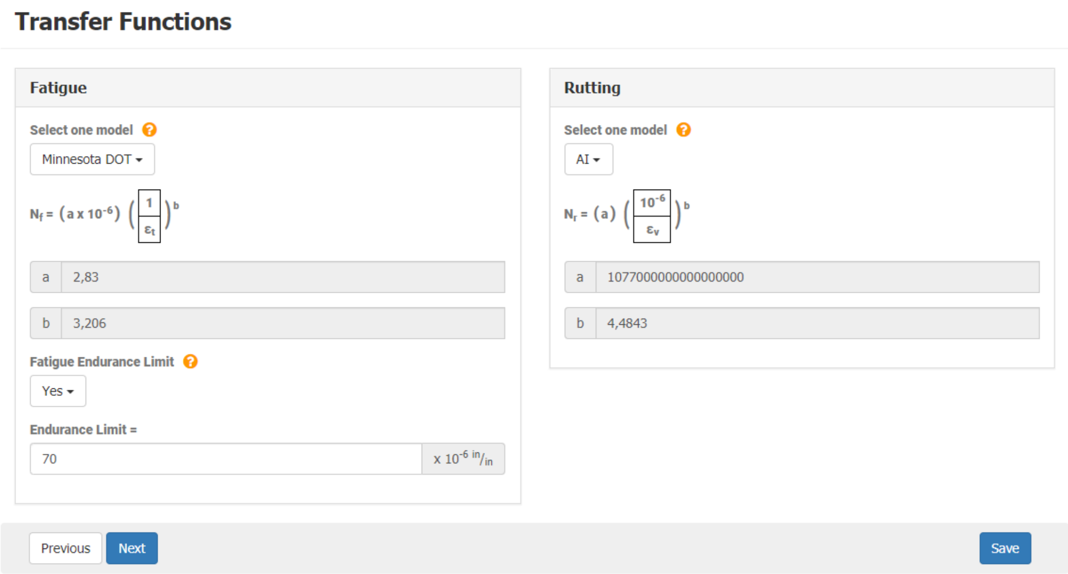


Figure n. 37 – PaveXpress software transfer function page

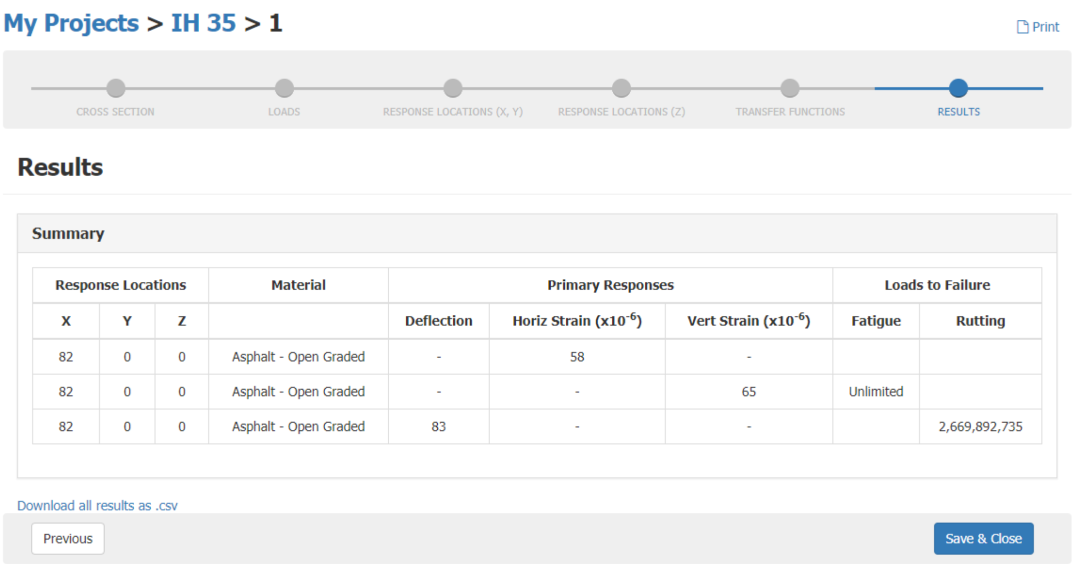
7.2 PaveXpress output interface

After the analysis, a first brief summary comes out in which there are the primary response predictions and information about loads to failure referred to just some of the all locations selected.

The primary response predictions are:

- Deflection
- Horizontal strain (x10-6)
- Vertical strain (x10-6)

For the loads to failure outputs, it is referred to fatigue cracking and structural rutting.



Response Locations			Material	Primary Responses			Loads to Failure	
X	Y	Z		Deflection	Horiz Strain ($x10^{-6}$)	Vert Strain ($x10^{-6}$)	Fatigue	Rutting
82	0	0	Asphalt - Open Graded	-	58	-		
82	0	0	Asphalt - Open Graded	-	-	65	Unlimited	
82	0	0	Asphalt - Open Graded	83	-	-		2,669,892,735

Figure n. 38 – PaveXpress software results page

Then, as in MEAPA, there is the possibility to download a PDF file in which there are the general information about the pavement. Otherwise, you can download a ".cvs" if you want to have a look in a more detailed way to the output. In this last case the outputs that you can obtain from the analysis are:

- Normal stress along x
- Normal stress along y
- Normal stress along z
- Shear stress along yz
- Shear stress along xz
- Shear stress along xy
- Major principal stress
- Intermediate principal stress
- Minor principal stress

- Major principal strain
- intermediate principal strain
- minor principal strain
- deflection along x
- deflection along y
- deflection along z
- normal strain along x
- normal strain along y
- normal strain along z

Lastly, in the *table n. 2* are summarized the general information about the different software in order to have comparison between them.

Software	PerRoad	MEAPA	PaveXpress
Material proprieties	Number of layers PG Moduli Poisson ratio Thickness	Layer Thickness Unit Weight Air Voids Effective Binder Content by Volume Poisson's Ratio Heat Capacity Thermal Conductivity Indirect Tensile Strength Reference Temperature for |E*| Master Curve Mixture dynamic modulus (|E*|) Binder dynamic shear modulus Phase angle	Moduli Poisson ratio Thickness Layer Coef. Drainage Coef.
Climatic Model	Seasonal Information	CMS	-
Traffic	Two-Way AADT % Trucks % Trucks in Design Lane Axle Groups/Day % Truck Growth Directional Distribution Vehicle type distribution Loading configuration Axle load distribution	Vehicle Class Distribution Growth % Growth Type Average number of axles per vehicle Monthly Distribution Factors	AADT Load equivalency factor Design period Traffic growth rate
Transfer Functions	Custom	Custom	Minnesota Do/OT Per Road Asphalt Institute Custom

Outputs	Horizontal stress Vertical stress Principal stress Horizontal strain Vertical strain Principal strain Vertical deflection Horizontal strain distribution	AC top-down fatigue cracking AC bottom-up fatigue cracking AC thermal cracking Rutting – AC, base, subbase, subgrade International Roughness Index (IRI)	Normal stress Shear stress Major principal stress Intermediate principal stress Minor principal stress Major principal strain Intermediate principal strain Minor principal strain Deflection Normal strain

Table n. 2 – Perpetual Pavement software comparison

7. Case studies

In order to compare the results obtained with the software described in the previous paragraphs, a first case study in Texas of Perpetual Pavement, already designed and analysed with the use of other software, has been considered.

Moreover, in the second part, a case study in Italy of a typical heavily traffic highway has been analysed with the aim to compare the results obtained using the classic design of flexible pavement and a hypothetical perpetual pavement, in order to evaluate the differences in terms of material thickness and the relative structural improvements.

7.1 IH-35 Laredo

The first case study is the IH-35. Interstate 35 (abbreviated I-35 or IH-35) in Texas is a major north-south Interstate Highway running from Laredo near the United States-Mexico border to the Red River north of Gainesville where it crosses into Oklahoma.

Interstate 35 is one of the 10 perpetual pavement (PP) sections in-service that has been designing and constructing in Texas.

For the design and the M-E checks they have used the software FPS 21. In the following *table n. 2* is indicated the initial design, with the pavement structure, and the predicted horizontal and vertical strain.

PP structure	3-inch SMA + 3-inch(¾") SFHMA + 8-inch (1") SFHMA + 3-inch RBL
Total HMA thickness	17 inches
Tensile strains (≤ 70 με)	34 με
Compressive strains (≤ 200 με)	103 με

Table n. 3 – IH-35 Perpetual Pavement structure and predicted responses

The traffic and loading condition input are summarized in the following *figure n. 39*:

Project No.	Highway	CSJ#	Distrct	County	ADT Begin	ADT End	%Traffic Growth	% Trucks	Σ ESALs (million)	# Lanes	Comment
1	IH 35	0018-05-062	Laredo	Webb (Price)	18,900 (2001)	34,800 (2024)	3%	30.4	18.508	6 total	
2	IH 35	0018-02-049	Laredo	La Salle (Zumwalt02)	11,900 (2002)	21,200 (2022)	3%	46.2	26.488	4 total	
3	IH 35	0018-01-063	Laredo	La Salle (Gilbert)	12,400 (2001)	23,100 (2021)	3%	36.1	24.097	4 total	
4	IH 35	0017-08-067	Laredo	La Salle (Zumwalt01)	NB only 6,600 (2001)	NB only 11,200 (2021)	3%	42.2	29.543	2	Only the NB lanes were rebuilt into a perpetual pavement.
5	IH 35	0016-04-091	San Antonio	Comal (Old San Antonio)	60,500 (2004)	95,200 (2024)	2%	16	41.593	6 to 8 total	
6	IH 35	0016-04-094	San Antonio	Comal (New Braunfels)	60,500 (2004)	95,200 (2024)	2%	16	41.593	6 to 8 total	
7	IH 35	0015-01-164	Waco	McLennan	64,750	112,300	3%	17.7	74.172	6 total	
8	IH 35	0048-09-023	Waco	Hill	41000	70900	3%	27.5	75.209	8	
9	SH 114	0353-01-026	Fort Worth	Wise	EB: 7,500 (2004) WB: 7,500 (2004)	EB: 15,000 (2024) WB: 15,000 (2024)	4%	EB: 27.3 WB: 23.7	EB: 37.242 WB: 13.086	EB: 2 WB: 2	WIM station on this section at about TRM 580+0.950.
10	SH 114	0353-01-026	Fort Worth	Wise	EB: 7,500 (2004) WB: 7,500 (2004)	EB: 15,000 (2024) WB: 15,000 (2024)	4%	EB: 27.3 WB: 23.7	EB: 37.242 WB: 13.086	EB: 2 WB: 2	Section starts at traffic lights.

Figure n. 39 – Traffic and loading condition input (*Lubinda F. et al.*)

Considering all this information, it's possible to use PerRoad, MEAPA and PaveXpress in order to compare the results.

7.1.1 Analysis of IH-35 with PerRoad software

As already mentioned, each software has a different approach and in particular, requires different input information. PerRoad doesn't consider a particular climate model but it divides the whole year in four main seasons, and you must refer to a mean average air temperature expressed, in Fahrenheit, for each season. This was obtained thanks to U.S. climate data, considering the maximum and minimum average monthly temperature for a given year and averaging over the 4 seasons.

For the AC layers, the Performance Grade is the same as indicated in the initial design. Instead for what concern the modulus, the temperature correction button was turned off and it was used an average value of the modulus with respect to the range of moduli suggested for each layer by Tdot.

The modulus and thickness coefficients of variability were left as a default.

For the bottom-up fatigue cracking, both horizontal strain distribution and single threshold criterion was selected. So, considering a reference value of 35 microstrain, which is the average value obtained in the initial design of IH-35, the software generated target percentiles equal to:

Percentile	Microstrain
95^{th}	60.55
85^{th}	45.85
75^{th}	37.1
65^{th}	30.8
55^{th}	25.9

Table n. 4 – Reference percentile value of 35 microstrain

However, for the vertical strain evaluation some problems came up. For the structural rutting it must be considered the vertical strain at the top of the subgrade, but in PerRoad as you select a pavement with 5 layer and setting the compressive strain at the top of the subgrade, the result is that it passes the control even if a minimum value of microstrain is considered, it is as if the vertical strain in that point is equal to zero. In order to overcome the problem, it is possible to aggregate the first two layers and select as a number of layers 4. In this way, considering the same performance criterion at the top of the subgrade, the software gives a more reliable result. It's possible to do this, because the software doesn't require different input such us the nominal maximum aggregate size or other, but just the PG and a reference value of the modulus

In the following *figure n. 40* is reported the structural and seasonal information window.

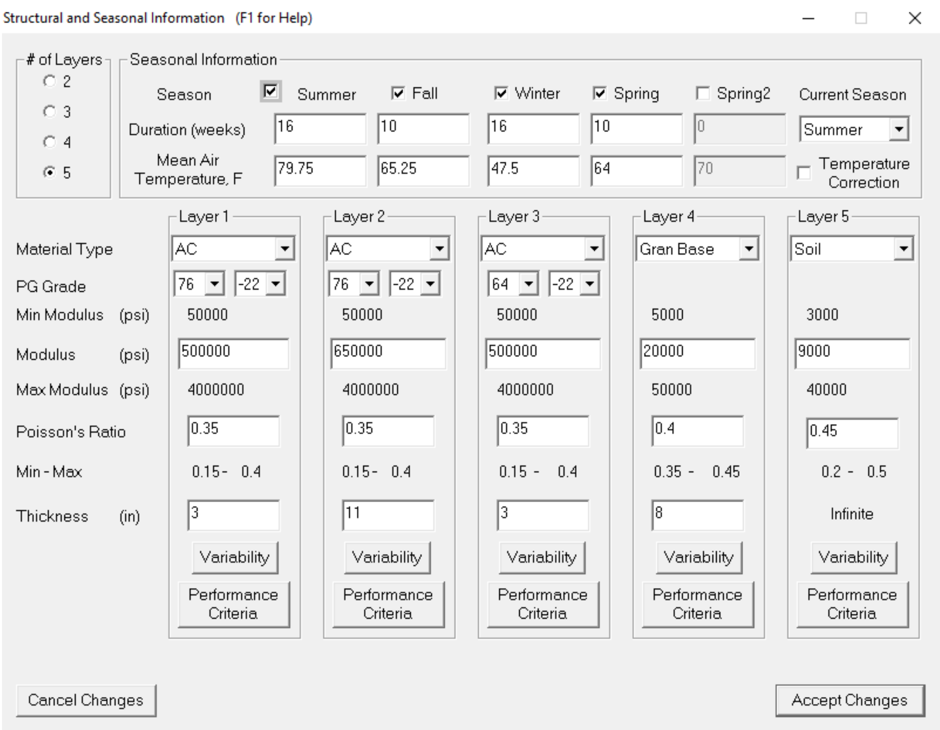


Figure n. 40 – PerRoad structural and seasonal information input page

Looking at the results, it’s possible to say that the vertical and compressive strain output are not so far from those one obtained with the use of FPS 21. Of course, the software doesn’t give a determinist result or an average value, but as mentioned before it has a probabilistic character, with the use of the percentiles.

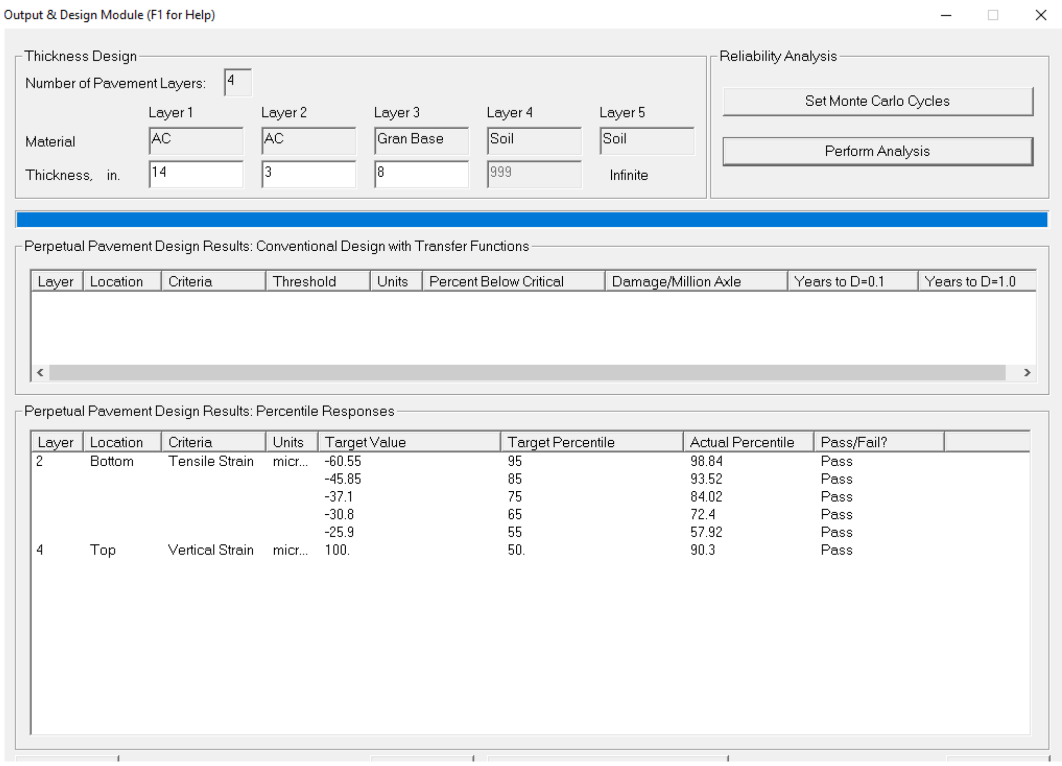


Figure n. 41 – PerRoad output and design modulus page

Of course, using the percentiles we don’t have information about the damage cumulation as in case of the transfer function. In other words, using this approach we cannot predict the number of years in which the damage is equal to 1, but simply if the all actual percentiles are above the target values , and in particular, considering 70 microstrain as threshold, it is possible to define the pavement as Perpetual Pavement.

7.1.2 Analysis of IH-35 with MEAPA software

As anticipated, with respect to PerRoad, MEAPA has a different approach.

The first step was to fill the general information about the project, with the information taken from the initial design and the given traffic condition by Tdot.

MEAPA has two version, one is free and the other is a paid version. Unfortunately, for this thesis it has been used the free version which limits the analysis period at 5 years.

As highlighted from the *figure n. 42*, inserting Laredo as location, automatically the software will find the closest climatic station.

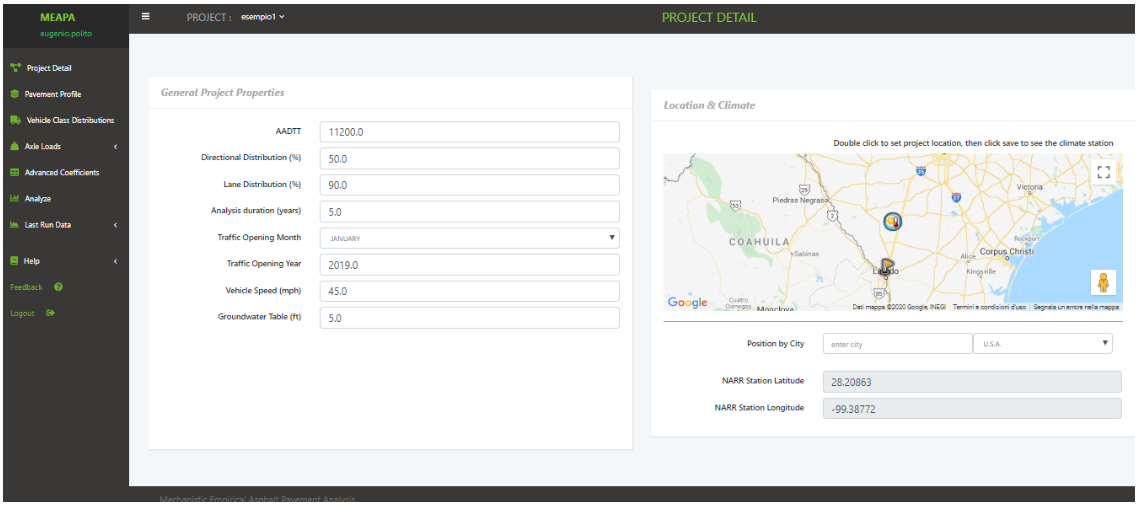


Figure n. 42 – MEAPA project detail of IH-35

In this other picture the pavement structure is showed. On the right the synthetic information about each layer are reported.

Figure n. 43 – MEAPA pavement profile of IH-35

Going more in detail, for the asphalt concrete layers a lot of input are required and unfortunately some of these are not available as input information. For this reason, some assumptions have been done; the unit weight, the heat capacity, the thermal conductivity, the indirect tensile strength @ 14F (-10C), and the reference temperature for |E*| Master Curve have been left as default value.

Moreover, not having information about the Mixture dynamic modulus (|E*|), binder dynamic shear modulus and phase angles, have been selected from the database, but also here some problems have arisen because inside the database not all the type of mixture and binder are present. The PG 76-22 12.5 mm and 19 mm NMAS gap-grade for the SMA and SFHMAC NMAS ¾ inch were not available inside the database and for this reason a PG 70-22 was used.

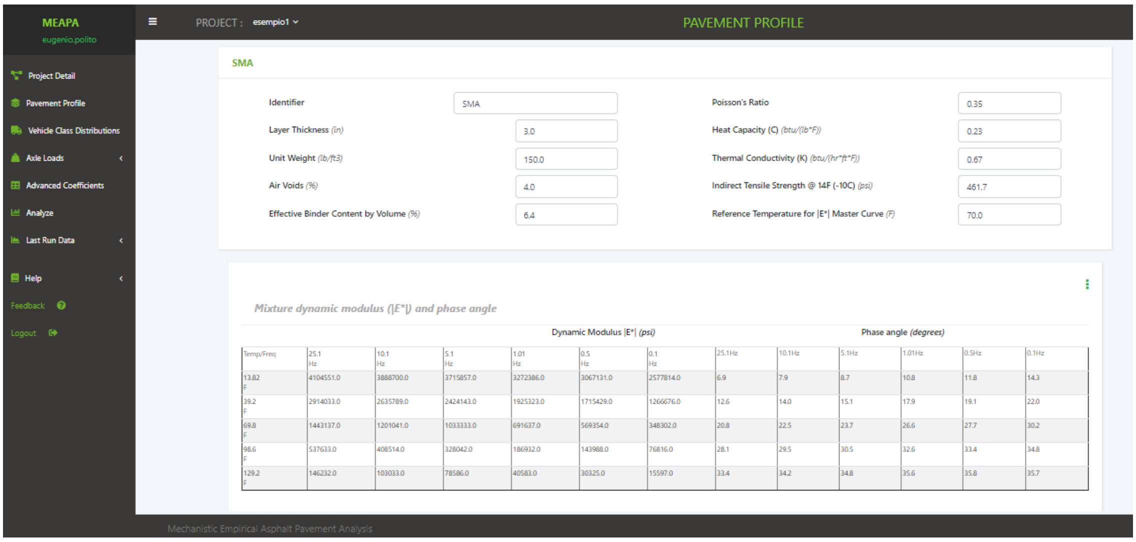
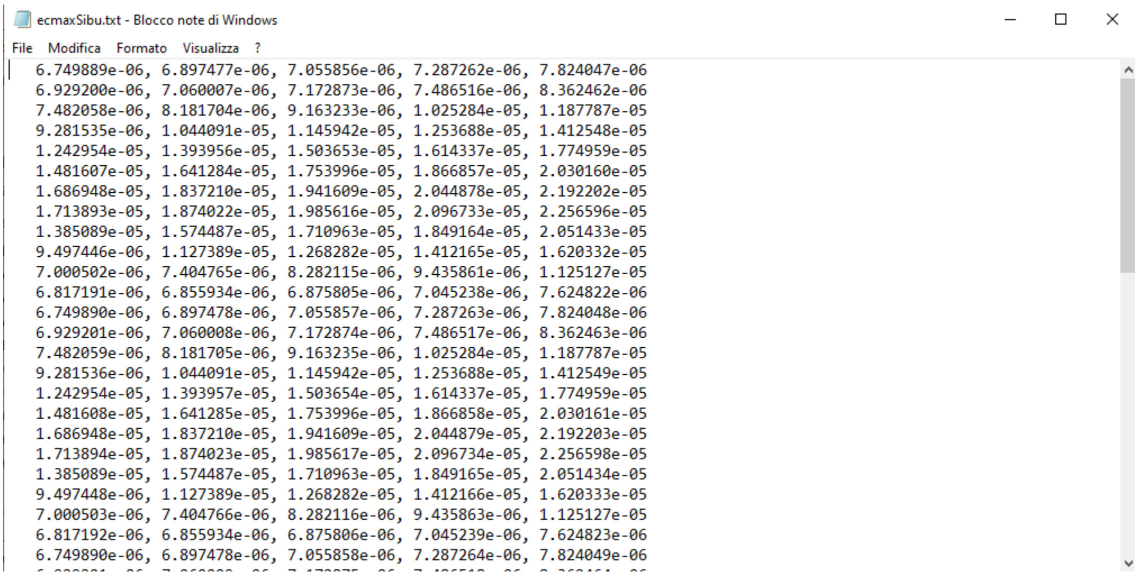


Figure n. 44 – MEAPA layer information of IH-35

Vehicle Class Distribution used was the same of a typical distribution of an interstate road.

From the several output available in MEAPA, we are interested in the maximum horizontal strain at the bottom of the asphalt layer. As highlighted on the top part of the below output window, this indicates the maximum strain considering single axle load configuration and bottom-up fatigue cracking failure criterion.

As we can see, the maximum value is about 23 microstrain, which is a little bit less with respect to the designed value with FPS 21 of 34 microstrain.



ecmaxSibu.txt - Blocco note di Windows
File Modifica Formato Visualizza ?

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7.482058e-06, 8.181704e-06, 9.163233e-06, 1.025284e-05, 1.187787e-05
9.281535e-06, 1.044091e-05, 1.145942e-05, 1.253688e-05, 1.412548e-05
1.242954e-05, 1.393956e-05, 1.503653e-05, 1.614337e-05, 1.774959e-05
1.481607e-05, 1.641284e-05, 1.753996e-05, 1.866857e-05, 2.030160e-05
1.686948e-05, 1.837210e-05, 1.941609e-05, 2.044878e-05, 2.192202e-05
1.713893e-05, 1.874022e-05, 1.985616e-05, 2.096733e-05, 2.256596e-05
1.385089e-05, 1.574487e-05, 1.710963e-05, 1.849164e-05, 2.051433e-05
9.497446e-06, 1.127389e-05, 1.268282e-05, 1.412165e-05, 1.620332e-05
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1.686948e-05, 1.837210e-05, 1.941609e-05, 2.044879e-05, 2.192203e-05
1.713894e-05, 1.874023e-05, 1.985617e-05, 2.096734e-05, 2.256598e-05
1.385089e-05, 1.574487e-05, 1.710963e-05, 1.849165e-05, 2.051434e-05
9.497448e-06, 1.127389e-05, 1.268282e-05, 1.412166e-05, 1.620333e-05
7.000503e-06, 7.404766e-06, 8.282116e-06, 9.435863e-06, 1.125127e-05
6.817192e-06, 6.855934e-06, 6.875806e-06, 7.045239e-06, 7.624823e-06
6.749890e-06, 6.897478e-06, 7.055858e-06, 7.287264e-06, 7.824049e-06
```

Figure n. 45 – MEAPA maximum horizontal strain at the bottom of the asphalt layer of IH-35

Using this type of approach for the fatigue cracking, it is necessary to compare the maximum value of strain obtained with the strain threshold of 70 microstrain, in order to define a pavement as Perpetual Pavement.

For the structural rutting MEAPA calculate the vertical compressive strains at the centre of the AC layers from the MatLEA analysis program and then it uses a formula to compute the rutting. So, in the output we don't have information about the compressive strain at the top of the subgrade, but the rutting is evaluated in the results as accumulated permanent or plastic vertical deformation in the HMA layer/sublayer, in.

7.1.3 Analysis of IH-35 with PaveXpress software

For the purpose of this thesis, in PaveXpress software just the “analysis of pavement struture scenario” has been evaluated, without considering the “determination of the pavement structure scenario”, because the pavement structure is already known.

So, after creating the pavement structure inside the software as showed in the following *figure n. 47* as pavement response locations on the x and y axis have been selected exactly the points of application of the axle loads, and along the z direction on the bottom of the last asphalt concrete layer and on the top of the subgrade.

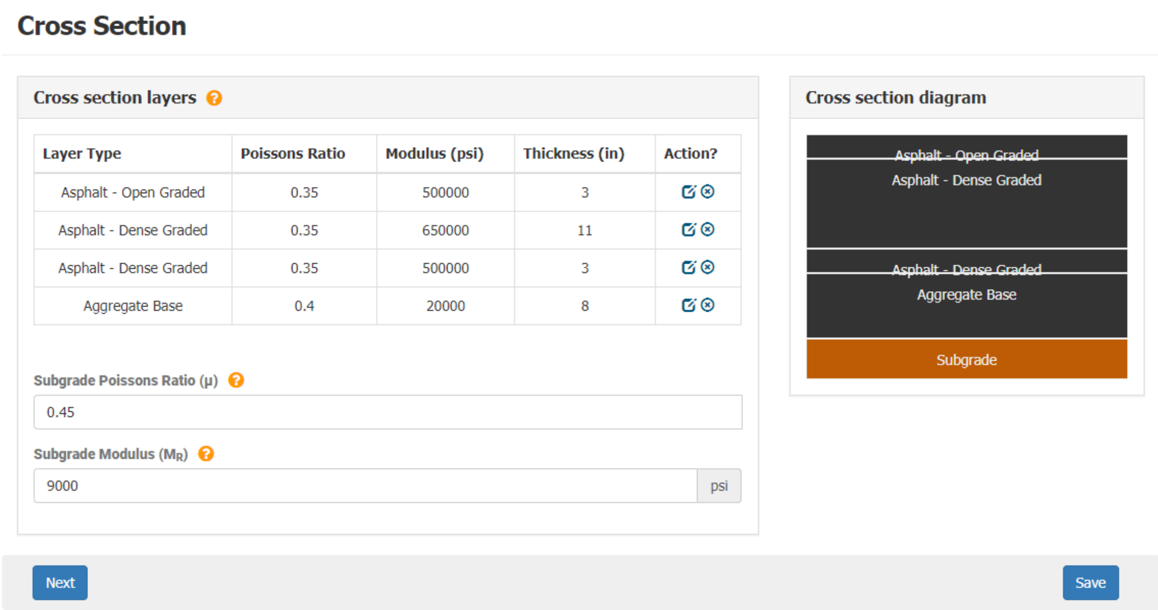

Figure n. 46 – PaveXpress cross section of IH-35

For the transfer function, the default models have been selected with their respective coefficients and a Fatigue Endurance Limit has been considered equal to 70 micro strain.

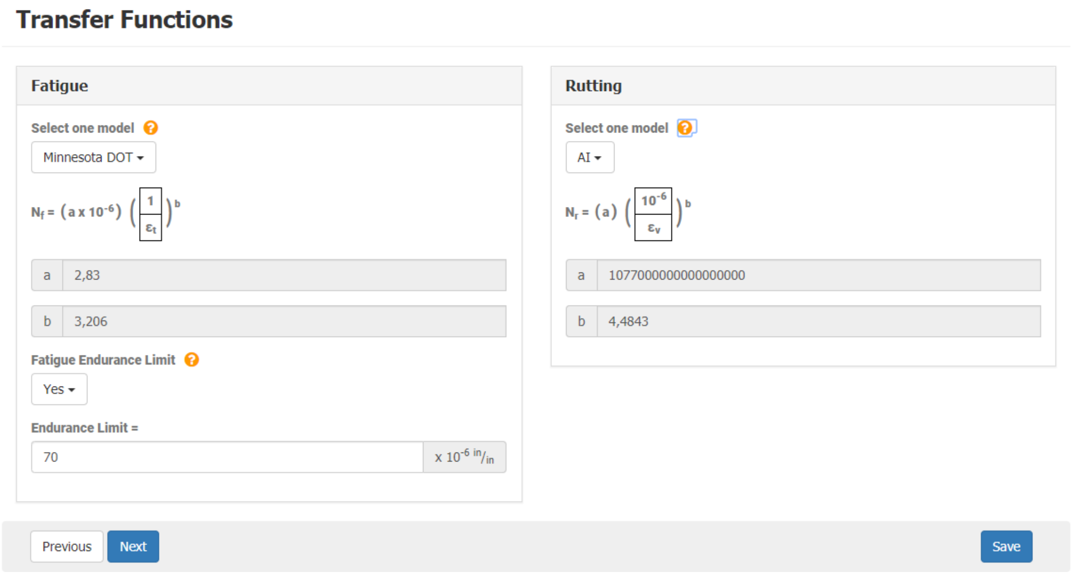

Figure n. 47 – PaveXpress transfer function of IH-35

Finally, downloading the output as ".cvs" file it has been obtained:

Horizontal strain along x-axis	35.89 με
Horizontal strain along y-axis	42.9 με
Vertical strain along z-axis	44.4 με

Table n. 5 – PaveXpress strain output

Also, in this case the results are not so far from the initial design.

7.2 Typical Italian heavily traffic highway

As already mentioned, the purpose of the analysis of this case study is to compare the results obtained using the classic design for a flexible pavement and a hypothetical perpetual pavement, in order to evaluate the differences in terms of material thickness and the relative structural improvements. In this way, a typical heavily traffic highway in Italy has been considered

One of the problems encountered in the analysis of this case study concerns traffic. The Italian roadway and vehicles classification are different from the American one. The Federal Highway Administration (FHWA) developed in the 1980s a 13-category classification, which is reported *in table n. 6*:

Class Group	Class Definition	Class Includes	Number of Axles
1	Motorcycles	Motorcycles	2
2	Passenger Cars	All cars Cars with one-axle trailers Cars with two-axle trailers	2, 3, or 4
3	Other Two-Axle Four-Tire Single-Unit Vehicles	Pick-ups and vans Pick-ups and vans with one- and two- axle trailers	2, 3, or 4
4	Buses	Two- and three-axle buses	2 or 3
5	Two-Axle, Six-Tire, Single-Unit Trucks	Two-axle trucks	2
6	Three-Axle Single-Unit Trucks	Three-axle trucks Three-axle tractors without trailers	3
7	Four or More Axle Single-Unit Trucks	Four-, five-, six- and seven-axle single-unit trucks	4 or more
8	Four or Fewer Axle Single-Trailer Trucks	Two-axle trucks pulling one- and two-axle trailers Two-axle tractors pulling one- and two-axle trailers Three-axle tractors pulling one-axle trailers	3 or 4
9	Five-Axle Single-Trailer Trucks	Two-axle tractors pulling three-axle trailers Three-axle tractors pulling two-axle trailers Three-axle trucks pulling two-axle trailers	5

Class Group	Class Definition	Class Includes	Number of Axles
10	Six or More Axle Single-Trailer Trucks	Multiple configurations	6 or more
11	Five or Fewer Axle Multi-Trailer Trucks	Multiple configurations	4 or 5
12	Six-Axle Multi-Trailer Trucks	Multiple configurations	6
13	Seven or More Axle Multi-Trailer Trucks	Multiple configurations	7 or more

Table n. 6 – FHWA vehicle classification definitions

The latter does not coincide with the typical CNR traffic spectrum reported below in *table n. 7*:

Veicolo ↓	**Strada tipo** →	**A**	**B**	**C**	**D**	**E**	**Distribuzione dei carichi in kN**						
								Avantreno			Retrotreno		
1	Furgoni medi	12,2	18,2	-	-	24,5	10			20			
2	Autocarri leggeri	-	18,2	13,1	-	-	15			30			
3	Autocarri medi	24,4	16,5	39,5	58,8	40,8	40			80			
4	Autocarri pesanti a 2 assi	14,6	-	10,5	29,4	16,3	50			110			
5	Autocarri pesanti a 3 assi	2,4	-	7,9	-	-	40			80	80		
6	Autocarri pesanti a 3 assi	12,2	-	2,6	5,9	4,15	60			100	100		
7	Autotreni ed articolati a 4 assi	2,4	-	2,6	-	-	40	90			80		80
8	Autotreni ed articolati a 4 assi	4,9	-	2,5	2,8	2,0	60	100			100		100
9	Autotreni ed articolati a 5 assi	2,4	-	2,6	-	-	40	80	80			80	80
10	Autotreni ed articolati a 5 assi	4,9	-	2,5	-	-	60	90	90			100	100
11	Autotreni a 5 assi con maxiruote	2,4	-	2,6	-	-	40	100			80	80	80
12	Autotreni a 5 assi con maxiruote	4,9	-	2,6	-	-	60	110			90	90	90
13	Mezzi d'opera	0,10	1,6	0,5	0,2	0,05	50	120			130	130	130
14	Autobus urbani	-	18,2	-	-	-	40			80			
15	Autobus urbani	-	27,3	-	-	-	60			100			
16	Autobus extraurbani	12,2	-	10,5	2,9	12,2	50			80			

Legenda: (A) - Autostrada extraurbana, (B) - Autostrada urbana o strada di scorrimento, (C) - Strada extraurbana a forte traffico, (D) - Strada extraurbana secondaria, (E) - Strada extraurbana turistica.

Table n. 7 – CNR traffic spectrum

So, in order to use the software, it is necessary to create e correlation between the two classification systems. Some useful information comes from a study done by Prof. Ciro Caliendo, whose title is "*Local Calibration and Implementation of the Mechanistic-Empirical Pavement Design Guide for Flexible Pavement Design"*

The CNR traffic spectrum is generally used in Italy for pavement design when the site measurements of truck traffic are not available. To minimize the differences between local truck traffic and MEPDG defaults, the 16 classes of Italian trucks were aggregated on the basis of the truck vehicle type, number of axles, and axle type (single, tandem, and tridem) into five of the 10 classes of trucks considered in the MEPDG. The other five classes of the MEPDG are not used in the analysis because they correspond to trucks that generally do not travel on Italian roads (*Caliendo, 2012*).

Table n. 8 summarizes the correspondence assumed between the MEPDG defaults as truck classes and Italian truck classes.

Table 2. Correspondence Assumed between the MEPDG Defaults as Truck Classes and Italian Truck Classes

MEPDG defaults as truck classes			Italian truck classes	
Truck class	Type of vehicle	No. axles	Truck class	Type of vehicle
4	Buses	2 and 3	14 + 15 + 16	2-axle buses
5	Single-unit trucks	2	1 + 2 + 3 + 4	2-axle single-unit trucks
6	Single-unit trucks	3	5 + 6	3-axle single-unit trucks
7	Single-unit trucks	≥ 4	—	—
8	Single trailer trucks	≤ 4	—	—
9	Single trailer trucks	5	9 + 10 + 11 + 12 + 13	3-axle tractor with 2-axle trailer 2-axle tractor with 3-axle trailer
10	Single trailer trucks	≥ 6	—	—
11	Multitrailer trucks	≤ 5	7 + 8	4-axle multitrailer trucks
12	Multitrailer trucks	6	—	—
13	Multitrailer trucks	≥ 7	—	—

Table n. 8 – Correspondence Assumed between MEPDG and Italian truck classes

The measurement of the traffic is referred to single carriageway with two lanes and was carried out considering the number of passages of heavy vehicle.

Type CNR	**Type** MEPDG	**Vehicles**
3 o 4	5	246,991
5 o 6	6	70,431
7 o 8	11	35,216
9 o 12	9	70,913
16	4	24,319
		447,870

Table n. 9 – Number of heavy vehicles measured

For the design pavement structure, the catalogue "*Catalogo delle pavimentazioni stradali*" of C.N.R has been considered.

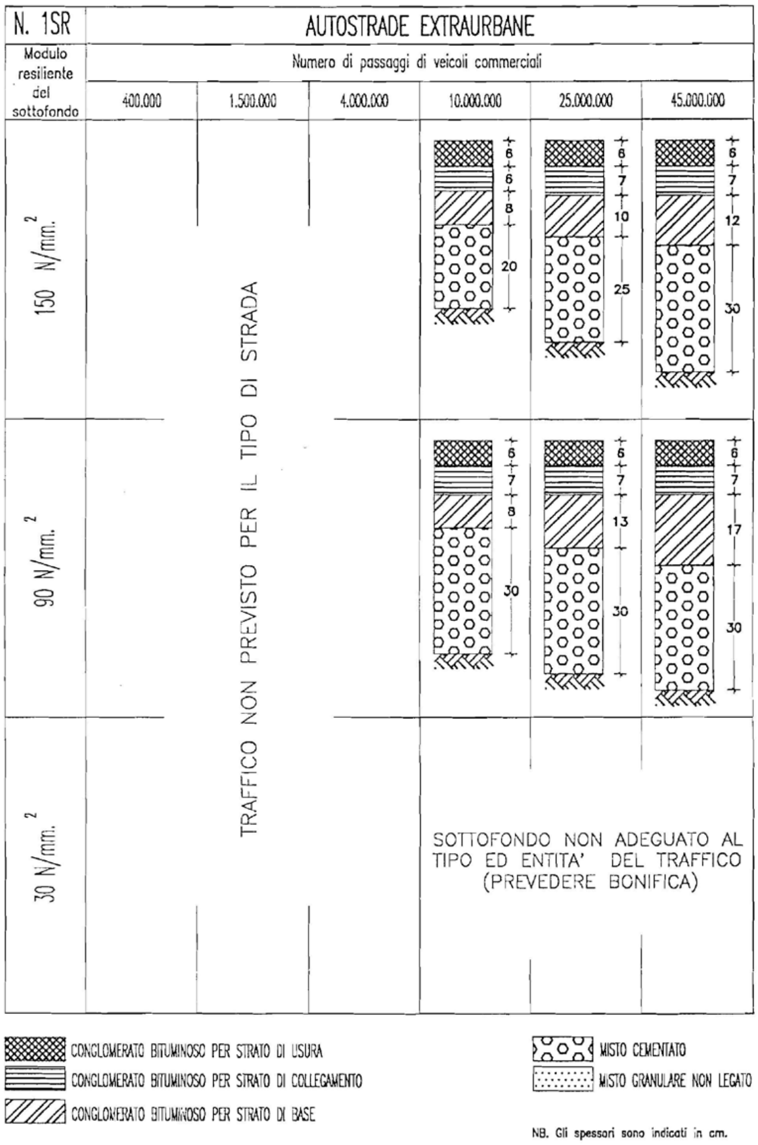

Figure n. 48 – Catalogo delle pavimentazioni stradali, C.N.R

In particular, considering the table "N. 1SR" there are different pavement proposal as a function of the resilient modulus and the amount of traffic. The hypothesized analysis period and the resilient modulus are respectively 20 years and 90 N/mm2. In this way, the following pavement structure is obtained:

- AC surface layer (2.36 in)
- AC intermediate layer (2.76 in)
- AC base layer (3.15 in)
- Sub-base (11.8 in)
- Subgrade

7.2.1 Design using PerRoad

The design pavement structure is evaluated in PerRoad using the three approaches:

- Strain distribution
- Single threshold
- Transfer functions

For the seasonal information, the mean monthly temperature has been considered.

To overcome the problem related to the calculation of the vertical compressive strain, two pavements have been considered. The first is composed of the unaltered design structure with 5 layers, instead, in the other one the intermediate and base layer have been aggregated in just one layer, and the total thickness remained the same.

The traffic condition can be summarised in the following *table n. 10*:

Vehicle Classification	**%AADTT**	**Average Number of Axles Per Vehicle**		
		Single	**Tandem**	**Tridem**
4	12,2	1,62	0,39	0
5	51,2	2	0	0
6	14,6	1,02	0,99	0
7	0	1	0,26	0,83
8	0	2,38	0,67	0
9	17,7	1,13	1,93	0
10	0	1,19	1,09	0,89
11	7.3	4,29	0,26	0,06
12	0	3,52	1,14	0,06
13	0	2,15	2,13	0,35
Total	100 %			

Table n. 10 – PerRoad traffic condition

First, the transfer function approach has been used, also to a have information about the damage cumulation and not only about the strain.

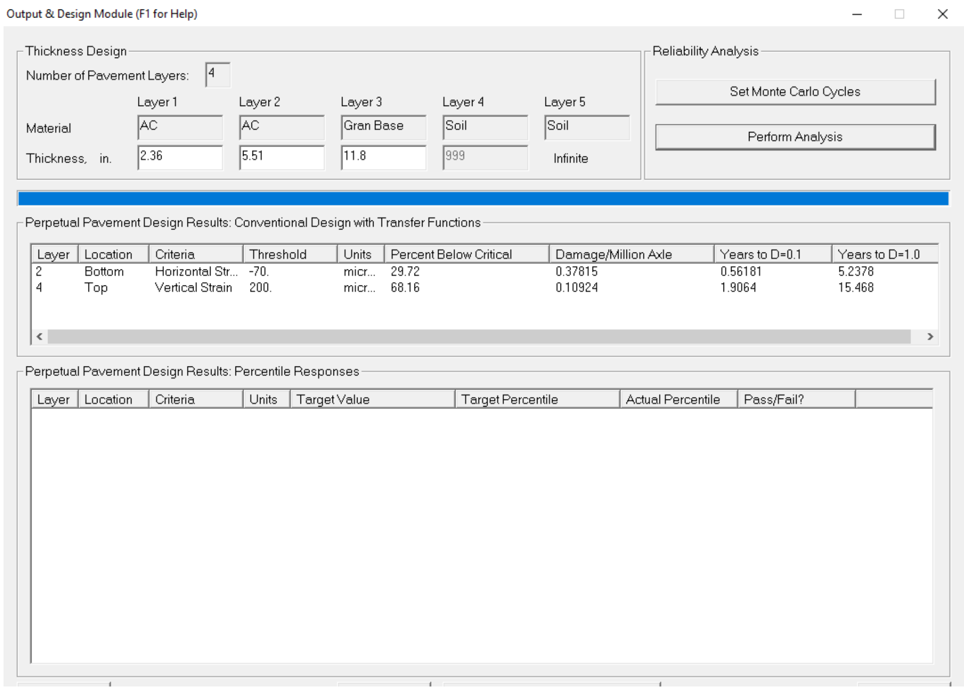

Figure n. 49 – PerRoad transfer function output of Italian case study

For the other two approaches, setting the fatigue endurance limit equal to 70, both the strain distribution and the single threshold output didn't pass the check. All the actual percentiles are below the target values (the figures regarding the last two approaches are reported in the attachment).

Re-design the pavement, changing the thicknesses of the layers in a way to have typical perpetual pavement thicknesses and changing some material proprieties such as described in the paragraph n. 4, the following results has been obtained:

	Initial design	**1st PP design structure**	**2nd PP design structure**	**3rd PP design structure**
Layered structure	AC (2.36 in) AC (2.76 in) AC (3.25 in) SB (11.8 in)	SMA (3 in) SFHMA (4 in) RBL (3 in) SB (11.8 in)	SMA (3 in) SFHMA (6 in) RBL (3 in) SB (11.8 in)	SMA (3 in) SFHMA (6 in) RBL (4 in) SB (11.8 in)
Threshold (micro strain)	70	70	70	70
Years to D=0,1	1,5	3,48	8,18	11,85
Years to D=1	20,42	24,88	44,52	55,73

% below the critical (70 microstrain with Target 50%)	45,04 %	59,96 %	71,38 %	77,84 %
Stain distribution check	Fail	Pass	Pass	Pass
Single Threshold check	Fail	Pass	Pass	Pass

Table n. 11 – Comparison of different pavement solution using PerRoad

Looking at the results, it is clear that the initial design doesn't satisfy the requirements of a Perpetual Pavement.

For the last two layers the high-temperature grade of the asphalt remained the same as the surface, but the low temperature requirement has been relaxed one grade, since the temperature gradient in the pavement is relatively steep and the low temperature in these layers would not be as severe as the surface layer.

In this way, using thicknesses that are inside of the typical range of thickness of a PP, in the different solutions proposed, there is an evident structural improvement on the pavement and an increase of the pavement life.

Considering a comparison between the bituminous layers of the initial design and the 2nd Perpetual Pavement solution, the following results are obtained:

Initial design	**2nd PP design structure**	**Increment/Decrement**	
		(in)	**(cm)**
AC (2.36 in)	SMA (3 in)	+ 0.64	+ 1.63
AC (2.76 in)	SFHMA (6 in)	+3.24	+8.22
AC (3.25 in)	RBL (3 in)	-0.25	-0.64
		+3.63	**+9.18**

Table n. 12 – Comparison between the bituminous layers of the initial design and the 2nd Perpetual Pavement solution

As highlighted from the *table n. 12,* in order to transform the classical flexible asphalt pavement in a Perpetual pavement, in which the pavement's life is close to 50 years, it requires a total increment of the bituminous layers of 9,18 cm.

Conclusions

While one might think pavements designed to last longer would incur more or have higher initial costs than pavements with shorter life-cycles, it has been shown that Perpetual Pavements have the following benefits *(Timm and Newcomb, 2006):*

- They provide a more efficient design, eliminating costly overly conservative pavement sections.
- They eliminate reconstruction costs by not exceeding a pavement's structural capacity.
- They lower rehabilitation-induced user delay costs.
- They reduce use of non-renewable resources like aggregates and asphalt.
- They diminish energy costs while the pavement is in service.
- They reduce the life-cycle costs of the pavement network.

In order to provide the above advantages, it is necessary to know what thickness of pavement section will support the heaviest anticipated traffic loads without grossly over-designing the pavement. Researches have shown that this can be identified mechanistically by identifying the stresses, strains, or displacements in a structure which are low enough to avoid the initiation of cracking or rutting deep in the pavement structure.

Of course, also the material proprieties have a fundamental role, where each layer is characterized by specific materials suitable for carrying out its functions.

The HMA base layer has the main function to resist to the repeated loading condition and to avoid the formation of cracks which can be propagating, giving the possibility to the water to go inside, reducing the structural function of the pavement, and this can be avoided using a mixture with an higher designed asphalt content, also called Rich Bottom Layer (RBL), which accomplishes two important goals: from one side it allows the material to be compacted to a higher density, and in turn, improve its durability and fatigue resistance.

The wearing surface requirements would depend on traffic conditions, environment, local experience, and economics. Performance requirements include resistance to rutting and surface cracking, good friction, mitigation of splash and spray, and minimization of tire-pavement noise. These considerations could lead to the selection of stone matrix asphalt (SMA), an appropriate Superpave dense-graded mixture, or open-graded friction course.

The intermediate or binder layer must combine the qualities of stability and durability. Stability in this layer can be obtained by achieving stone-on-stone contact in the coarse aggregate and using a binder with an appropriate high-temperature grading. The internal friction provided by the aggregate can be obtained by using crushed stone or gravel and ensuring an aggregate skeleton. One option would be to use a large nominal maximum size aggregate which could reduce cost due to a lower asphalt content (*Newcomb et. al.*). Both binder and aggregate are of importance for resisting shearing failure and formation of ruts. It is typically the thickest layer in the system expose to both tension and compression by situating on both sides of the neutral axis.

Rutting can be prevented by using an appropriate high temperature grade binder. The high-temperature grade of the asphalt should be the same as the surface to resist rutting. However, the low temperature requirement could probably be relaxed one grade, since the temperature gradient in the pavement is relatively steep and the low temperature in this layer would not be as severe as the surface layer (*Newcomb et. al*., *2006*).

Looking at the results obtained thanks to the software and the case studies, it was possible to demonstrate that using the appropriate materials, rearranging the thicknesses of each layer, in particular increasing the intermediate layer with respect the other two, and considering an acceptable increase of the total thickness of the bituminous layers, there is the possibility to obtain a huge structural improvement.

In conclusion, the Perpetual Asphalt Pavement design approach is relatively recent concept, but it allows to design and built an asphalt pavement that lasts for more than 50 years without requiring major structural rehabilitation or reconstruction, with just a periodic surface renewal. Clearly, using this type of approach the possible risk is to overdesign the pavement, because there are limits after which the increase of the thickness doesn't give any structural improvements, in this way, the concept of perpetual pavements was developed with the idea to keep the advantages coming from the long-life pavement, and, at the same time, improving the methodology, avoiding possible economic risks.

Attachments

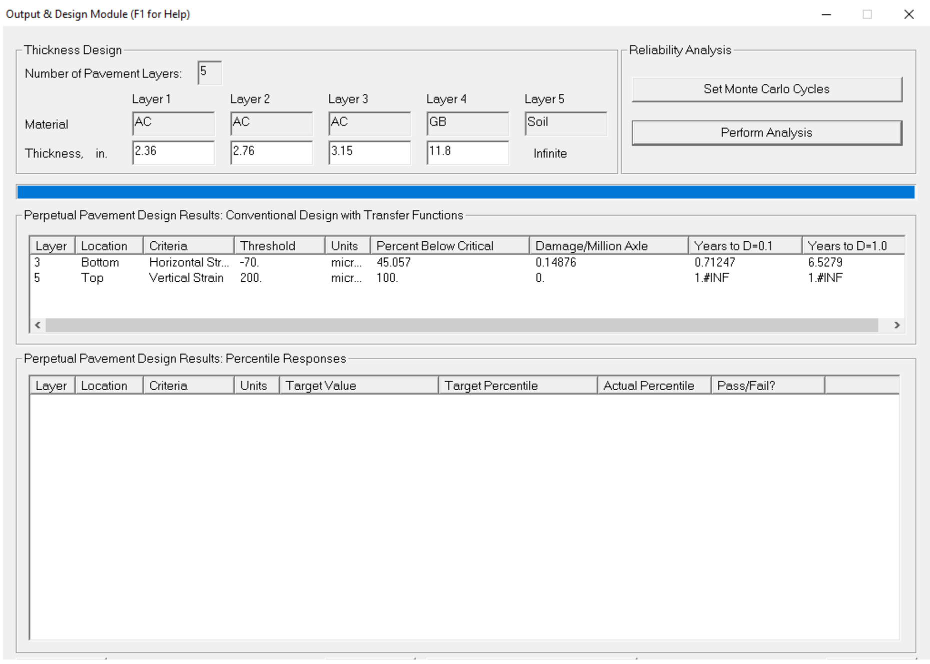

Figure n. 51 – Five-layer PerRoads tranfer function output of Italian case study

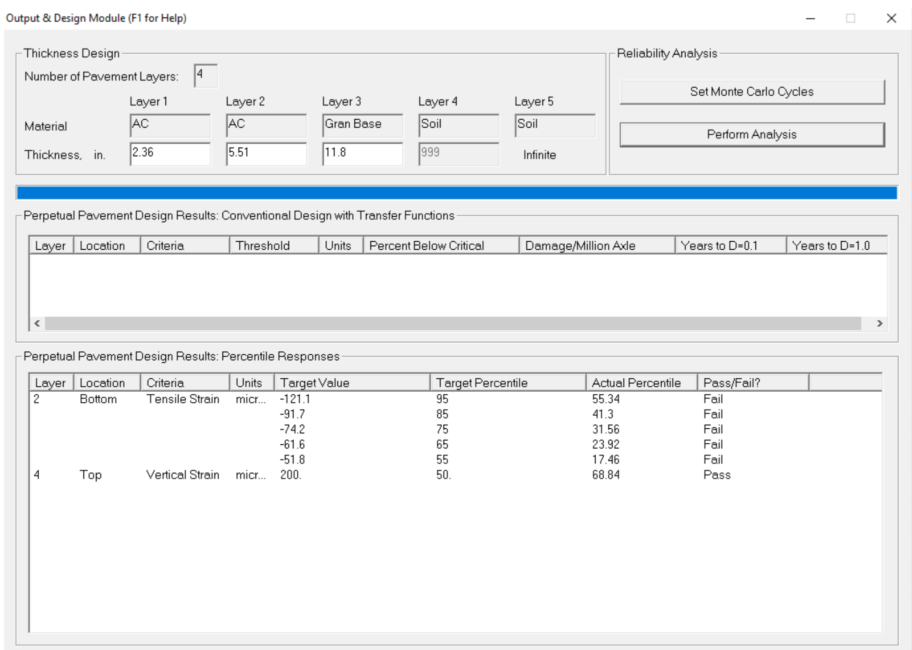

Figure n. 52 – Five-layer PerRoads strain distribution output of Italian case study

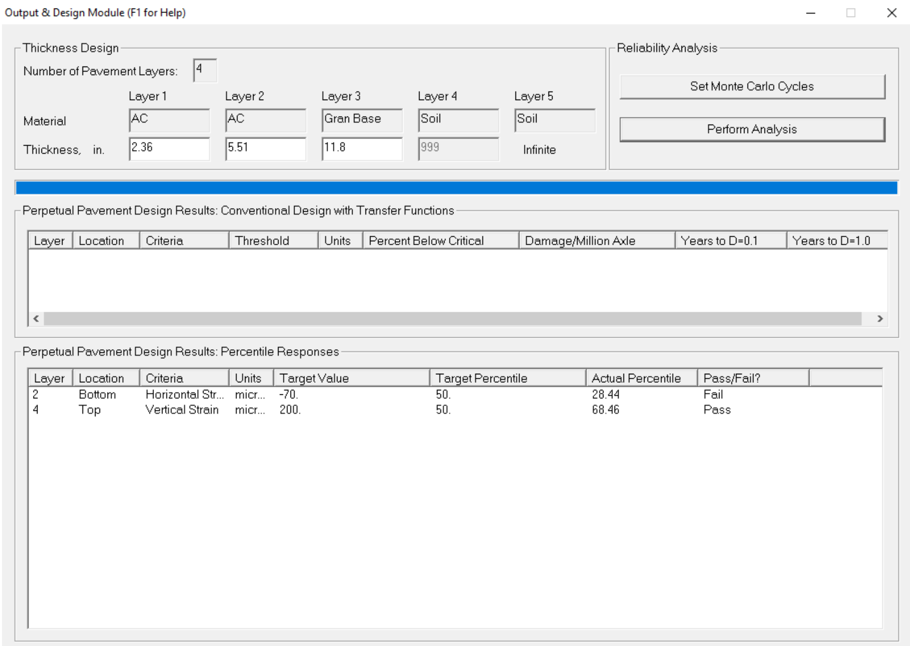

Figure n. 53 – Five-layer PerRoads single threshold output of Italian case study

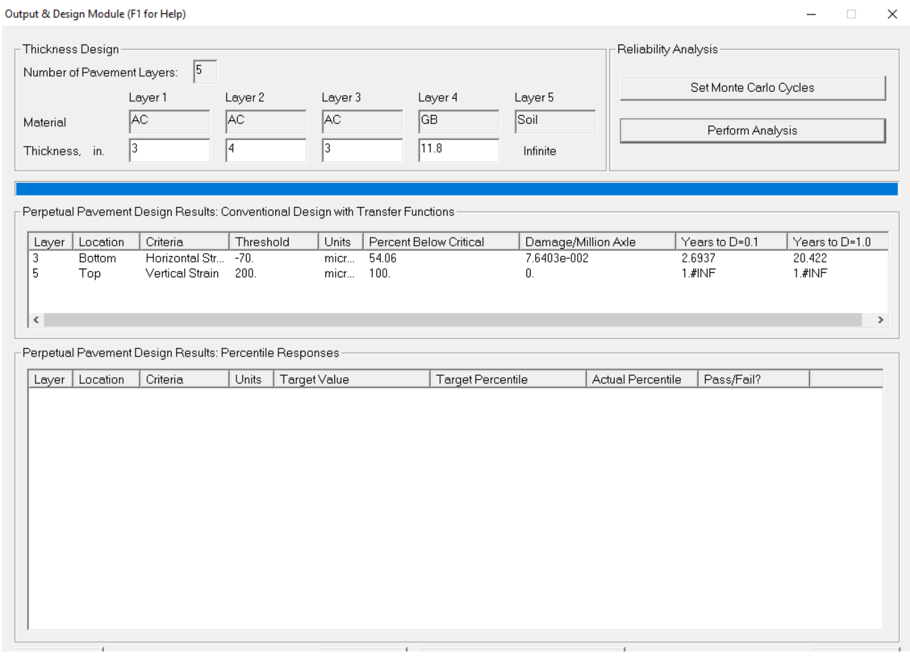

Figure n. 54 – 1st Perpetual Pavement solution of Italian case study, PerRoads transfer function output

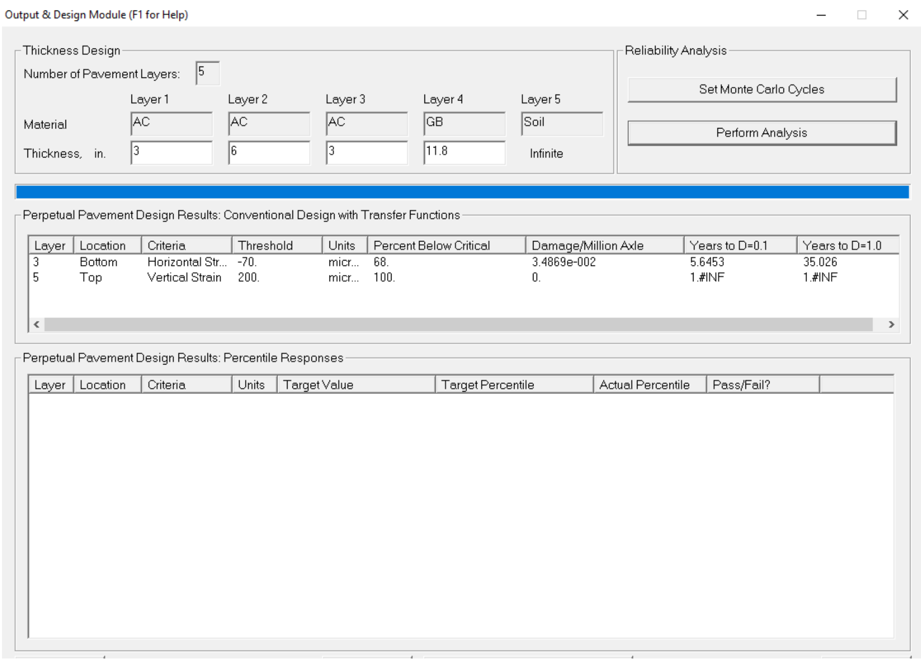

Figure n. 55 – 2nd Perpetual Pavement solution of Italian case study, PerRoads transfer function output

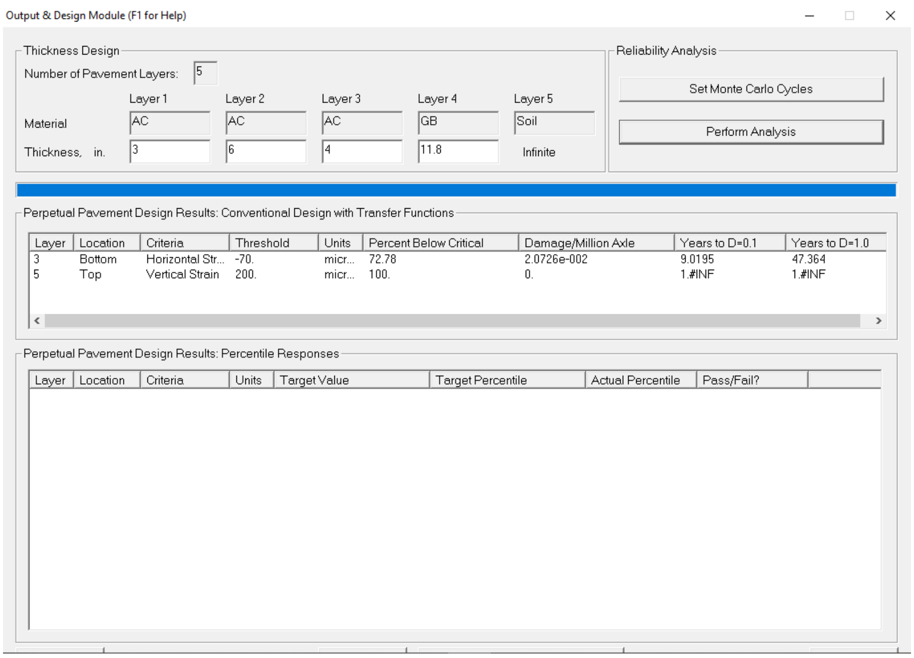

Figure n. 56 – 3rd Perpetual Pavement solution of Italian case study, PerRoads transfer function output

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