CEDR Transnational Road Research Programme
Call 2012: Recycling: Road construction in a post-fossil fuel society

funded by
Denmark, Finland, Germany,
Ireland, Netherlands, Norway

Report on incorporation of cold-recycled pavement layers in empirical and mechanistic pavement design procedures

Deliverable D3.1
30.12. 2014

Coordinator: Czech Technical University in Prague (CTU)
Partner 1: University of Kassel (UK)
Partner 2: University College Dublin (UCD)
Partner 3: Laboratório Nacional de Engenharia Civil, I.P. (LNEC)
Partner 4: Wirtgen GmbH

CEDR Call2012: Recycling: Road construction in a post-fossil fuel society

CoRePaSol
Characterization of Advanced Cold-Recycled Bitumen Stabilized Pavement Solutions

Report on incorporation of cold-recycled pavement layers in empirical and mechanistic pavement design procedures

Deliverable D3.1

Due date of deliverable: 30.09.2014
Actual submission date: 31.12.2014

Start date of project: 01.01.2013 End date of project: 31.12.2014

Author(s) of this deliverable:
Jan Valentin, CTU, Czech Republic
Petr Mondschein, CTU, Czech Republic
Jiří Fiedler, independent expert, Czech Republic
Konrad Mollenhauer, University of Kassel, Germany
Fátima Batista, LNEC, Portugal
Ana Cristina Freire, LNEC, Portugal

Table of contents

Executive summary ... 4
1 Introduction ... 6
2 Literature review ... 8
2.1 Information from accelerated pavement test and trial sections ... 8
2.1.1 Accelerated pavement test and trial sections in New Zealand, on pavements comprising cold recycled mixtures using foamed bitumen and/or cement as binder ... 8
2.1.2 Trial sections in Australia, on pavements comprising cold recycled mixtures using mainly foamed bitumen as binder ... 12
2.2 Notes on pavement design methods for cold recycling ... 16
2.2.1 Empirical design methods ... 16
2.2.2 Analytical design methods ... 19
2.3 Design catalogues ... 23
2.3.1 Pavement parameters ... 24
2.3.2 Model pavement design ... 24
3 Conclusions from the literature review ... 26
4 Notes on the application of fatigue tests of asphalt mixes in the analytical design for flexible pavements ... 27
5 Proposal for the analytical design methods for cold recycled mixes ... 33
5.1 Design criteria for fatigue of cold recycled layer ... 34
5.2 Design criteria for permanent deformation of cold recycled layer ... 36
6 Acknowledgement ... 37
7 References ... 38

Executive summary

Inclusion of cold recycled mixes in pavement design methods can be found in the design manuals of different countries only in a very limited scope. Often pavement structures including layers with cold recycled materials are shown in some predefined structures for expected traffic loadings, life-time and/or deterioration progress with already set thickness and eventually bearing capacity for the cold recycled layer. This can be found e.g. in Germany or France; same approach is closely described also in the Wirtgen Cold Recycling Manual. Simple empirical computing design methods exist in the United States and are also used in the UK or Ireland. Analytical computing design methods are common in Australia, New Zealand or South Africa. Alternative approaches can be found in case of the last mentioned country, where the assessment of cold recycled layers are today usually based on results from triaxial testing and an interesting approach of so called well-balanced pavement structure is promoted. Then there are countries where the cold recycled materials are not included in the pavement design manuals and the only application of cold recycling is during rehabilitation works, mainly done in situ. In these cases it is usually expected that the bearing capacity might be similar to other stabilized base layers. Review of the existing knowledge is given in the first part of this report.

In the second part of the report attention is paid to the parameters which are usually required to be included in a pavement multilayer structure design. Besides stiffness, permanent deformation or fatigue life is the key aspects which are well known from asphalt pavement design methods used worldwide. Stiffness might be the simplest characteristic, which can be determined for this type of material and is in detail described in the Project Report D2.1 on stiffness. Determination of resistance to permanent deformation based on test methods like wheel tracking test might not be applicable to cold recycled mixes. This might be true especially if bituminous emulsion is used as one of the binders. The reason is a long consolidation period which is required for the emulsion and the experience or findings might be very similar with experience known for cold asphalt mixes. Therefore there is only on reasonable alternative in case permanent deformation characteristics are studied and this is triaxial testing. Nevertheless, this type of testing is in case of cold recycled mixes not common for European countries and it might be limited only to mixes with low or none content of hydraulic binders. Last but not least fatigue life is usually one of the key characteristics to be known if flexible (asphalt) pavement structures are designed. From the results and findings gained during the CoRePaSol project there are several limitations for fatigue testing. Firstly the test might not be suitable for cold recycled mixes with lower content of bituminous binder (less than 2.5 % of residual bitumen) and hydraulic binder (less than 3.0 %). Secondly it was repeatedly proven that the only viable test method is indirect tensile fatigue test, mainly and foremost because of test specimen preparation. Any other tests used more often for fatigue determination (2-point test or 4-point test) are practically not applicable. Therefore it cannot be recommended to assess fatigue for each cold recycled mix and it would not be of preference to recommend fatigue testing as a standard requirement if cold recycled mixes are used – mainly for new payment structures. If effective analytical pavement design is requested by a road administration, it is recommended to follow principles defined in chapter 5. It should be also critically analyzed – country by country –

how well the predefined pavement structures shown e.g. in the Wirtgen Cold Recycling Manual are applicable or how they can be modified respecting the national criteria and design conditions.

1 Introduction

Cold recycling technologies include various materials and binders as presented earlier (bitumen emulsion, foamed bitumen, hydraulic binders). Different binder combinations and dosing are used as well as different ratios between Reclaimed Asphalt Pavement (RAP) and natural aggregates. That is why those materials need the development of special dedicated studies, namely regarding its behaviour under repeated loading and changing climatic conditions. Furthermore, after the manufacture and placement of cold recycled mixtures, the water (present on the bituminous emulsion, on the foamed bitumen and/or added as raw material) starts being eliminated, mainly through the compression induced by the rolling compactors and later by evaporation. During this process, bitumen particles of the bituminous binder start to establish “bridges” among each other and with the aggregates/RAP particles, acting as a bituminous binder that holds the granular particles in place, being desirable that a strong adhesive bond is achieved at the end of this process [1]. Therefore, cold stabilised mixture will only present their “final” characteristics when its curing is concluded, which can take several months. Nevertheless, there is an intermediate phase, before the curing is completed, when the pavement can already be trafficked, being necessary to ensure that no pavement damages occur that could compromise pavement performance, either in the short or in the long term [2].

Cold recycled mixes with lower binder content behave is some extend more as unbound granular materials, mainly for early ages of curing. Thus cumulated permanent deformations are of major importance. The mixes with higher bitumen content might in some cases present fatigue behaviour similar to hot bitumen mixes, mainly for advanced curing. However the aggregates are usually not completely coated with the new binder. On the other hand, binder included in RAP might play an active role since activity of the binder is expected and was proven within CoRePaSol project as well. Instead of complete aggregate coating local bridges are formed, which can be destroyed due to repeated loading even if typical fatigue cracks do not appear on the surface. This is especially true for foamed bitumen. Better coating of aggregates is achieved with bitumen emulsions. Higher binder content (either in form of added binder or in RAP) usually improves the coating and fatigue resistance of those materials, but often decreases the mix stiffness. With increased content of hydraulic binder, the material properties narrows that of purely hydraulic bound pavement materials with typically brittle properties and early-life cracking due to shrinkage and temperature loads.

Due to the various recycled mixes and the change of their behaviour it is very difficult to establish a unified analytical pavement design that takes into account all these variables.

It is well known that the pavement design for new roads has to cope with many uncertainties. Pavement design for rehabilitation is even more complex due to the variability of materials mentioned above and due to the fact that the designer has normally only partial information about the state of the existing pavement.

Simple empirical design methods have been used in the past. However the analytical methods are increasingly used for the rehabilitated pavement with heavy traffic. There are differences in the design methods in Europe and in the world. They are usually related to local experience. Thus, the new methods developed abroad are not easy to implement and a harmonized approach, although advantageous, is usually rather difficult to be used in different countries.

The proposition of a unified European design method doesn't seem to be possible at present. However, a common approach can be established that might be accepted by various national administration and agencies. It can be used by them for the development of appropriate local methods. The presentation of this unified design approach is attempted here.

The overview of design methods and the evaluation of their mutual advantages and inconveniences are presented in the latest version of Wirtgen Cold Recycling Manual (WCR Manual) [3] which includes also a list of references. However there are some important new papers on the subject which have been published since 2012. Overview of this latest research as well as of results given in some interesting older research papers not quoted in WCR Manual [3] is presented in chapter 2.

The conclusions from the literature review are presented in chapter 3. Notes on the problems related to the application of fatigue tests of asphalt mixes in the analytical pavement design for flexible pavements are in chapter 4. The formulation of the principles of the design approach is presented in the chapter 5.

2 Literature review

Literature overview is focused on the information from the accelerated pavement tests which permit to estimate the real behaviour of pavements with recycled layers, especially if the various sensors are used for the stresses, strains or deflections measurements. However some recent studies on laboratory properties of recycled mixes are mentioned here at first which illustrate the differences in behaviour of recycled mixes.

The increase of stiffness for mixes with foamed bitumen content between 2.0 % and 3.5 % as well as cement content of 1.0 % to 2.5 % and the impact on indirect tensile strength and ITSR is shown in the report of Iwański and Chomicz-Kowalska (2013) [4]. According to Jian Xu *et al*. (2011) [5] optimal cement content for mixes with emulsion is around 1.5 %.

Literature review related to the pavement design is divided into two parts. At first information from accelerated pavement test and trial section is presented which illustrates the behaviour of cold recycled mixes. Then brief information about various empiric and analytical methods is given.

2.1 Information from accelerated pavement test and trial sections

2.1.1 Accelerated pavement test and trial sections in New Zealand, on pavements comprising cold recycled mixtures using foamed bitumen and/or cement as binder

Lot of research on cold recycling, mainly produced by adding foamed bitumen as binder, has been carried out in recent years in New Zealand. Some experiments were realised in the Canterbury Accelerated Pavement Testing Indoor Facility (CAPTIF) in Christchurch. Advantage of this facility is that the experiments are realised in controlled climatic conditions. Comprehensive report on these experiments was published in 2013, [6]. Recycled materials with different dosing of foamed bitumen and cement were tested in CAPTIF. The load on the single tyre was 40, 50 and 60 kN. Deflection tests and strain measurements were carried out. Trench profiles and photos illustrating the damage are presented in the report. Material properties were evaluated by the repeated load triaxial testing, ITS, UCS and fatigue tests. Pavement analysis based on these results permitted to establish some recommendations for the design of rehabilitated pavements in New Zealand. Analysis of the behaviour on some job sites was also undertaken. The comparison with AUSTROADS and South African pavement design method (that is described in WCR Manual) [3] is also presented in the report which has nearly 200 pages. Short description of tested materials and some interesting results are briefly mentioned here.

The composition of the pavements tested in CAPTIF was similar in all sections. There was a thin AC wearing course placed on the base course from recycled material. The base course thickness was 200 mm. The subgrade soil was compound of clay (average in situ CBR value estimated from penetrometer tests was in individual sections from 7 % to 9 %). FWD tests performed on the surface of compacted subgrade gave a modulus of about 60 MPa.

The first experiment included 6 sections. Four of them had the recycled mix with 1 % of cement. Foamed bitumen content was 0; 1.2; 1.4; 2.8 %. Reference section had unbound layer instead of a recycled layer. The last section had 2.7 % of foamed bitumen and no cement added. The second experiment included also 6 sections. Mixes only with hydraulic binders were tested. Dosing of cement was 0; 1; 2; 4 %. Lime was used in one section. The last section was a reference section made from unbound materials.

The results of the second experiment on mixes with cement only are mentioned here very briefly as the behaviour of these recycled mixes is better known and was described also in older documents. The significant decrease of the resilient modules established by FWD tests and the increase of measured strain in the lower part of the base course and subsoil during the experiment corresponds to the concept of two phase behaviour - the fatigue life phase where the initial modulus is high, but then the modulus drops rapidly. The second phase is characterised as an equivalent granular phase where the modulus of the recycled layer remains relatively constant. The recycled layer with 4 % of cement had the highest stiffness, but it decreased significantly (according to FWD test results) tending to a stiffness value for mix with 1 % of cement.

Interestingly no cracks were observed on the surface of the pavement. It is visible from photos included in the report. This fact was also mentioned on the page 64 of the report [5] devoted to the development of tensile fatigue criteria in NZ. Sawn cut beams from the pavement after the experiment had low strength and flexural beam modules. Some specimens were impossible to test in the laboratory. This suggests that the test sections were highly damaged during the experiment. Although has to be kept in mind that pavement configuration in CAPTIF resulted in very high tensile stresses and strains. (if the modulus of 1000 MPa was assumed the tensile stress 1.0 MPa and strain 694 microstrains were calculated by linear multilayer elastic analysis. Values for assumed modulus 4000 MPa were 1.4 MPa and 234 microstrains). That is why the bound behaviour was short and granular phase was important.

The loading and observed behaviour of mixes with foamed bitumen and cement in the first experiment was more complex. The loading of 40 kN was used for 150 000 load cycles. Due to the little rutting the load was increased to 50 kN till 500 000 load cycles. Then the load was increased to 60 kN and the wheel speed lowered from 40 km per hour to 30 km per hour.

The thickness of the AC layer was only 20 mm at the beginning of the experiment. This had led to degradations in this layer and that is also the reason why after 200 000 load cycles the pavement was overlaid by another 30 mm of AC layer. Thus the thickness for the main period of the experiment was 50 mm. After 1.35 million of cycles water was permitted to flow into the pavement. This led to the increase of rutting in all sections. The wet testing at the end of the experiment indicated that 1.4 % and 2.8 % foamed bitumen contents considerably reduced the moisture susceptibility of the stabilised materials.

FWD tests showed that none of the test sections appeared to lose stiffness during the experiment in CAPTIF. This confirms the observation of other researchers that the behaviour of foamed bitumen during repeated loading does not correspond to fatigue behaviour of hot mixes with the continual decrease of stiffness and apparition of fatigue cracks. That is why the pavement modelling in the report of Alabaster *et al*. (2013) [6] and evaluation of the improvements of the durability (increase of load cycles to failure) due to the recycled layer was based only on the extrapolation of the measured surface deformations.

The development of rutting is presented on figure 1 (figure 2.5 in the original paper). Mean rut depths values are presented there.

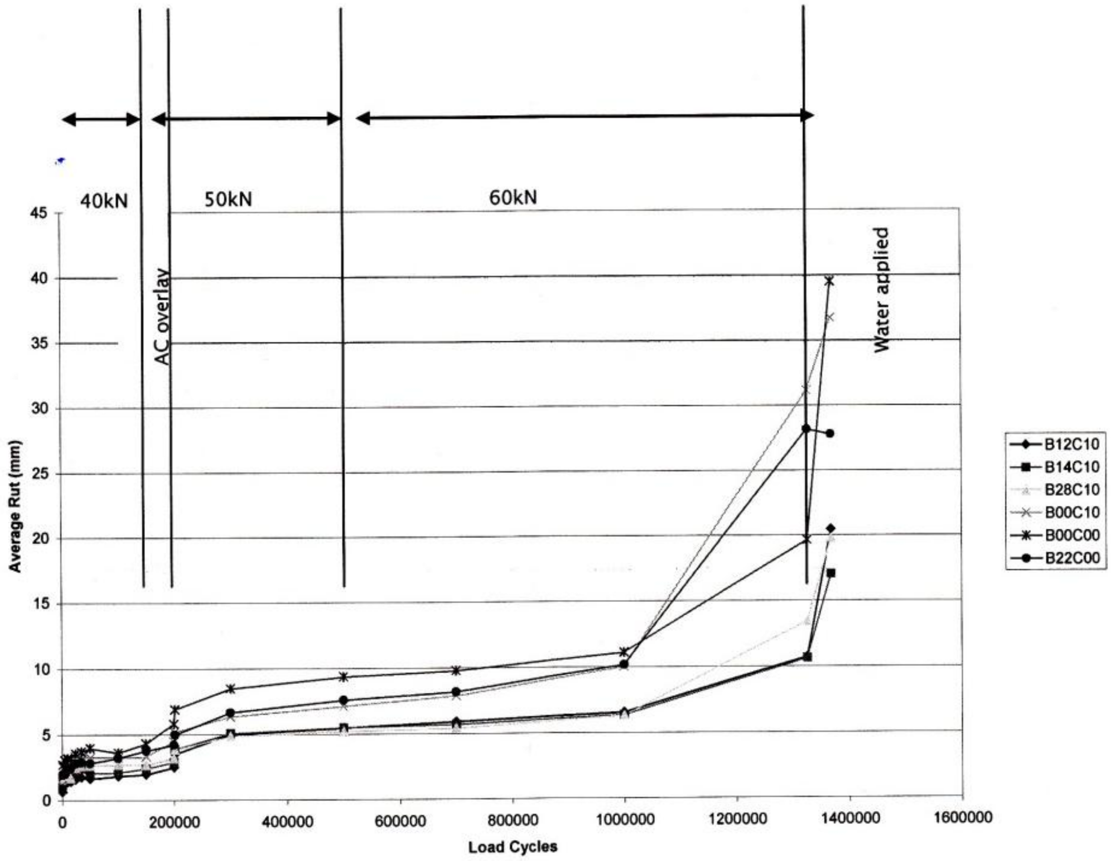


Figure 1: Rutting measured in CAPTIF experiment for foamed bitumen stabilized mixes, [4]

The report includes also a figure with the statistical evaluation of 90 % level of confidence (for example the rut depth after 1 million of cycles in the section with 2.2 % of foamed bitumen was 10 mm for the probability of 50 %, but 15 mm for the 90 % level of confidence. The rut depth for this confidence was calculated as mean value plus 1.28 times standard deviation). The rutting of mixes with foamed bitumen and 1 % of cement was distinctly lower than for mix with 1 % of cement without foamed bitumen. This corresponded to higher laboratory resilient modulus (about 400 MPa to 500 MPa for mixes with 1 % of cement and different contents of foamed bitumen and about 200 MPa for mix with 1 % cement and no foamed bitumen). Laboratory tests confirmed observation of other researchers (for example [8] and [5]) that there is an optimum content of foamed bitumen.

The rut depth of the mix with 1 % cement only started to increase rapidly after 1 million of cycles.

Results of the modelling are presented in the table 1 for 2 different calculation models.

Regarding the results of modelling it has to be kept in mind that the 20 mm rut on 10 % of the surface was considered. Usually lower rut depths are allowable in Europe. Thus the number of the cycles to failure for every section presented in the report is only indicative. However the percent improvement gives a good idea about the possibilities of tested technologies.
Main conclusions of the report were as follows.

- The addition of 1 % of cement to foamed bitumen mix was recommended as properties as well as the durability of the pavement are distinctly improved.
- The pavement life was longer than calculated by currently used methods.
- Material behaves like a stabilized (bound) when3-4 % of cement is present. At 3 % cement content fatigue failures were observed in some studied job sites.
- Prudent limit for design to start considering bound behaviour would be at 2 % cement content. Above this limit there is a risk of cracking leading to risk of water penetration into pavement structure, and potentially rapid failure and difficult repairs.
- The Austroads tensile strain criterion appeared to produce inappropriate results for New Zealand conditions. The South African approach appeared to produce more appropriate results and should be further investigated.

Table 1: Load repetitions to terminal conditions, (table 2.10. in original paper) [6]

Section	Material	Basic model		Alternative model	
		60kN load cycles	Percent improvement	60kN load cycles	Percent improvement
A	1.2% foamed bitumen, 1% cement	2.6E+06	246%	4.4E+06	333%
B	1.4% foamed bitumen, 1% cement	2.8E+06	264%	6.06E+06	454%
C	2.8% foamed bitumen, 1% cement	2.4E+06	230%	6.8E+06	519%
D	1% cement	1.2E+06	114%	2.9E+06	223%
E	Unbound (no binder)	1.1E+06	100%	1.3E+06	100%
F	2.2% foamed bitumen	1.1E+06	106%	1.9E+06	142%

Some supplementary information on these experiments is given in [8]. Photos illustrating that cracking appeared in the section with 1.2 % of foamed bitumen and 1 % of cement at the end of the experiment, but no cracking was observed in sections with 1.4 % and 2.8 % of foamed bitumen and 1 % of cement.

The results of this very extensive research cannot be directly transposed to European conditions, especially the aspects concerning pavement design. However they help to understand the real behaviour of different recycled materials in roads loaded with low traffic, where the recycled layer is overlaid by one thin layer only and the stresses due to the traffic load are very high.

2.1.2 Trial sections in Australia, on pavements comprising cold recycled mixtures using mainly foamed bitumen as binder

Information on pavement design, construction and field performance of 9 trial sections in Australia was published in 2013 [10]. Four short sections were under-designed to assist in better understanding of foamed bitumen stabilised pavement performance (for example 100 mm, 150 mm thick foamed bitumen layers were used for under-designed sections of the length of 75 m and 200 mm for the rest of the job site). No conclusion concerning the pavement design has been obtained from these trials yet. However the project will continue for another 2 years. New broad field experiment has been started in 2013, [11].

Foamed bitumen stabilised pavements have been widely used in South Africa. Various research reports on laboratory testing, field observations and pavement design were published. The results of recent experimental section which was constructed in 2012 are described in [12]. There were 25 sub-sections, each approximately 350m in length with different cement and residual bitumen content. No evidence of stiffness reduction within the first year, due to the damaging effects of traffic loading was detected even if the significant fluctuation of FWD test results was observed.

Results of the 10 years research on cold in situ recycling were presented on a TRB workshop in 2013 [8]. This research included laboratory tests and the monitoring on the couple of job sites. It demonstrated also the impact of the rain on the stiffness of recycled mixes during the first days after placement. Partial decrease of the stiffness of the mix (due to the increase of the water content) was observed after heavy rain followed by the new stiffness increase. On a Delaware county job site the decrease of the stiffness was observed even 11 days after placement.

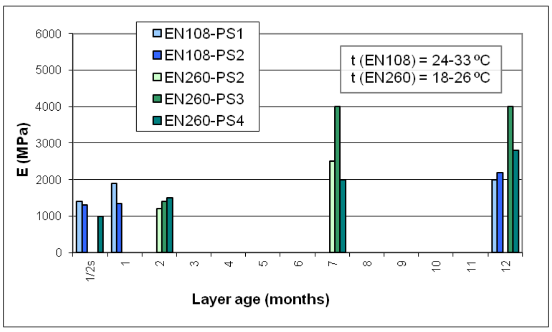
2.1.3 Follow-up of road sections in Portugal, on pavements comprising cold recycled mixtures using mainly bituminous emulsion as binder

Curing of asphalt cold mixtures has a great influence on the evolution of the mixture properties and therefore on the performance of the entire pavement. A research study developed between 1998 and 2004, in the frame of a PhD thesis [14], should be considered, in which pavement rehabilitation works of Portuguese National Roads sections where cold mixtures were used either through the application of a new overlay or through in situ cold recycling of the existing pavement were followed-up. Monitoring of the cold layers properties through the curing process and the structural assessment of the pavement after rehabilitation were among the main activities that were undertook. Besides some parallel laboratory studies were also performed with view to correlate their results with in situ performance.

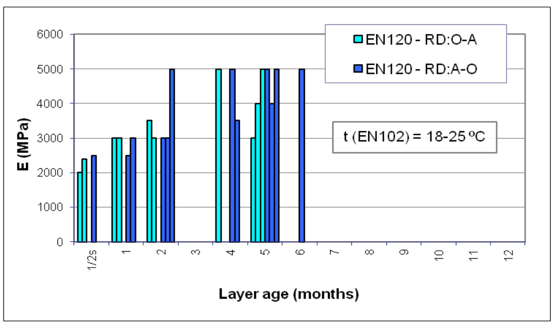
Four of the addressed pavement rehabilitation works in which cold mixtures were used either through the application of a new overlay or through in situ cold recycling of the existing pavement, were the following:

- Road stretch of the National Road EN 108 in the North of Portugal, where the pavement rehabilitation works carried out in 1997, comprised *in situ* recycling of existing bituminous layers in about 15 cm depth (using 3.0-3.5 % of a cationic slow-setting bituminous emulsion and 1.5 % of cement), and the placement of a binder course (6 cm thick) and a wearing course (6 cm thick).
- Road stretch of the National Road EN 260 in southern Portugal, where the pavement rehabilitation works carried out in 1998, comprised, in situ cold recycling of existing asphalt and granular layers up to 12 cm deep (using 5 % of a cationic slow-setting bituminous emulsion, 10 % of 0/5 aggregates for grading correction and 1-2 % of lime for reduction of the plasticity of fines in the RAP material), and the application of a slurry surfacing, which acted as a wearing course for a few weeks. Later, an asphalt cold mix binder course (6 to 10 cm thick) and a new slurry surfacing were applied.
- Road stretch of the National Road EN 120 in the South of Portugal, where a dense-graded asphalt cold mixture (using □ 6.5 % of a slow-setting cationic bituminous emulsion and □ 2.5 % of added water) was applied as overlay, with a thickness of 10 cm, followed the application of a slurry surfacing [2].
- Road stretch of the National Road IP2 in southern/central Portugal, where the pavement rehabilitation works carried out in 2002, comprised *in situ* recycling of existing bituminous layers in about 15 cm depth (using 3 % of a cationic slow-setting bituminous emulsion and 3 % of added water), and the placement of a base course (7 cm thick) and a thin open graded wearing course (3 cm thick).

In order to achieve results concerning the structural behaviour of the cold asphalt layers after the rehabilitation works, tests with the Falling Weight Deflectometer (FWD) were performed in EN 108, EN 260 and EN 120, whose cold layer modulus (E) of each section was back-analysed as shown in Figure 2.



(a)



(b)

Figure 2: E modules obtained from FWD back-analysis on: (a) in situ cold recycled layers; (b) "new" cold stabilized layers (adapted from Batista, 2004) [13])

Generally, it can be concluded that the modulus strongly increases from early ages (less than one month) to older than two months after construction. For the testing temperature, the "final" modulus of the cold bituminous mixtures is above 2000 MPa for the recycled pavement layers of EN 108 and EN 260, and above 3000 MPa for the new mixture applied at the EN 120.

Furthermore, experimental tests were performed in order to determine performance related properties both of specimens extracted from the field compacted layers and of specimens produced in laboratory and submitted to different curing procedures:

- The stiffness modulus and fatigue behaviour, by indirect tensile tests (Figure 4);
- Permanent deformation resistance, both by repeated load uniaxial compression tests (Figure 5) and wheel tracking tests.

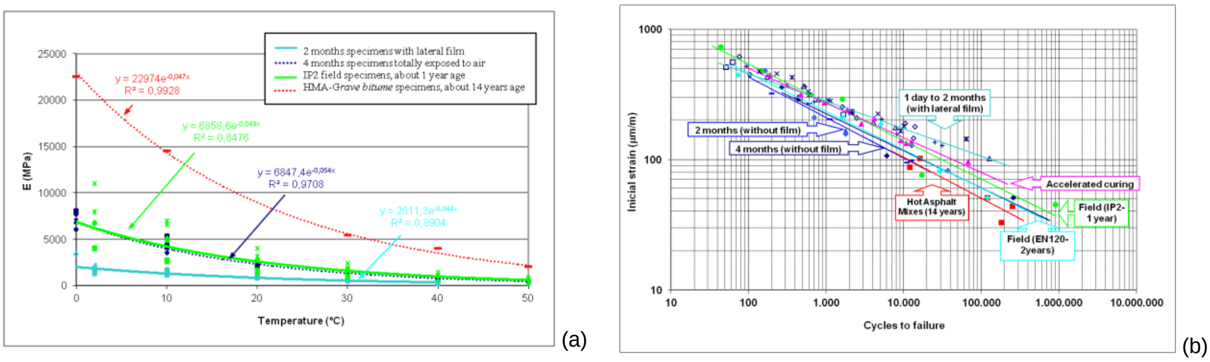


Figure 3: (a) Variation of the asphalt cold mix specimens' stiffness modulus (E) with the temperature; (b) Fatigue life of asphalt cold mix specimens with varying curing and of hot mix specimens (adapted from Batista, 2004) [13])

The results obtained in this study [2] allowed for the following conclusions:

- There is a considerable influence of the temperature on the modulus (determined by ITT) of all the mixtures tested, which can be represented by a relationship between the stiffness modulus at a given temperature (T) and the stiffness modulus at a reference temperature of 20 °C, as follows:

$$E_{ITT}\left(T\right) = 2{,}72.E_{ITT}^{20^{\circ}C}.e^{-0{,}05T} \qquad (1)$$

- Fatigue properties of the mix were similar for all specimens tested at relatively early ages (up to 2 months). For these mixtures, the slope of the fatigue life is lower than that generally obtained for hot mix asphalt (HMA). When the curing process of the cold mixtures is completed, their fatigue behaviour is very similar to the one obtained for the HMA tested. Another important aspect to note is that the fatigue behaviour obtained for samples extracted from field was similar to that obtained for test specimens laboratory prepared.

In the same study [13], a comparison between the obtained fatigue life by ITT on cold bituminous specimens ($\varepsilon = a.N^{-b}$) with the fatigue laws proposed by Shell [14] and by the Asphalt Institute [15] was made, considering the same properties for the mixtures as the ones used in the study in the production of cold dense bituminous mixtures (i.e. $V_b = 9$ %; $V_v = 10$ %; E = 1000 MPa for early curing – about 1 week, E = 2000 MPa for medium curing – about 2 months, and E = 3000 MPa for complete curing – at least 4 months). The following conclusions were pointed out:

- The fatigue law obtained for relatively early ages (up to 2 months, E = 1000 MPa) showed a similar slope (parameter b) to the obtained by the Shell fatigue law, and a shift factor of about 1300 should be applied in order to obtain equivalent fatigue life;
- The fatigue law obtained for advanced curing stages (E = 2000 MPa / 3000 MPa) showed a similar slope (parameter b) to the obtained by the Asphalt Institute fatigue law, and a shift factor of about 130 should be applied this time in order to obtain equivalent fatigue life.

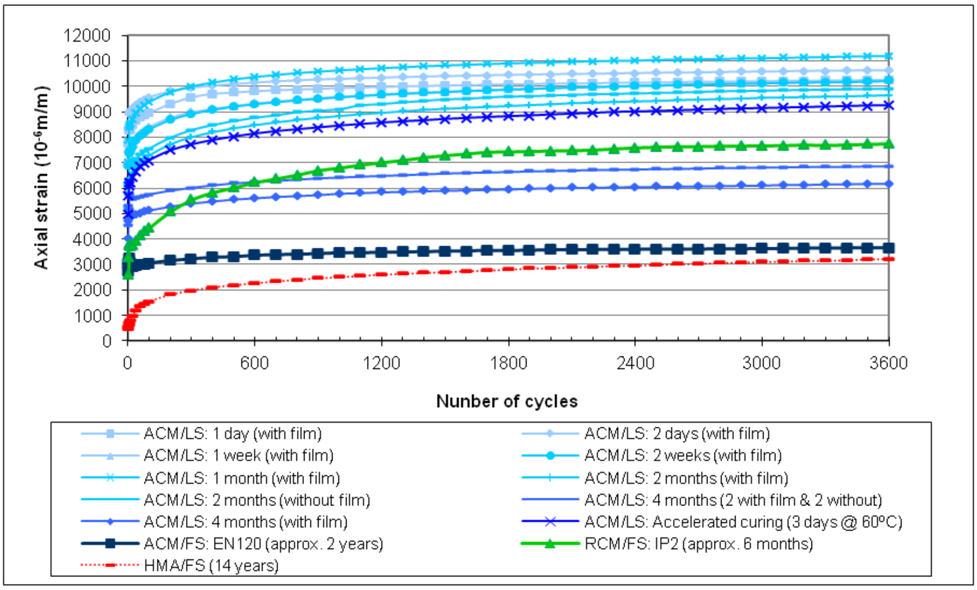


Figure 4: Permanent deformation of cold mix specimens both laboratory prepared (LS) and extracted from field (FS), determined by repeated load axial tested (adapted from Batista, 2004) [13])

With respect to permanent deformation, the results obtained through repeated loading uniaxial tests were similar to the ones obtained by wheel tracking tests, showing that cold bituminous mixes, mainly those cured for a short time (up to 2 months), present relatively high deformations in the first cycles (primary phase). The fact that cold mixtures are still curing when the test starts will allow for the mineral particles to "move" during the first load applications. When this primary phase is concluded, the deformation rate of cold bituminous mixes decreases considerably, resulting in a secondary phase with a reduced deformation rate, even when compared to HMA. Recycled cold mixtures (IP2 test specimens) showed a behaviour somewhat between "new" cold bituminous mixtures and HMA.

The above presented results, for cold mixtures produced using bituminous emulsion as binder, are in accordance with same statements presenting in chapter 1:

- At a relatively early curing (up to 2 months), cold mixtures usually behave similarly with unbound materials, since they show relatively low stiffness modulus, higher fatigue life for the same level of strain (lower slope) and higher deformations on primary phase;

- The stiffness modulus strongly increases from early ages of curing, until the curing process is completed;
- It can be considered that, upon completion of curing (for which, usually at least 4 months are needed), the fatigue behaviour is similar to those of HMA.

2.2 Notes on pavement design methods for cold recycling

2.2.1 Empirical design methods

The traditional approach to the design of the Full Depth Recycling (FDR) in the USA was the "Structural Number (SN) method" or "Granular Equivalency" used in Minnesota and "gravel equivalent (GE) method" used in California. All these methods are fully empiric. Structural number SN (or GE) is an abstract value that expresses the structural strength of the overall pavement. If no drainage effect is assumed the structural number of a pavement is

$$SN = \sum a_i \, D_i$$

where

a_i layer coefficient of the layer *i*,
D_i thickness of the layer *i* (inch).

Layer coefficients are usually written without units. In fact it is expressed as $inch^{-1}$, as the SN is a number without units.

Allowable number of design axles can be calculated from SN according to the formulae given in the literature (for example on www.pavementinteractive.com).

"Layer coefficient" or "gravel equivalent" are very approximate representation of the layer contribution to the pavement performance. Their value depends on several factors (resilient modulus, underlying support, stress state etc.). It can be evaluated from field experiments or comparative calculations for various pavements compositions. Layer coefficients can be estimated also from the results of FWD test using "effective structural number" SN_{eff} which represents the contribution of all layers above the subsoil. SN_{eff} can be calculated from the following formula

$$SN_{eff} = 0{,}0045 \times D \times E_p^{\;0{,}333}$$

where

D total pavement thickness above the subgrade (inch),
E_p effective pavement modulus of all layers above the subgrade (psi).

The formulae for the calculation of E_p from the deflection under the load plate was presented in the AASHTO Pavement Design Guide dated 1993. E_p is calculated using an iterative process. This approach was used for example in [11].

Layer coefficient for AC was usually assumed to be a = 0.44. It has been proposed to increase it to a=0.54 after the recalibration tests on NCAT track [12] carried out for DOT Alabama. However the authors concluded that the increase is "the result of the environmental conditions on the test track and care should be taken when applying this coefficient to other states.” This confirms the approximate nature of layer coefficients and difficulties with the selection of appropriate values for empirical methods.

Values a = 0.20 – 0.28 were used for foamed asphalt stabilised base materials according to [13] page 24. Values a = 0.30 – 0.35 for cold recycled mixes are given in [14]. New layer coefficient for foamed asphalt has been recently proposed a = 0.36 $inch^{-1}$ (0,142 cm^{-1}) in [15].

“Granular equivalency” GE is calculated according to the method given in [14]. Design chart permits to calculate the GE of the entire pavement that is needed for supposed traffic and subsoil stiffness calculation (which is expressed by so called R-value that depends on the resilient modulus of the subsoil). The graph contains also “minimum bituminous line” and “minimum base line”. This allows calculate the minimum thickness of asphalt and unbound layers that has to be respected even for small traffic.

“Granular equivalency” of a layer is “GE factor” multiplied by the thickness of the layer in inches. “GE Factor” of the unbound base layer is 1.0. Value of GE Factor =2.25 was used in Minnesota for surface layer of AC and 2.0 for base layer of AC. GE Factor = 1,5 was recommended in [14] for stabilised full depth recycled material, called SFDR in the report (existing asphalt layers and part of the underlying material which are blended and stabilised with some additives).

“Gravel equivalent” method used in California is similar. “Gravel Factor” G_f is the relative strength of the material compared to gravel. “Gravel equivalence” GE of a layer is GE= G_f x t where *t* is a layer thickness in feet. Gravel factor of hot mix asphalt depends on the traffic intensity. The relationship is given in the Chapter 630 of Highway design Manual of the State of California (http://www.dot.ca.gov/hq/oppd/hdm/pdf/english/chp0630.pdf). Values of G_f for AC are in the range from 1.5 to 2.5. Cold in place recycled asphalt has G_f=1.5. Gravel factor for cold recycled mix with emulsion is G_f=1.4, if the RAP content is lower than 50 % and G_f=1.5 if the RAP content is > 50 % and fine content is low. G_f for aggregate subbase (AS) is G_f=1.0 and for aggregate base (AB) is G_f=1.1 according to the table in chapter 660 of the Manual. Development of the gravel factor G_f for foamed asphalt was described in the report [16].

Gravel equivalent method in still used in the latest version of the design manual which is on-line on the web page of the DOT of California. However the Manual includes the following note “Mechanistic-empirical analysis procedures can also be used for FDR pavement structure design.”

In Spain, the technical specifications for road pavement rehabilitation [22] establish two different types of in situ cold recycling of bituminous layers (in 6-12 cm depth) using bituminous emulsion, depending on the thickness of the recycled layer: RE1 for applications in layers with larger thickness (≥ 10 cm) and RE2/II for layers where the thickness is between 6 cm and 10 cm. For each of these types of cold recycling, different grading envelops are required, but some other requests are only dependent on the traffic levels. As regard the design of pavements containing cold recycled layers with emulsion, a normative document issued in 2003 [23] refers that special studies should be conducted, recommending that a fatigue law of the recycled material should be determined, but failing that, allowing for simplification that the evaluation of the required recycled layer thickness be performed can use a coefficient of equivalence of traditional asphalt concrete thickness of 0.75.

It is worth mentioning, that some Spanish authors [24] have traditionally considered three different types of in situ cold recycling of flexible pavements using bituminous emulsion, as follows:

- RFE-I: When recycling bituminous layer (< 4-5 cm) plus a base granular layer, using bituminous emulsion in approx. 4-7 % content, and for final cold recycled layers of 8-12 cm thick.
- RFE-II: When recycling bituminous layer (< 5-10 cm) plus a base granular layer (in relative percentage lower than 50 %), using bituminous emulsion in approx. 3-5 % content, and for final cold recycled layers of 8-12 cm thick as well.
- RFE-III: When recycling only bituminous layers, using bituminous emulsion in approx. 2.5-4 % content, and for final recycled layers of 6-12 cm thick.

For the above mentioned types of in situ cold recycling, the following coefficients of equivalence between the recycled materials and traditional asphalt concrete base layers are provided: RFE-I: 0.6; RFE-II: 0.7; RFE-III: 0.8.

In Portugal, similar procedures are adopted in the pavement design comprising in situ cold recycled layers.

Recently, a similar method to “Structural number” or “gravel equivalency” method was implemented in Ireland. It is presented in [17]. The structural equivalence number SEN is calculated using the formulae

$$SEN = \sum_i \boldsymbol{h}_i \times E_i^{0,333}$$

where

h_i thickness of the layer (m),
E_i design stiffness of the layer (MPa).

Long term design stiffness values for bitumen bound materials are given in the table of the Manual. Minimum SEN values for each Road Type Category that have to be achieved are specified in the Manual.

This is the application of the equivalent stiffness method proposed by ULLIDTZ. Even if the equivalent layer theory is approximate (as shown for example in older work [18]), it is sufficient for this purpose.

Similar approach in specifying long term stiffness modulus for cold stabilised materials was used in the South Africa in Pavement Number (PN) design method implemented 5 years ago [19]. However the Effective Long Term Stiffness (ELTS) is not a stiffness value that can be determined by means of a laboratory or field test. It is a model parameter, which is calibrated for use in the PN design method and it may therefore differ from stiffness values typically associated with material classes. As such, the ELTS averages out the effects of decreasing stiffness owing to traffic related deterioration, as well as seasonal variations in stiffness. Thus the ELTS does not represent the stiffness of a material at any specific time. There are other parameters then stiffness which are also considered. The layer contribution is calculated for each layer by multiplying the thickness, the ELTS, the “thickness adjustment factor” and “Base Confidence Factor” (BCF). There is a graph in the Guide that gives relation between pavement number PN and allowable number of ESALs.

Advantage of the Irish or South African method over the above mentioned American empirical methods is that the stiffness modulus of each pavement layer is directly included in the basic formulae. This is more understandable for investors and designers not specialised in the pavement design than layer coefficients method (even if the stiffness of road materials was considered during the establishment of layer coefficients and similar empiric parameters).

It has to be kept in mind that all empirical methods mentioned here were validated in the country and state where they were developed, but their transmission to other climatic conditions is delicate. An interesting case of a premature cracking in foamed bitumen pavement along the length of about 80 km designed with empirical methods has been recently described by [20].

2.2.2 Analytical design methods

Analytical methods use various approaches. Some methods evaluate only deformations of cold recycled materials other also their fatigue. New Zealand method assumes unbound behaviour of foamed bitumen bound layers. Pavement is designed in such a way that the elastic strain on the top of the subsoil is lower than allowable value for supposed traffic. Australian method considers fatigue behaviour of foamed bitumen by empirical formulae. Laboratory verification of fatigue properties is not used in Australia.

The comparison of the 3 pavement design methods for foamed bitumen is given in [21]. Different pavements were obtained by these 3 pavement design methods. Differences in the usual mix composition and design philosophies are also described in that paper. Pavement structures on the subgrade with E = 50 MPa for 5 and 10 millions ESALs were compared. Results for 10 millions ESALs are presented in table 2. Design stiffness values for recycled layer are also given as well as the typical cement and bitumen content.

There are big differences in the design stiffness of recycled layer. This corresponds to the typical compositions of the mixes used.

Comparison of different pavement design methods for foamed bitumen mixes were carried out in [22]. Australian and England method which are based on fatigue behaviour and NZ and South African method were compared and one example was calculated by all four selected methods.

Table 2: Comparison of pavement design with foamed bitumen from three countries

Parameter/layer	**NZ**	**Australia**		**South Africa**	
Thin overlay	Chip seal	Chip seal	AC 40	AC 40	AC 55
Recycled layer (mm)	185	320*	270	265	195

Unbound layer (mm)	265	130	130	200	200
Cement content (%)	≤ 1.5	≤ 2.5 hydrated lime		≤ 1 % cement or lime	
Bitumen content (%)	2.7 – 3.0	3.0 – 4.0		1.7 – 2.5	
E for recycled layer (MPa)	800	1 960		486	
NOTE: placed in 2 layers					

The Mechanistic Empirical Design Guide (MEPDG) developed under NCHRP Project 1-37A has been adopted in some states in USA since 2009. Initial version of the MEPDG Manual is still on http://onlinepubs.trb.org/onlinepubs/archive/mepdg. Comprehensive information on MEPDG software is on www.AASHTOWare.org or on www.me-design.com/MEDesign. This new pavement design guide incorporates input parameters based on performance criteria. Among them are bottom-up and top-down fatigue cracking, permanent deformation, etc. Basic idea of MEPDG is to use pavement models based on the mechanics of materials to predict pavement responses (as strains or stresses) and on the use of empirically based transfer functions to estimate distress initiation and development based on these responses.

Various calibration parameters denoted as β are used in all formulas for the performance criteria. Procedures and inputs for national calibration of all models were presented in annexes of the guide published in 2009 (calibration of fatigue model is in annex II and calibration of deformation model in annex GG). The Guide for the local calibration of MEPDG was published in 2010. Some webinars on local calibration of MEPDG are on internet (www.asphaltfacts.com/webinars or www.aashto.org).

Estimation of fatigue damage in MEPDG is based upon Miner's law. Total damage is the sum of damage in individual periods. Number of repetitions to fatigue cracking N_f is presented in MEPDG in a following form.

$$N_f = \beta_1 \; k_1 \, (\varepsilon_t)^{-\beta_2 \, k_2} \; (E)^{-\beta_3 \, k_3}$$

where

ε_t	tensile strain at the critical location,
E	stiffness (stiffness modulus) of the material,
k_1, k_2, k_3	laboratory regression coefficients,
β_1, β_2, β_3	calibration parameters.

National calibration made in the USA for MEPDG led to the following formula:

$$N_f = 0{,}00432 \, k'_1 \; C \left(\frac{1}{\varepsilon_t}\right)^{3{,}9492} \left(\frac{1}{E}\right)^{1{,}281}$$

where

k'_1	function of the asphalt layer thickness,
C	laboratory to field adjustment factor.

Calculated N_f in every period is used for the evaluation of damage by Miner´s law. Damage is then transformed into the value of the fatigue cracking FC (expressed as the percentage of the total lane area) using sigmoid function containing 3 calibration factors (C_1, C_2, C_3). Finally the designer has to select the design reliability to be able to draw the curve representing assumed developments of cracking during the life time of the pavement. The whole procedure is very complex and requires extensive back calculation and calibration for each national specific climatic and traffic loading condition. Further the method has not yet been widely applied on cold recycled mixes. Therefore it is not yet practically available for European pavement design applications. However, some parts of it are used in the method proposed for the cold recycled mixes in the chapter 5 of this report.

Some information on the use of MEPDG method for cold recycling are given in [23] but there are still few information at present on the material properties of cold in place recycling (CIR) and Full-depth reclamation (FDR) available for the design of pavements with MEPDG.

That is why the research project NCHRP 09-51 has been started in 2012. The objective of this research is to propose material properties and associated test methods and distress models for predicting the performance of pavement layers prepared with CIR of asphalt concrete and FDR of asphalt concrete with aggregate base and minimal amounts of subgrade material using asphalt-based materials. The end of this project is planned in 2014. Brief information about this project was published in a presentation [24].

Fatigue test is not among the tests planned in the NCHRP 09-51 project. It is probably related to the fact that laboratory fatigue tests have not been used for the final calibration of the model in MEPDG. Bottom up fatigue cracking has been evaluated in MEPDG using empirical formula established on the basis of validating some trials and observation of pavements behaviour included in the Long Term Pavement Program (LTPP) – described by FHWA web: http://www.fhwa.dot.gov/research/tfhrc/programs/infrastructure/pavements/ltpp/.

A mechanistic pavement design procedure for asphalt pavements was issued in Germany [41]. Classical fatigue theory is introduced as the design principle, where fatigue damage at the bottom asphalt layer is avoided by limiting the strain at the bottom of the asphalt base course. Further design criteria are vertical deformations of the subgrade and the unbound base layers which are controlled by limiting the vertical stress on top of these layers. For hydraulically bound base layers, the horizontal stress at their bottom is analysed. For all design criteria Miner’s law is applied.

To calculate the stresses and strains, linear elastic multilayer theory is applied. Herein, the road pavement is divided into homogeneous sub-layers. Each sub-layer is described by the parameters thickness, elastic modulus, Poisson's ratio and bonding to the underlying layer. Due to the iterative mechanistic design approach, the actual traffic conditions and the temperature conditions in the pavement can be considered in more detail. Heavy vehicle load exposure is represented by a number of 11 load classes representing axle loads between 2 tonnes and 22 tonnes. The frequency distribution of a load class is selected with respect to the road category. Temperature exposure occurring during the year is represented by 13 surface temperature classes. Each surface temperature is associated with a temperature development within the pavement, which can be calculated by classical law of heat transfer. As a result, 13 typical pavement temperature distributions were derived, each with a specific annual distribution of frequency. A typical distribution of the frequency of occurrence can be seen in Figure 5.

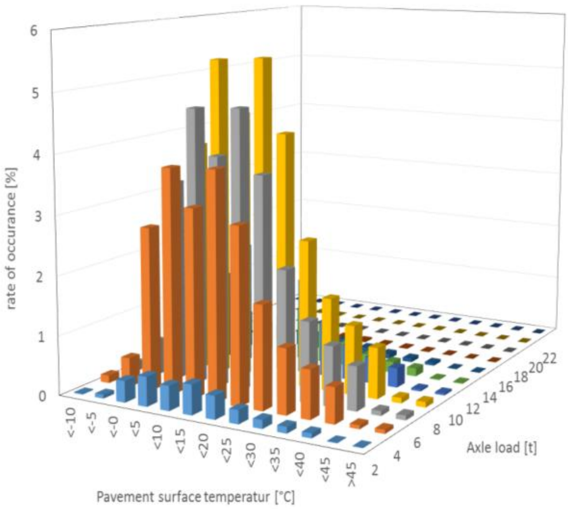


Figure 5: Frequency of occurring of 143 load and temperature classes analysed during German mechanistic-empirical pavement design procedure, [41]

Thus, every asphalt sub-layer can be associated with a constant temperature. For each surface temperature class, the layer properties of the individual sub-layers are kept constant which allows the calculation of the horizontal bending tensile strain ε at the bottom of the asphalt layer. For one specific geometrical pavement model, the design process covers a total number of 143 individual models, resulting from 13 temperature cases and 11 load cases. Results of Indirect Cyclic Tensile Stress Tests (IDT) are used to estimate the number of load cycles which can be endured without any material failure. From these tests fatigue equation is derived incorporating the parameter "a" and the exponent "k" (cp. Chapter 3.2). Any difference from laboratory test to real pavement conditions is covered by a shift factor SF as well as a safety factor F. For each of the 143 calculated strain values ε, the maximum allowed number of load cycles is calculated. Miner's hypothesis is used to estimate accumulation of fatigue damage (Equation 2). Resistance to fatigue macro cracking is given as long as the sum of the partial damages is less than or equal to one.

Further failure modes are applied for hydraulic bound base layers (applying the bending strength for estimating the fatigue resistance) and unbound base layers and subsoil, where the layers bearing capacity expressed as modulus derived from plate bearing test is applied as a factor for determining the allowed permanent deformation. Later approach goes back to design approaches developed by HEUKELOM [40].

There are few recommendations in the literature for the fatigue parameters of recycled mixes for the pavement analysis due to the difficulties with the fabrication of specimens for fatigue tests and variability of results (compare CoRePaSol Deliverable D2.1_fatigue). Some information on fatigue properties of recycled mixes was presented in [25]. Parameter *n* for frequently used formulae $N = K(1/\varepsilon)^n$ was in the range of 3.5 to 7.4. The parameters *K* and *n* for the mixtures considered were related by the empirical formulae obtained from 12 mixes (mixes with foamed bitumen and emulsion considered together)

$$\log K = 4.81 - 3.49n$$

Which gives for n = 4.0 value of K = 8.128E-10 and for n = 5.0 value of K= 2.630E-13.

The rearranged fatigue relation, which includes the parameter ε_6 according to EN 12697-24 for the fatigue test of asphalt mixes (which is used in the Czech and French pavement design method) can be shown in following formula:

$$N = 10^6 \left({}^{\varepsilon_6}/_{\varepsilon} \right)^B$$

which is preferred in this text over the more common formulae $N = K(1/\varepsilon)^n$.

Rearranged formulae shows better the relation of fatigue parameter ε_6 to the mixture strain ε. Both values can be expressed for simplicity in microstrains (μm/m or 10^{-6}). It avoids the computation with very low values of the parameter *K* and the formula is easily understandable even for laymen. The ε_6 values recalculated from the formulae given in [25] are as follows:

ε_6 = 300 μm/m for K = 8.128E-10 and
ε_6 = 192 μm/m for K= 2.630E-13.

Values of ε_6 calculated from the Thompson´s formula are given here only as an example of the possible approach. It is well known that fatigue parameters depend not only on the mix composition, but also on a temperature, test type and loading conditions, etc. Parameters described in [25] are much higher than values used for pavement design in France and Czech Republic, as fatigue parameters for these methods are related to 2PB fatigue test at 10°C. The type of fatigue tests and test temperature quoted in [25] were not mentioned in the paper. Probably the formula was related to 4PB test at 20°C which is common in USA.

Results of the recent indirect tensile fatigue tests (ITFT) on cold recycled bituminous emulsions mixes were presented in [26]. Detailed description of the stiffness and fatigue tests at 20 °C and 30 °C is in the thesis of OKE, [27]. The emulsion content was 6.5 % by aggregate mass. Binder content was sufficient to produce the fatigue behaviour of the cold mix. The fatigue behaviour of these mixes was compared to the hot mixes. Hot mixes had the slope of the fatigue line in ITFT test ≈4, but cold recycled mixes only about ≈2. Thus the strain for 1 million cycles to failure was much lower for cold mixes than for hot mixes.

2.3 *Design catalogues*

For traditional pavement materials, some countries developed design catalogues giving suitable pavement structures and layer thicknesses according to empirical analyses for specific regional parameters. In this section, the design catalogue applied in Germany according to [43] and [44] will be compared with typical pavement structures as included in Wirtgen cold recycling manual, [1]. In order to allow the comparability between the existing design procedures, common parameters for loading, soil conditions and climatic conditions are defined. Based on these model pavements the resulting pavement designs as evaluated by various methods are compared.

2.3.1 Pavement parameters

For the comparative pavement design analysis, three typical traffic loads are defined, as summarised in Table 3 and Table 4. The parameters for the varied cold recycling model mixtures are defined in Table 5.

Table 3: Traffic load parameters for pavement model designs

Model traffic load		**T1**	**T2**	**T3**
Type of road		low volume road	main rural roads	Multi-lane highway
Annual average daily traffic (AADT); % HLV; 2.0 8 t ESAL / vehicle (1 lane, 1 direction; no gradient, wide lane width, no traffic growth)	AADT (lane)	200	1.000	10.000
	% of HLV	5 %	10 %	10 %
Number of 8 t ESAL (30 years)		1 million	10 million	100 million
Number of 10 t ESAL (30 years)		0.4 million	4.1 million	41 million

Table 4: Subbase parameters for pavement model designs (layer below cold recycled mixture)

Model subbase	**B1**	**B2**	**B3**
Example	low capacity sub-soil below cold-recycled layer	standard frost-resistant layer below cold-recycled layer	high sub-base bearing capacity
CBR [%]	5	20	50
Modulus (loading plate test E_V) [MPa]	50	120	200

Table 5: Cold recycling model mixtures

Model mixture	**M1**	**M2**
Type of cold recycled mix	BSM	bitumen-dominant cold mix
Bitumen content	≤2 %	>2 %
Cement content	≤ 1 %	≤3 %
Stiffness @ 5°C [MPa]		~ 5 000
ITS @ 5 °C [MPa]		≥ 0.75 ; ≤ 1.20
ITS @ 15°C [MPa]	≥ 0.30	≥ 0.30 ; ≤ 0.70

2.3.2 Model pavement design

Based on these model pavement parameters, as defined in section 2.3.1, pavements were designed according to relevant design procedures for cold recycling mixtures as applied internationally as well as selected European countries.

The results are given in **Ошибка! Источник ссылки не найден.**6. In order to allow a clear comparison of the design results, all designed pavements are flexible structures with following layers (from bottom to top). The design thickness values [mm] are further formatted differently:

⇨ subbase (soil, existing unbound base) (no thickness) – *added unbound base (CBR/mm)* – **cold recycled mix** – hot-mix asphalt (mm).

Table 6: Results of comparative model pavement designs

Traffic load	**Subbase cond.**	**Model cold-recycled material**	**Pavement structure and layer thickness (mm):** asphalt **– cold recycled material –** ***unbound base layer / CBR***	
			Wirtgen [1]	Germany [43, 44]
T1	B1	M1	5 – **150** – *50/10 % (∞/5 %)*	
		M2		80 – **160** – *∞/5 %*
	B2	M2		60 – **140**– *∞/20 %*
	B3	M1	5 – ***100*** – *150/10 % (∞/20 %)*	
		M2		
T2	B1	M1	40 – **250** – *140/50 % 150/10 % (∞/5 %)*	
		M2		180 – **180** – *∞/5 %*
	B2	M1	40 – **125** – *150/50 % (∞/20 %)*	
		M2		120 – **200** – *∞/20 %*
T3	B2	M1	50 – **250** – *50/80 % 150/50% (∞/20 %)*	

As can be seen in table 6, the two compared design catalogues refer to different types of cold recycling materials. Nevertheless, in figure 6 two pavement structures as highlighted in table 6 by frames are shown for comparability. It can be concluded, that the overall pavement thickness is similar for both approaches. Nevertheless, in German design guide, thicker hot-mix asphalt surface layers are applied. This may refer to higher requirements for evenness and thus lower allowed rutting depth.

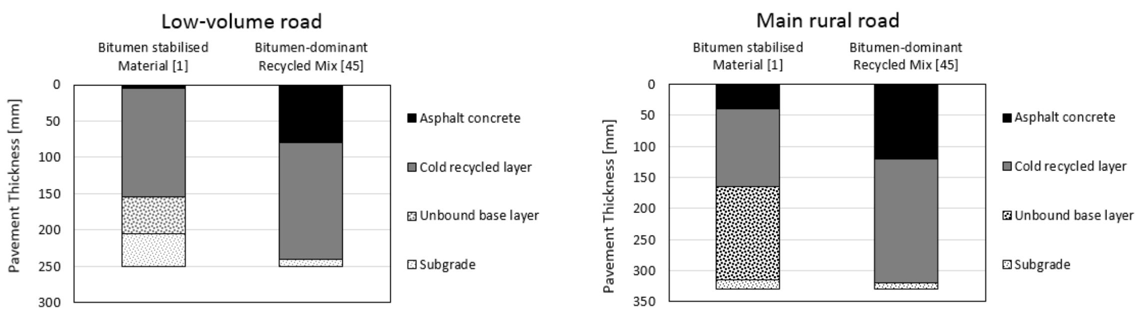


Figure 6: pavement structures for comparable traffic and subgrade conditions according to Wirtgen and German design guide

3 Conclusions from the literature review

Even if there have been some large scale experiments in accelerated loading facilities and experiments field trials during the last 3 years which brought many interesting information on the pavement behaviour as well as new research work in various countries a reliable pavement design method for pavement with cold recycled layers is still lacking.

The fatigue behaviour similar to the hot mixes was observed in some cases, but frequently the resistance against rutting is decisive for the proper long term behaviour of pavements with cold recycled layers.

The use of various monograms, tables or catalogue with the thickness of recycled layers and asphalt overlay created on the base of practical experience with the behaviour of existing pavements remains the standard pavement design method. The design monograms for different cold recycling technologies have been developed for example in Canada. It can be downloaded from the web page http://www.bitumequebec.ca/_publications (section “autres publications”).

Empirical design method as American SN, G_f , GE method, Irish and South African method used today will probably remain also in the future in use due to simplicity of their application.
However either simple or more sophisticated analytical method can be used for important projects or for forensic investigation of some premature damage. This approach will be described later in the report. Some notes on the problems related to the analytical pavement design are at first mentioned in the next chapter.

4 Notes on the application of fatigue tests of asphalt mixes in the analytical design for flexible pavements

There are important differences in the evaluation of the flexible pavement fatigue resistance in various analytical pavement design methods. The problems related to the application of the fatigue parameters for the pavement design have been described recently in detail in a series of Czech papers, [28].

The fatigue tests after EN 12697-24 done by 2PB or 4PB test method are not suitable to cold recycled mixes, due to the problems with the fabrication of the test specimens. The tests on cylindrical samples are preferable for cold mixes. However it is well known that the number of cycles to the failure in indirect tensile or uniaxial compression/tension or tension tests is lower than for bending tests. That is why other shift factors have still to be developed for cold mixes in countries where the reference fatigue test for hot mixes used in the pavement design method is 2PB or 4PB test methods.

Some pavement design methods are based on the field experiments only, as MEPDG in USA, which does not use laboratory fatigue tests. The methods which use the results of laboratory fatigue test apply some shift factor between laboratory and real pavement. Some of them use the safety factor approach (as for example German and new Austrian pavement design method), other use a couple of partial factors related to the reliability as the Czech method or allowable stress (or strain) as the French method. It can be expected that these states would prefer to use the same design principles also for cold recycled mixes, for the case that fatigue resistance of these mixes is taken into account in the future. This complicates the establishment of a common approach for different European states.

Anyway a distinction has to be made between the corrective coefficient which assures that the cracking damage has a low probability and corrective coefficients for the shift between laboratory and pavement (due to the rest periods between loadings, traffic wander, crack propagation to the surface of the pavement etc.).

The coefficient assuring the confidence level is called “coefficient of dispersion” *SN* in the French pavement design method and “partial variance coefficient of the fatigue test” γ_{up} in the Czech pavement design method. The shift coefficient between laboratory and pavement is called “coefficient de calage” k_c in the French method and “coefficient of the application of fatigue test” γ_u in the Czech method.

Czech and French pavement design method suppose the parallel shift of the fatigue line to assure low probability of pavement damage. The design value of the slope of the fatigue line for hot mixes asphalt in log-log scale is fixed in both methods as B = 5.0. Statistical evaluation on a large number of laboratory fatigue tests on hot mix asphalt in 2PB tests according to EN 12697-24 presented in [29] confirmed that the slope of the fatigue line for hot mix asphalt is around 5. The exponent 5 is used in the formula for allowable number of design axles in new Austrian method (proposed in the framework of the project OBESTO [30]). The exponent 5 is also used in the Australian pavement design method for allowable number of design axles. (The same formula is used for foamed asphalt and for asphalt concrete in Australia). However other slopes of the fatigue line are obtained for another type of fatigue test.

The measured value of the fatigue parameter ε_6 from the laboratory test on AC according to EN 12697-24 can be used by the designer in Czech, French and Austrian pavement design method (with some limitations specified in every method). Slope of the fatigue line has to remain B = 5, even if another value is obtained in the laboratory fatigue test.

The Czech design method permits the maximum increase of ε_6 by 10 % in comparison to the design value given in the design manual TP 170. French norm for pavement design NF P 98-086 gives for asphalt mixes for performance related approach (“approche fondamentale”) in Annex F the maximum value of ε_6 for different types of asphalt mixes (maximum value is 10 or 15 µm/m higher than the minimum value design value for empirical approach). New Austrian pavement design method uses safety factor which value depends on the value of the ε_6 measured in the laboratory (formula is on the page 88 of [30]). This approach is more logic than to fix arbitrarily the upper limit for the increase of laboratory measured value above the design value given in the Manuals.

If the shift between laboratory and pavement is realised for the strain (or stress) then the corrective coefficients are relatively small. The coefficient in the Czech method called “coefficient of the application of fatigue test” is γ_u= 1.6. This means that the shift factor expressed in design axles is $(\gamma_u)^B$=1.6^5= 10.5. Similarly the French method uses in the formula on the page 15 of the norm NF P 98-086 shift factor called “coefficient de calage” k_c = 1.3 which increases allowable strain $\varepsilon_{t,adm}$. This increases the allowable number of design axles 1.3^5 = 3.7 times.

If the allowable number of design axles is calculated from number of cycles from laboratory fatigue equation, much higher shift factors will be applied. For example, in German mechanistic –empiric design guide [41] a shift factor of SF = 1.500 is applied for linking the fatigue test results obtained in cyclic ITFT to the number of allowed cycles on site. Assuming a fatigue function exponent B = 5, this would result in a factor applied directly on the strain of 4.3. This factor is considerably higher compared to the French or Czech method because of the applied stress-controlled fatigue test which results in significant lower fatigue life for a given value of strain ε compared to strain-controlled fatigue tests. Assumption that the slope of the fatigue line for all asphalt mixes is the same, simplifies the specifications of design values of fatigue parameters in Design Manuals. The application of fatigue test for the pavement design for cold recycling mixes is more complicated especially for mixes with two different binders (hydrocarbon and hydraulic one), as the slope of the fatigue line depends on the quantity of hydraulic binder in the mix. Slope B increases with the increase of the cement content.

Another problem is that the fatigue is usually expressed in analytical pavement design methods from strain controlled tests for asphalt mixes and from stress controlled tests for hydraulic bound mixes. This approach is used in the French design method, in American MEPDG and German RDO. The question arises how to express the fatigue for mixes with two types of binders – bituminous and hydraulic binders.

That is why it is difficult to give some design values of fatigue parameters in the manuals for pavement design of cold recycled mixtures. The content of cement and bituminous emulsion in the practice depend not only on the mix properties, but also on the ratio between the price of cement and bituminous emulsion. If the price of cement is low in comparison to emulsion contractors have tendency to use more cement and less bituminous emulsion.

To avoid such approach, the road administrations could specify the ratio between bituminous emulsion and cement in the mix and in parallel fix some design fatigue parameters in the design manual. Alternatively the binder ratios would be indicated just as an indication to have the possibility for fatigue life prediction. However if the mix composition is not specified in advance in the manual, it is very difficult to estimate fatigue parameters from simple tests as unconfined compression test or indirect tensile strength test. Even if the fatigue test on cold recycled mix samples will be carried out for an important project, the conservative evaluation of the test results would be necessary, as the global experience with fatigue tests on cold recycled mixes is very limited in comparison to hot mix asphalt.

The Guide for cold recycling has been issued by the French administration in 2003, [34]. It contains also the instructions for the pavement design. There are 5 classes of recycling techniques or approaches there. The first three classes are for recycling with emulsion, class 4 is for hydraulic binders and class 5 for so called "composed binders" (mixtures with hydraulic and bituminous binders). The foamed bitumen is not included in this guide.

The design guide for cold recycling with bituminous emulsion and cement contains fatigue parameters only for one mix composition (2 % of cement +3 % of bituminous emulsion). This high dosing of binders surely assures the bound behaviour of the mix. Thus the fatigue should be considered in the design. However this mixture composition is given in the guide only as an example. The guide does not state unequivocally how to proceed if other mix composition is selected.

Pavement design for cold recycling is closely related to the French pavement design method described in the manual issued in 1994 by the French national road administration (English version of the guide [35] was published in 1997). The design method for new pavements has been issued in 2011 as a French norm [36]. However this norm does not treat pavements with recycled layers.

Mechanical parameters for the pavement design are given in the guide [34]. There are 2 qualities of recycling R1 and R2. Higher quality R1 is for higher traffic load.

Design values given in table 7 are recommended for the recycling with emulsion only.

The fatigue parameters of recycled layer with bituminous emulsion only are not needed for the pavement design.

Recycling with cement is based on the evaluation of the fatigue strength. Horizontal tension strength has to be higher than allowable stress for the estimated traffic. Recycling with composed binders can be evaluated for strength or for deformations (that is slope of the fatigue line and parameter σ_6 or ε_6 can be considered).

Table 7: French classification for cold recycled mixes and their use

Class	Goal	Module @15°C (MPa)	R_c (MPa)	Criterion for design
I RAP < 75 %	reinforcement	1 500 2 500	1.5 – 2.2 2.2 – 3.0	Vertical subsoil strain
II RAP 75 - 90 %	rehabilitation	2 000 3 000	Rc < 4.0 Rc > 4.0	Vertical strain on subsoil and recycled layer
II RAP > 90 %	rehabilitation	3 000 4 000	Rc < 4.0 Rc > 4.0	Horizontal strain at the base of AC above recycled layer
III	reinforcement, rehabilitation	4 000		Horizontal strain at the base of AC above recycled layer
NOTE: R_c is unconfined compressive strength (Duriez) after 14 days **Rehabilitation means the repair that does not increase the bearing capacity of the pavement.**				

Recycling with emulsion and cement (named “liants composés”) is described in the part 3 of the mentioned guide. It is carried out according to the pavement design method for new pavements with aggregates bound with hydraulic binders. Thus the value of allowable stress is calculated. Fatigue parameters are expressed as σ_6 and slope B. However there are some differences in corrective coefficients.

The shift factor called “coefficient de calage k_c” of the recycled layer has the value $k_c = 1.6$, if the remaining part of the existing road is at least 5 cm. Otherwise shift factor is $k_c = 1.5$. This is slightly higher than the values for aggregates bound with hydraulic binders which have k_c = 1.5 or 1.4 (see table F.4 of the norm NF P 98-086) and greater than for asphalt concrete which has $k_c = 1.3$ (see table F.5 annex F of the norm). Thus the allowable stress on the base of the recycled layer $\sigma_{t,adm}$ can be higher. The coefficient of dispersion SN for the quality class R1 is SN = 1.0 which is equal to the SN for aggregates bound with hydraulic binders, but for the quality class R2 is SN = 1.5. The higher SN means the lower allowable stress. Also the coefficient of the dispersion of the thickness of the recycled layer S_h is the same as for aggregates bound with hydraulic binders for the quality class R1, but higher for class R2.

There is an example of a mix with 2 % of cement and 3 % of emulsion in the manual that has the design stiffness of 5,500 MPa and the slope of the fatigue line B = 9.5. The slope is lower than for cement treated aggregates in the French norm (slope B = 10 to 15 according to the type of the mix), but distinctly higher than for asphalt concrete which has B = 5. This corresponds roughly to the mutual differences of the fatigue line slope in indirect tensile fatigue test of cold mixes with natural aggregates described in [31]. Slope of the fatigue line was B = 3.9 for hot mix, B = 2.9 for cold mix with emulsion only and B = 5.6 for cold mix with emulsion and 2 % of cement. The values of ε_6 were 47 μs for AC, 29 μs for cold mix and 59 for cold mix with emulsion and cement.

It is well known that the fatigue resistance of asphalt mixes depends also on the temperature. This was demonstrated by many laboratory research studies on hot mixes (for example [32]). Differences in the fatigue parameters of cold recycled mixes with emulsion only which were tested at 20°C and 30°C were observed in [26]. It can be assumed that the temperature sensitivity of the fatigue properties will be smaller for cold recycled mixes with bituminous emulsion and cement.

Some design methods admit that the parameters in the formulae for allowable number of design axles depend on temperature. The Austrian pavement design method assumes unique relation between parameters K_1, K_2 and temperature for all hot asphalt mixes. K_2 decreases with temperature (from $K_2 = 6.2$ for 0°C to $K_2 = 5.0$ for 20°C).

MEPDG and RDO assume that these two coefficients do not depend on temperature. However the impact of temperature on the allowable number of design axles is considered in MEPDG by implementing the stiffness modulus into the fatigue equation. This facilitates the recalibration of the formula according to the results of accelerated loading test as shown e.g. in [33]).

There is one supplementary problem in analytical pavement design with cold recycled mixes in comparison with hot mixes. The resilient modulus of cold recycles asphalt mix depends on the stress state. This non-linear behaviour has an impact on the strain on the base of the cold mix base layer in the pavement.

The comparative calculation in [27] showed that the strains in cold recycled base layer of the pavement calculated by Kenpave computer program which considers non-linear behaviour were distinctly different from standard linear elastic analysis calculated by BISAR program. Non-linear behaviour was assumed for the base, sub base and subgrade. Graphs with the distribution of vertical and horizontal stresses and strains for the 2 pavements are presented in [27]. The pavement with 50 mm of AC, cold recycled base 200 mm, granular sub-base 200 mm is called “case 6” in [26, 27]. The horizontal strain in base layer about 50 μs was calculated by Kenpave, but the strain calculated by BISAR (with elastic modulus of the base layer 3000 MPa) was 4 times higher.

The difference between linear and non-linear model will depend on the pavement composition and resilient properties. Thus this individual result cannot be generalised, but the difference between linear and non-linear model surely exists. The design of cold recycled mixes presented in [26] was based on nonlinear model. The shift factor of 77 was used for number of cycles for crack initiation and 440 for failure. However it was admitted that this shift factor might not be appropriate for cold mixtures and it was stated that “no universally accepted values for cold mixtures are available at present”.

Considering all the uncertainties and problems related to the performance properties of cold recycled mixes the Czech pavement design manual uses as input for the cold recycled mixes only their stiffness. The fatigue resistance of cold mixes is not considered in the pavement design. The elastic modulus and Poisson´s ratio of cold recycled mixes are applied and the pavement design is carried out as with other road materials. Thus the horizontal strain at the base of asphalt layers above recycled layer and the vertical elastic strain on the top of the subsoil are considered in the analysis. It is surely sufficient for the low to medium traffic. However it would be preferable if the fatigue of cold recycled mixes could be taken in account for important job sites for mix compositions where long term bound behaviour could be expected, mainly for heavy loaded roads with high to very high traffic intensities.

Some Spanish authors [24] have considered three different types of in situ cold recycling of flexible pavements using bituminous emulsion, as follows

As stated before (see 2.2.1), in Spain three types of cold recycling are traditionally considered (RFE-I, RFE-II and RFE-III). Some authors [24] proposed the following guideline values for recycled layers (table 8), according to the used analytical approach.

Table 8: Spanish guidelines values for cold recycled mixes

Class	Target	Dynamic modulus, (MPa)	Poisson ratio	Cold recycled layers thickness
RFE-I	Improved mechanical or geometrical characteristics of existing pavement	1 200 - 1 800	0.35	8-12 cm
RFE-II	Idem type I and eventually regeneration of existing binder	1 500 - 2 500	0.35	8-12 cm
RFE-III	Recycling and regeneration of existing binder	2 500 - 3 500	0.35	6-12 cm

5 Proposal for the analytical design methods for cold recycled mixes

Summing up the results of the literature review as well as from the comparison of international approaches for pavement design following conclusions can be drawn to propose harmonised approaches for analytical pavement design of cold recycled materials:

- **Stiffness** of the cold recycled mixture is relevant for failure modes of the cold recycled pavement layer itself as well as for the failure modes of other pavement layers (e. g. asphalt base course fatigue, sub-base deformation). Stiffness will be dependent on temperature, speed of loading and stress state. Further the loading of the other layers in the pavement above the cold recycled material is significantly affected by the interlayer bonding to the cold recycling layer.
- **Fatigue** of the cold recycled pavement layer should be of importance for high bitumen contents (> 2.5 % residual bitumen content) as well as high hydraulic binder contents (> 3.0 %).
- **Permanent deformation** of the cold recycled pavement layer in case of low binder contents determined by terms of suitable triaxial test.

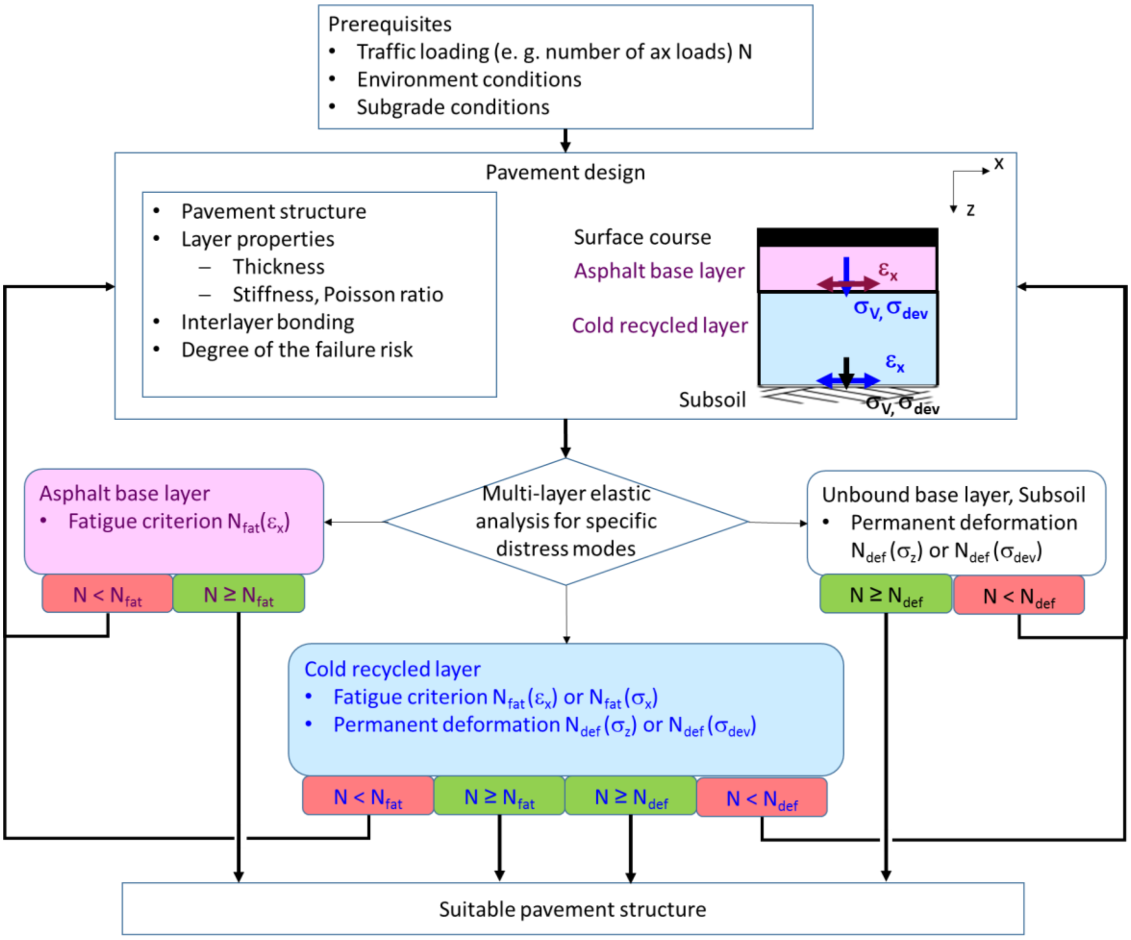


Figure 7: Analytic design principle for pavement with cold recycled layer

Fatigue resistance of cold recycled mixes with lower bituminous binder content (proposed are <2 % of residual bitumen) can be neglected. Design criterion for sub-grade strain shall be used. Vertical strain on the top of the recycled layer should be checked as well.

The proposed analytic design procedure is shown in Figure 7. For the design calculations and design checks on the asphalt base layer as well as subsoil, existing design approaches can be applied. For the design checks of the cold recycled layer at least two failure modes have to be checked.

5.1 Design criteria for fatigue of cold recycled layer

Fatigue resistance of cold recycled mixes with higher bituminous binder content should be considered. The minimum binder content which permits the consideration of the fatigue behaviour will depend also on climatic conditions. Based on the behaviour during accelerated load tests and some laboratory fatigue tests it can be tentatively assumed as a first approximation that bound behaviour can be expected for mixes with bituminous emulsions and cement with more than 2 % of cement and at least 4 % of total binder content (cement + residual bitumen from emulsion and RAP). This can be adjusted when more laboratory test results or field experiments will be available. In this respect it is highly recommended to continuously collect necessary data related to monitoring of cold recycled mixes/pavement performance.

Due to the differences in analytical pavement design methods in European countries, problems with the realisation of fatigue tests and limited experience with fatigue test on cold recycled mixes, an analogical approach as in MEPDG seems a logic solution under these conditions. General formula is presented here which contains various calibration coefficients. Different European states can adapt this general formula in modified form used in their national pavement design by the selection of the values for these calibration (adjustment) coefficients.

However there is a difference in MEPDG approach and approach described here. It is supposed in USA that the basic models of the pavement response used in MEPDG will be accepted in all US states. These basic models are now calibrated to local conditions in various states in USA (according to the “Guide for the local calibration”).

The general formulae presented here can be adapted to different response models used in some European countries for the pavement design and then locally calibrated.

Thus the basic formulae for the evaluation of the fatigue resistance of the cold recycled layer of in the pavement design method could be written as follows.

$$N_f = C\,\beta_{1p}\;\beta_{1t}\,k_1\left(\frac{1}{\varepsilon_t}\right)^{\beta_{2p}\,\beta_{2t}\,k_2}\;\left(\frac{1}{E}\right)^{\beta_{3t}}$$

where

C laboratory to field adjustment factor (taking in account rest periods, traffic wander etc.),

β_{1p}, β_{2p}	adjustment factors to assure low probability of the crack appearance,
β_{1t}, β_{2t}, β_{3t}	temperature adjustment factors,
k_1, $k_{2\,l}$	laboratory fatigue parameters,
E	resilient modulus of the asphalt mix,
ε_t	tensile strain at the critical location.

If it is preferred to neglect the impact of the temperature or consider equivalent temperature as in the Czech and French pavement method (which is in our opinion the reasonable approach for the cold recycled mixes, especially for mixes with emulsion and cement), then the temperature adjustment factors β_t will be considered as $\beta_{3t} = 0$ and $\beta_{1t} = \beta_{2t} = 1.0$. Nevertheless it is recommended to decide about the final coefficients dependent on the calibration which should be done.

If it is preferred to respect the dispersion of the fatigue test only through the parallel shift of laboratory fatigue line (as in the Czech and French design method) β_{2p} can be taken as 1.0. Due to the uncertainties with the fatigue test, their dispersion and the big impact of the slope of the fatigue line on the N_f, it is recommended here to use the value $\beta_{2p} \leq 1$, if the fatigue parameters measured in the laboratory are considered for the pavement design. Nevertheless due to overall limited existence of fatigue data coefficients should be decided based on calibration. One of the problems recognized so far is that fatigue behaviour for bitumen stabilized mixes has fairly different pattern if compared to HMA. The development of E-modulus is during the loading entirely different and it is still unclear if same equation for “damage” status is applicable for cold recycled mixes as is used for HMAs.

The value of β_{1p} and β_{2p} could be expressed as a function of measured fatigue parameters similarly as confidence coefficient F related to ε_6 in the new Austrian method.

Coefficient k_1 can be shifted into the bracket and adapted in the form (K_1' . ε_6), if the form of the fatigue equation with ε_6 used in the Czech or French method is preferred. This nevertheless depends on a broader discussion and preferences of the road administrator. In general coefficient outside the bracket might be less influenced by other coefficients.

Thus the proposed basic formulae can be adapted to the different approaches used in some national design methods by the appropriate selection of values for adjustment coefficients β.

Coefficients and fatigue parameters can be selected in different countries according to their own experience with laboratory results and behaviour of realised pavements.

The basic formulae can be also expressed as a function of σ_t instead of ε_t for mixes with higher content of hydraulic binders. Naturally the different values of coefficients and fatigue parameters k_1, k_2 have to be used in such case.

The same annotations as in MEPDG are used here for the coefficients to visualise the analogy to MEPDG approach. The annotations used in Eurocodes (where various partial coefficients for limit state design have the annotation γ) can be used if the approach proposed here is accepted.

5.2 Design criteria for permanent deformation of cold recycled layer

In order to assess the permanent deformation of cold recycling materials, the results of monotonic triaxial tests can be applied according to experience by JENKINS [42]. By testing several stress states, the cohesion and friction angle can be obtained and limits for deviatoric stress considering the traffic loading may be developed. Further the stress-dependent stiffness of the cold recycled material then can be evaluated and included to the pavement design procedure. For the future, if triaxial tests are generally recognized as more suitable for this type of structural materials, it is necessary to further analyse and recommend if cyclic triaxial tests or Superpave Shear Test do not offer better and more suitable information about the resistance of the material to permanent deformations at occurring in the pavement.

Additionally the uniaxial cyclic text could be also consider as an alternative, since it is an easier test and require simpler equipment. Test temperature should also be selected for each country, taking in consideration each country climatic characteristics.

6 Acknowledgement

The research presented in this deliverable was carried out as part of the CEDR Transnational Road research Programme Call 2012. The funding for the research was provided by the national road administrations of Denmark, Finland, Germany, Ireland, Netherlands, Norway list funding countries

7 References

[1] Wirtgen GmbH: Wirtgen Cold recycling Technology [online]. Windhagen, Germany, 2012.

[2] Iwański, M., Chomicz-Kowalska, A.: Laboratory Study on Mechanical Parameters of Foamed Bitumen Mixtures in the Cold Recycling Technology, Procedia Engineering 57, p. 433-442, 2013.http://www.researchgate.net/publication/257727028_Laboratory_Study_on_Mechanical_Parameters_of_Foamed_Bitumen_Mixtures_in_the_Cold_Recycling_Technology

[3] Jian Xu, Songchang Huang, Yongchun Qin, Feng Li: The Impact of Cement Contents on the Properties of Asphalt Emulsion Stabilized Cold Recycling Mixtures, 2011.

[4] Alabaster, D., Patrick, J., Arampamoorthy, H., Gonzalez, A.: The design of stabilised pavements in New Zealand, Report, 2013.

[5] Gregg, A.: Development of tensile fatigue criteria for bound materials, NZTA research report 463, 2012.

[6] Lee, D.: Findings from Cold-in-Place Recycling Research on Optimum Foamed and Emulsified Asphalt Content and Curing Time, Workshop 118 In Place Pavement Recycling TRB, January 2013.

[7] Gonzalez, A., Cubrinovski, M., Pidwerbeski, B., Alabaster, D.: Performance of foamed bitumen pavements in accelerated testing facility, Revista Ingenieria de Construccion, Vol.27 No2, 2012.

[8] Vuong, B., Jameson, G.: Improved Design of Bituminous Stabilised Pavements, AP-T226-13, 2013.

[9] Jameson, G.: Design and Performance of Foamed Bitumen Stabilised Pavements: Progress Report One, AP-T247-13, 2013.

[10] Lynch, A., Jenkins, K.: Materials recycled using foamed bitumen stabilisation: What is their long term load spreading capacity, 15th Int. Flexible Pavements Conference of AAPA, 2013.

[11] Galal, K., Diefenderfer, B., Alam, J.: Determination by the Falling Weight Deflectometer of the Insitu Subgrade Resilient Modulus and Effective Structural Number for I-77 in Virginia, Final Report VTRC 07-R1, 2007.

[12] Davis, K., Timm, D.: Recalibration of the asphalt layer coefficient, NCAT report 09-03, 2013.

[13] Recycling and Reclamation of Asphalt Pavements Using In-Place Methods, NCHRP SYNTHESIS 421, 2011.

[14] Labuz, J., Tang, S., Cao, Y.: Structural Evaluation of Asphalt Pavements with Full-Depth Reclaimed Base, Report MN/RC 2012-36, 2012.

[15] Khosravifar, S., Schwartz, Ch., Goulias, D.: Mechanistic structural properties of foamed asphalt stabilised base materials, Journal of Pavement Engineering, 2014.

[16] Jones, D., Fu, P., Harvey, J., Halles, F.: Full-Depth Pavement Reclamation with Foamed Asphalt, Final Report 2008.

[17] NRA Interim Advice Note 01/11 on Low-Energy Pavements, March 2011.

[18] Pronk, A.: Equivalent layer theories – State of the art report. Rijkswaterstaat, Road and Hydraulic Engineering Division, Delft, 1994.

[19] Technical Guideline: Bitumen Stabilised Materials, TG 2Second edition, Asphalt Academy, May 2009.

[20] Menendez, J., Thenoux, G., Gonzalez, M.: Premature Cracking in Foamed Asphalt Pavement: Peru Brazil Highway Case Study, 2013.

[21] Browne, A.: Foamed Bitumen Stabilization in New Zealand – A Performance Review And Comparison With Australian And South African Design Philosophy, 25th ARRB Conference, 2012.

[22] Gonzales, A.: Review of Structural Design Procedures for Foamed Bitumen Pavements, AP-T188/11, 2011.

[23] Nantung, T., Ji Yigong, Shields T.: Pavement Structural Evaluation and Design of Full-Depth Reclamation (FDR) Pavement, TRB 2011 Annual Meeting, 2011.

[24] Schwartz, Ch.: NCRP 09-51 Update: Material Properties of Cold In-Place Recycled and Full-Depth Reclamation Asphalt Concrete for Pavement Design, Workshop 118 In Place Pavement Recycling TRB, 2013.

[25] Thompson, M., Garcia, L., Carpenter, S.: Cold In-place Recycling and Full-depth Recycling with Asphalt Products (CIR&FDRwAP), FHWA-ICT-09-036, 2009.

[26] Oke, O., Parry, T., Thom, N.: Fatigue Characteristics of Cold Recycled Bituminous Emulsion Mixtures Using the Nottingham Asphalt Tester in the ITFT Mode of Testing, 2014.

[27] Oke, O.: A study on the development of guidelines for the production of bitumen emulsion stabilised RAP for roads in the tropics, 2010.

[28] Fiedler, J., Mondschein, P., Jarušková, D., Žák, J.: Statistical evaluation of fatigue tests and their application for pavement design (in Czech), Part 1 to 4, Silnice mosty No 3/2013, 4/2013, 1/2014, 2/2014.

[29] Tijou, P.: Base de données fatigue, Seminaire LCPC, Formulation des enrobés du laboratoire au chantier, 2005.

[29] Blab, R., et al.: Vergleichende Beurteilung des Gebrauchsverhaltens von flexiblen Oberbaukonsturktionen mit Dünnschichtdecken in Kaltbauweise (Mikrobelag) und in konventioneller Deckenbauweise. Expert report 0615E, Technical University Vienna, 2006.

[31] Thanaya, I., Zoorob, S., Forth, J.: A laboratory study on cold-mix, cold-lay emulsion mixtures, 2007.

[32] Mollenhauer, K.: Dimensionierugsrelevante Prognose des Ermüdungsverhaltens von Asphalt mittels einaxial Zug-Schwellversuche, 2008.

[33] Timm, D., Priest, A.: Methodology and calibration of fatigue transfer functions for mechanistic-empirical flexible pavement design, 2006.

[34] LCPC: Guide technique “Retraitement en place à froid des anciennes chaussées”, 2003.

[35] LCPC: French design manual for pavement structures, LCPC/SETRA Ed., 250 pages, 1997.

[36] NF P 98-086 Dimensionnement des structures de chaussées routières, application aux chaussées neuves, 2011.

[37] Material Properties of Cold In-Place Recycled and Full-Depth Reclamation Asphalt Concrete for Pavement Design, NCHRP 09-51.

[38] Full Depth Reclamation Using Engineered Emulsion, Caltrans Division of Maintenance, 2013.

[39] Full Depth Reclamation Using foamed asphalt, Caltrans Division of Maintenance, June 2012.

[40] Heukelom, W.: Dynamic testing as a means of controlling pavements during and after construction, Shell Bitumen Monograph, 1953.

[41] FGSV: Richtlinie für die rechnerische Dimensionierung des Oberbaus von Verkehrsflächen mit Asphaltdeckschicht, (technical rules), Köln a.R., 2009.

[42] Jenkins K.J and Mulusa W.K.. Characterisation of Granular and Bitumen Stabilised Materials using Triaxial Testing. International Society for Soil Mechanics and Geotechnical Engineering ISSMGE. July 2011. Maputo, Mozambique

[43] FGSV: Richtlinie für die Standardisierung des Oberbaus von Verkehrsflächenbefestigungen RSTO, (technical rules), Köln, 2012.

[43] FGSV: Merkblatt für Kaltrecycling in situ im Straßenoberbau, (technical rules), Köln, 2005.

[44] Asphalt Institute: MS14 - Asphalt Cold Mix Manual. Asphalt Institute Manual Series No. 14. Third Edition. 1990, USA.

[45] Batista, F.A., Antunes, M.L.: Asphalt Cold Mixtures for Pavement Rehabilitation: Curing and Mechanical Characteristics. Proceedings of the 7th International Conference on the Bearing Capacity of Roads, Railways and Airfields BCRA'05, Trondheim, Norway, 27-29 June 2005, CD-ROM, Paper 160, 10p.

[46] Batista, F.A: Novas técnicas de reabilitação de pavimentos - Misturas betuminosas densas a frio (Innovative pavement rehabilitation techniques - Dense asphalt cold mixtures). PhD dissertation prepared under the cooperation between LNEC and FEUP, Oporto, Portugal, June 2004.

[47] Shell International Petroleum Company, Lda. – “Shell pavement design manual – Asphalt pavements and overlays for road traffic”. London, 1978.

[48] COST 333 - "COST 333 – Development of New Bituminous Pavement Design Method - Final Report of the Action". Transport Research, European Cooperation in the field of Scientific and Technical Research, European Commission, Directorate General Transport, 1999

[49] PG4 – Art. 20: Reciclado in situ con emulsión de capas bituminosas (Specifications for in situ cold recycling of bituminous layers using emulsion). Dirección General de Carreteras- Ministerio de Fomento, Spain, 2001.

[50] Norma 6.3-IC: Rehabilitación de firmes (Pavement rehabilitation), de la Instrucción de carreteras. Orden FOM/3459/2003, de 28 de noviembre de 2003.

[51] Soto, J., Peña, J.: Reciclado en Frio con Emulsion Assiciacion Tecnica de Emulsiones Bituminosas (ATEB).