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IlliTC – low-temperature cracking model for asphalt pavements

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Low-temperature cracking (LTC) is a major distress and cause of failure for asphalt pavements located in regions with cold climate; however, most pavement design methods do not directly address LTC. The thermal cracking model (TCModel) utilised by American Association of State Highway and Transportation Officials Mechanistic-Empirical Pavement Design Guide relies heavily on phenomenological Paris law for crack propagation. The TCModel predictions are primarily based on tensile strength of asphalt mixture and do not account for quasi-brittle behaviour of asphalt concrete. Furthermore, TCModel utilises a simplified one-dimensional viscoelastic solution for the determination of thermally induced stresses. This article describes a newly developed comprehensive software system for LTC prediction in asphalt pavements. The software system called 'IlliTC' utilises a user-friendly graphical interface with a stand-alone finite-element-based simulation programme. The system includes a preanalyser and data input generator module that develops a two-dimensional finite element (FE) pavement model for the user and which identifies critical events for thermal cracking using an efficient viscoelastic pavement stress simulation algorithm. Cooling events that are identified as critical are rigorously simulated using a viscoelastic FE analysis engine coupled with a fracture-energy-based cohesive zone fracture model. This article presents a comprehensive summary of the components of the IlliTC system. Model verifications, field calibration and preliminary validation results are also presented.

Keywords: asphalt; thermal cracking; fracture; performance; simulation; cohesive zone; transverse cracking; viscoelasticity; model; pavement; IDT; DC(T); IlliTC

1. Motivation and introduction

One of the main advantages of asphalt concrete over Portland cement concrete (PCC) is the smoothness and cost savings afforded by continuous paving, i.e. without the need for transverse joints. Unlike PCC and other infrastructure materials, asphalt concrete is generally able to undergo thermal cycling without the need for expansion or contraction joints due to its viscoelastic nature. Under imposed strain, which is constantly occurring in pavements due to temperature change, viscoelastic materials are able to relax stress over time. In addition, asphalt is generally a fracture-resistant material, owing to its flexible mastic matrix and particulate composite morphology. Significant energy is required to initiate and propagate a crack through asphalt concrete, as the asphalt mastic is tough, strain tolerant, and viscoelastic (stress relaxing), and the aggregates add strength, crack bridging, and crack surface tortuosity. However, improper selection of asphalt grade, excessive ageing of the asphalt binder, and/or a weak asphalt mixture (weak aggregates, low cohesion, and low adhesion) can all contribute to poor mixture fracture resistance. Poor mixture fracture resistance can lead to the development of thermal cracks, which are typically transversely oriented with traffic and periodic in nature.

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Thermal cracking is very serious pavement distress, as it can significantly increase pavement roughness and because it creates a permanent discontinuity in the pavement structure. In a recent study by Islam and Buttlar (2012), pavements allowed to reach a rough condition were shown to increase user costs (vehicle repair, tyre, and fuel cost) by over five million dollars per lane mile over a 35-year life cycle. In contrast, by investing just 1/50th of this cost in additional maintenance, the pavement could be kept in smooth condition over its lifespan, avoiding these additional user costs. However, in order to achieve this result, the pavement would need to be properly designed to avoid the development of medium- or high- severity thermal cracking, since an improper mixture design could lead to the development of thermal cracks prior to the application of the first maintenance or rehabilitation treatment.

Asphalt technologists have long recognised the need to control thermal cracking in asphalt pavements, and the tests and models available to assist in this endeavour have continuously evolved. Early efforts to control asphalt behaviour at low temperatures were focused on the asphalt binder. A comprehensive review of early binder tests in the USA has been documented by Brown et al. (2009). The penetration test, especially if run at two temperatures (generally 25°C and 4°C), provided some control over binder 'consistency' and 'temperature susceptibility'. The ductility test provided a simple measure of binder 'stretch' or strain tolerance. However, neither of these tests was applicable to temperatures below 0°C, where thermal cracking is likely to occur. In Europe, the Fraass breaking point test is used (as specified by EN 12593:2007), which is a torture-type binder test designed to determine the temperature at which a thin film of binder bonded to a small rectangular brass plate becomes intolerant to a bending strain arising from flexing the brass plate to a specified curvature. Although this test allowed direct mechanical testing of the binder at temperatures below 0°C, its direct relation to thermal cracking is questionable due to the very high strain level imposed. Superpave (American Association of State Highway and Transportation Officials (AASHTO) M320) addressed thermal cracking with modern binder tests run at low temperatures and in fundamental testing configurations. A bending beam rheometer characterised the stiffness and *m*-value (a measure of the ability of the binder to relax stress). An optional direct tension test was also specified, which was designed as a second referee test to address certain polymer-modified binders that possessed not only higher stiffness but also high strain tolerance and fracture resistance. However, the system was never intended to directly control thermal cracking; rather, it was developed as a binder purchase specification.

Although binder tests are convenient, practical, and important from the perspective of binder selection, purchase, and quality control, asphalt technologists have acknowledged the need to address thermal cracking more directly through testing of the asphalt mixture and modelling of the pavement structure. Canadian researchers developed limiting mixture stiffness recommendations based upon a comprehensive field investigation at Ste. Anne (Deme & Young, 1987). During the Strategic Highway Research Programme (SHRP), a mixture-based testing and analysis scheme were developed to validate the binder test and specification system being developed. A low-temperature mixture creep and strength test was developed, later called the Superpave indirect tension test (the acronym 'IDT' was developed in early Superpave publications, and is still commonly used), as specified in AASHTO T-322. Master creep compliance curves, shift factors, tensile strength, and optionally, mixture coefficient of thermal expansion and contraction (CTEC) at low temperatures can be obtained with the Superpave IDT. SHRP researchers, working under project A-005 also developed a computer-based thermal cracking model, called TCModel, as part of the effort to validate the Superpave binder specification. The first version of TCModel was completed in 1992, near the end of the SHRP programme.

TCModel made great strides in modelling some of the key physics underlying the thermal cracking mechanism. In particular, TCModel had a highly sophisticated viscoelastic pavement response model, which predicted pavement tensile stress versus depth on an hourly basis throughout the life of the pavement based upon principles of linear viscoelasticity. Due to limitations in computational power, a phenomenological pavement cracking model was used for distress prediction. Change in crack length was predicted using a power-law-type model reported by Paris, Gomez, and Anderson (1961), which uses change in stress intensity (which was calculated in an approximate manner by interpolating presolved two-dimensional (2D) elastic finite element (FE) runs) to predict change in crack length. In TCModel, Paris law parameters were empirically linked to IDT strength and to the slope of the log mixture compliance versus log time relationship at long loading times (mixture *m*-value). TCModel was selected for inclusion in the *AASHTO Mechanistic-Empirical Pavement Design Guide* (*MEPDG*), and was improved and streamlined as part of the National Cooperative Highway Research Program 1-37A project. For instance, an automated mixture master curve generation programme was bundled with TCModel. In addition, additional field data, including that obtained from MnROAD test sections, were used to recalibrate TCModel.

With changes in asphalt binder and mixture designs and materials over the past 20 years, such as the increased use of polymers and other additives and the increased use of recycled asphalt pavement, it became apparent that the heavy reliance on mixture tensile strength in cracking predictions was limiting the prediction accuracy of TCModel. New mixture fracture tests were developed to address these new materials, including the disk-shaped compact tension test, or DC(T), which provided a convenient means to obtain mixture fracture energy using a fracture mechanics-based approach. Recognising the shortcomings of TCModel in light of modern mixture fracture tests and computational power, a new TCModel was developed at the University of Illinois Urbana-Champaign, called ‘IlliTC’, as part of a national Pooled Fund Study on low-temperature cracking (LTC). IlliTC improves the manner in which fracture is handled in the simulation scheme, namely, the one-dimensional (1D) Paris-law phenomenological modelling approach was replaced with a 2D, cohesive zone fracture modelling approach implemented within a viscoelastic FE modelling framework. The cohesive zone approach considers both material strength and fracture energy in computing crack initiation and propagation using fundamental fracture mechanics principles. In summary, the new approach used in IlliTC has the following improvements over TCModel:

- A 2D model is used instead of 1D.
- The physics of cracking in a quasi-brittle, heterogeneous particulate composite is more correctly captured by using a cohesive zone approach, where softening and fracture have a distinct length scale that is captured.
- Asphalt mixtures may have unique combinations of strength and ‘ductility’ (as characterised by mixture fracture energy). For instance, some polymer-modified mixes portray moderate tensile strength and high fracture energy; some have high strength and lower fracture energy, and some have both high strength and high fracture energy. Mixtures with higher recycled material content may have high strength, but low fracture energy. IlliTC can capture all of these combinations in a direct manner, while TCModel could only capture these effects in an indirect manner.
- A user-friendly graphical interface (GUI) has been provided for IlliTC. The GUI programme module within IlliTC is referred to herein as Visual LTC.

This article presents the IlliTCmodel components, model verification, model calibration, and preliminary validation results. Ongoing research, aimed at adding additional software capabilities and modelling features to the IlliTC programme are also described.

2. IlliTC framework

The software programme (IlliTC) provides an intuitive and user friendly GUI, as a convenient gateway to the rigorous viscoelastic FE/cohesive zone modelling engine. The overall flow of the IlliTC programme along with various inputs and outputs are graphically illustrated in Figure 1. The code consists of a GUI, which is represented as a red box in Figure 1, and four analysis modules, which are represented by blue boxes in Figure 1. The analysis modules include a preprocessor, an input file generator, a preanalyser, and a FE-based thermal cracking prediction engine. An overview of the GUI and analysis modules is presented in the remainder of this section and further implementation details are presented in Sections 3–5.

To initiate an IlliTC thermal cracking simulation, the user is queried to enter information pertaining to the project location, design life, pavement structure, and material properties into the GUI. The GUI handles cumbersome tasks such as data organisation and unit conversion; then automatically assembles additional data needed for the analysis from internal databases.

The GUI passes the raw creep compliance data to the preprocessor, which returns the thermo-viscoelastic material properties in the form of Prony series parameters (generalised Maxwell model) and time–temperature shift factors. The preprocessor consists of two modules, the first one is based on the code 'Master' developed by Buttlar, Roque, and Reid (1995) and second one is based on the TCModel (Lytton et al., 1993; Roque, Hiltunen, & Buttlar, 1995a; Roque, Hiltunen, Buttlar, & Farwana, 1995b). The first module generates the creep compliance master curve from the raw creep compliance data and fits a generalised Voigt–Kelvin model to the master curve. The Voigt–Kelvin model for creep compliance is then converted to the generalised Maxwell model in the form of relaxation modulus.

The GUI reads the Maxwell model parameters from the preprocessor output, organises it along with other data, and passes it to the input file generator. The input file generator conducts two main tasks: (1) to develop a FE mesh for the pavement geometry specified by the user, called the geometric data file (*.mesh) shown as a green box in Figure 1 and (2) to create a material data file, based on information provided by the user, called the material data file (*.mtr), shown as a green box in Figure 1.

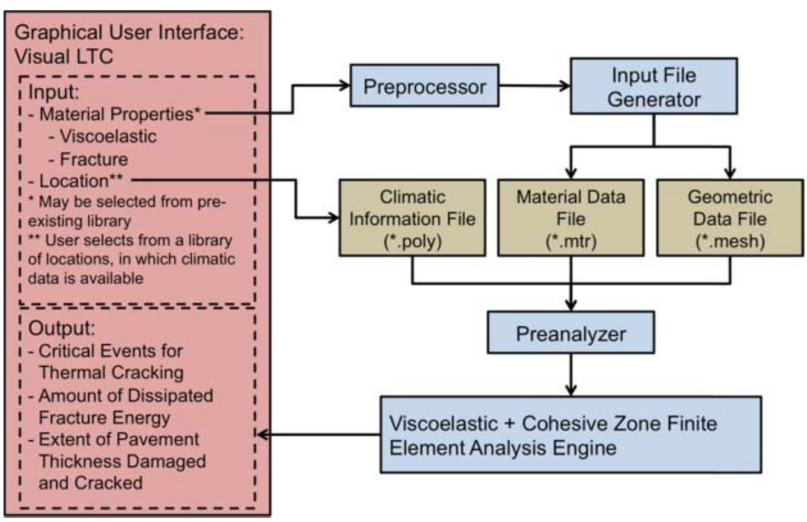

Figure 1. Flowchart of new TCModel – IlliTC.

The GUI collects the climatic data necessary to perform the analysis in the chosen location for the specified duration from internal databases. These data are stored in the climatic information file (*.poly), shown as a green box in Figure 1. Then the GUI passes all input files to the preanalyser.

The preanalysis module (or 'preanalyser') serves to minimise the analysis time of the more rigorous FE engine. Typically, LTC analysis is focused around critical cooling events. Hence, critical cooling events are identified by the preanalyser so that the full model can be focused on critical cooling events, typically reducing computational time by more than 90% (i.e. in a 365 day year, less than 36 days would typically need to be simulated). The GUI reads the results of the preanalysis module and extracts the data pertinent to the critical cooling events. These data are then passed to the FE model.

The user is not required to have direct interaction with the viscoelastic and cohesive zone FE simulation; instead, results are sent back to the GUI, which are interpreted for the user.

3. Graphical user interface

The GUI, called 'Visual LTC', collects and compiles the input conditions provided by the user, executes various analysis modules to conduct finite-element analysis (FEA), and interprets and displays the results. Visual LTC was written with the object-oriented programming language C# (pronounced 'see-sharp') under Microsoft's .NET framework, and which is intended for the development of deployable software. Visual LTC was designed to be intuitive to use by practitioners or researchers. A series of windows are used to query the user for required model inputs. For example, the window that collects the asphalt layer material properties is shown in Figure 2. User inputs and options shown in Figure 2 are discussed in more detail in subsequent sections. For further details on conducting analysis using Visual LTC, the reader is referred to Dave, Leon, and Park (2011).

3.1. *Communication with analysis modules*

Data are passed between Visual LTC and the analysis modules via input/output files. Visual LTC reads the user input then performs the series of converting data, writing files, executing programmes, and reading output given in Table 1.

3.2. *User types*

Visual LTC is intended for use by practitioners and researchers alike. Therefore, two user types are supported: 'Standard User', and 'Advanced User'. Both users have access to all functionality previously described. However, Advanced Users have the additional capability of adding new asphalt mixes and modifying properties of existing asphalt mixes. The distinction between these user types is present so that existing properties are protected from inadvertent user error. The user can easily change from one user type to the other.

3.3. *Visual LTC user inputs*

The main user inputs that are required for LTC analysis are the analysis location, analysis duration, and the pavement material properties. A series of integrated climatic model (ICM) simulations were conducted to create a library of pavement temperature profiles available to the user in Visual LTC. Sets of temperature profiles were generated for one cold, one intermediate and one warm location in each participating state of the pooled fund LTC study, as given in Table 2. Eventually, IlliTC will be coupled with the ICM model, so that climatic data from thousands of geographic are available within the model. The data required to generate these libraries are the same as those

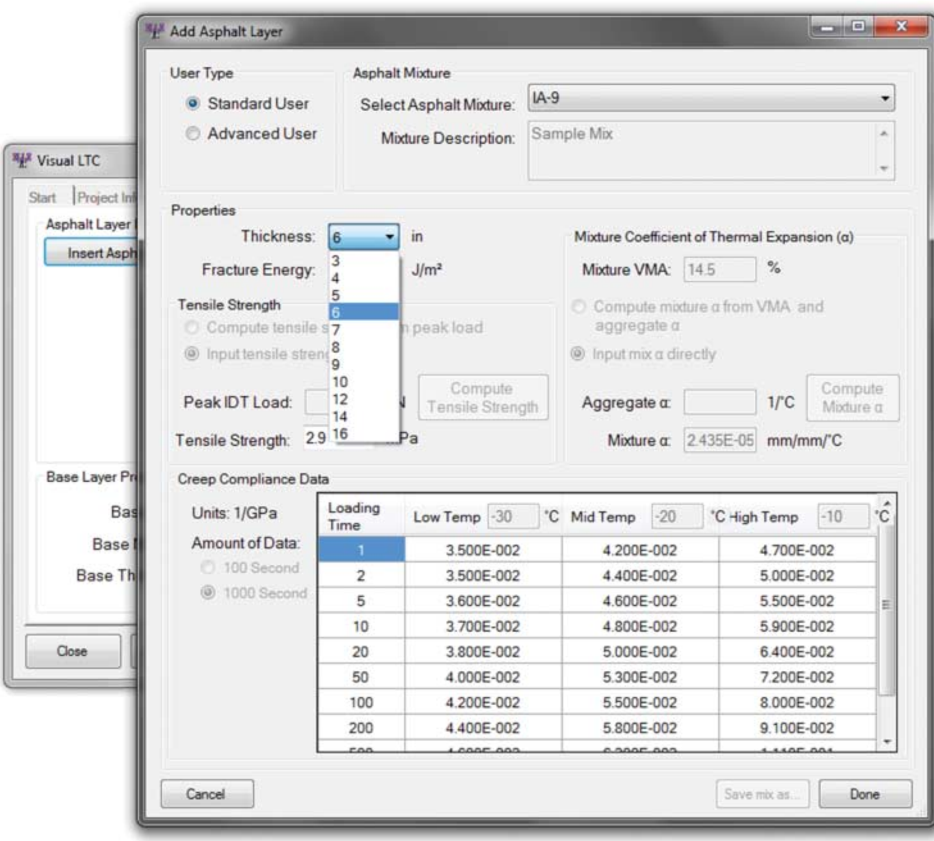

Figure 2. Visual LTC window – asphalt layer material properties.

Table 1. Visual LTC steps.

(1) Read and store user input
(2) Extract and store climatic information for user specified analysis period
(3) Write input files for preprocessor
(4) Run preprocessor
(5) Read and convert preprocessor output
(6) Write input files for input file generator
(7) Run input file generator
(8) Run preanalyser
(9) Process preanalyser output to identify critical events and generate FE temperature input
(10) Run FEA engine
(11) Read FEA output
(12) Convert crack depth to amount of cracking
(13) Display results

used in the AASHTO *MEPDG* system. Temperature profiles at each location were generated for the following asphalt concrete thicknesses: 75, 100, 125, 150, 175, 200, 225, 250, 300, 350, and 400 mm.

In Visual LTC, the user selects a location that is the most climatically similar to the analysis location. The user also provides the pavement cross-section. Visual LTC extracts the appropriate

Table 2. Climatic locations available to user in Visual LTC.

State	Climate	City	Air temperature	Superpave PG grade
Connecticut	Cold	Norfolk	−29.5°C	−28°C
	Intermediate	Hartford	−26°C	−22°C
	Warm	New Haven	−20.5°C	−22°C
Illinois	Cold	Elizabeth	−37°C	−34°C
	Intermediate	Urbana	−31.5°C	−28°C
	Warm	Anna	−27°C	−22°C
Iowa	Cold	Decorah	−40.5°C	−34°C
	Intermediate	Des Moines	−32°C	−28°C
	Warm	Fort Madison	−30.5°C	−28°C
New York	Cold	Massena	−39°C	−34°C
	Intermediate	Albany	−33.5°C	−28°C
	Warm	New York	−19.5°C	−16°C
North Dakota	Cold	Westhrope	−44°C	−40°C
	Intermediate	Bismarck	−41.5°C	−40°C
	Warm	Wahpeton	−38°C	−34°C
Minnesota	Cold	International Falls	−43.5°C	−40°C
	Intermediate	St Cloud	−41.5°C	−34°C
	Warm	Worthington	−34.5°C	−34°C
Wisconsin	Cold	Minong	−46°C	−40°C
	Intermediate	Steven's Point	−36.5°C	−34°C
	Warm	Milwaukee	−32°C	−28°C

Table 3. Summary of pavement material user inputs for IlliTC.

Property		Units	Test
Tensile strength		MPa	AASHTO T-322 or extracted from DCT test (Buttlar, Sahu, Behnia, & Dave, 2013)
Fracture energy		J/m^2	ASTM D7313[a]
Option – 1	Unit weight	g/cm^3	AASHTO M323
	Mixture VMA[b]	%	AASHTO M323
	Aggregate CTEC[b]	mm/mm/°C	No standardised test
Option – 2	Mixture CTEC[c]	mm/mm/°C	No standardised test
Creep compliance test data (100 or 1000 seconds for three temperatures)		1/GPa	AASHTO T-322
Creep compliance test temperatures		°C	AASHTO T-322

[a]Fracture energy may be obtained with different test geometries; however, the model is calibrated for the ASTM D7313 (DCT) test procedure.
[b]Mixture voids in mineral aggregate (VMA) and aggregate CTEC do not need to be entered if mixture CTEC is provided.
[c]Mixture CTEC will be calculated if mixture VMA and aggregate CTEC are provided.

data from the temperature profiles associated with the location and pavement cross-section. These data are passed to the preanalyser and FE engine where nodal temperatures are computed. The pavement material property-related user inputs are summarised in Table 3 and discussed below.

Tensile strength of asphalt concrete can be determined using the Superpave IDT, as specified in AASHTO T-322. Recently, a procedure was developed to extract mixture tensile strength from DCT test data, which will be incorporated into IlliTC in the near future. The fracture energy can be determined using a variety of test geometries, such as DCT, semi-circular bend and single-edge notched beam test, although the model has been calibrated and validated based on the DCT test geometry. Furthermore, the fracture test is expected to be performed at crack mouth opening displacement rate of .0167 mm/s and at temperature of 10°C above the 98% reliability Superpave performance grade (PG) low-temperature grade, as dictated by the project location.

The user can either directly input the CTEC or provide asphalt mixture volumetric properties. If volumetric properties are provided, the CTEC is estimated using the approximation equation utilised by the AASHTO *MEPDG* software. The researchers at the University of Wisconsin have proposed experimental procedures to measure the CTEC of asphalt mixtures (Marasteanu et al., 2007; Nam & Bahia, 2004, 2009); the use of their procedure is recommended for added accuracy in prediction. The work by the same research group has shown a bilinear trend in the volumetric changes that occur in asphalt binder and mixes, with significantly different CTEC values above and below glass transition temperature. This feature is not currently implemented in the IlliTC system; however, it is recognised as one of the tasks for implementation in future versions.

The user can directly enter laboratory measured 100 or 1000 second creep test data from three temperatures following the AASHTO T-322 test procedure. These data are passed to the preprocessor, which converts the data into thermo-viscoelastic material properties in form of Prony series parameters (generalised Maxwell model) and time-temperature shift factors.

3.4. *Data storage*

A simple and intuitive class structure is employed to store and maintain data required for LTC analysis, i.e. material properties, climatic data, pavement structure, and project information. The data should be easily accessible by the user and should not require installation of additional software. A working directory containing input files stores all of the data necessary for Visual LTC to conduct analysis. Furthermore, the user is not required to directly access the files, as Visual LTC creates and modifies files automatically. The project input file stores general information (i.e. project name, description, date, etc.), climatic information, and the pavement structure. Asphalt concrete input files store all material properties associated with the mix. A working directory can contain many project files, thus giving the user the option of creating a new project by modifying an existing one. Similarly, the working directory can contain as many asphalt concrete input files as necessary, which creates a library of mix designs for the analyst or designer to investigate.

4. Input generator and preanalyser

The IlliTC prediction system is designed to be practitioner friendly and hence all the necessary inputs for the FE simulation are generated by the software. The input generator module handles the creation of FE mesh and corresponding data file with all geometric information and also generates a material data file in the necessary format for the FEA code. The first task of this module is to develop a FE mesh for the pavement geometry selected by user. The details on the mesh generation were previously discussed by Dave, Paulino, and Buttlar (2012). Briefly, the code generates a FE mesh using four node quadrilateral elements (Q4) and it automatically generates a transition mesh with element sizes increasing as the distance from potential thermal crack increases. A single line of cohesive zone elements are inserted into the mesh. These elements allow for the simulation of a single thermally induced crack that can span across the pavement thickness. The crack is simulated in the transverse direction. The input generator takes the material data provided by the user as well as the viscoelastic parameters determined by preprocessor and generates the material data file (.mtr).

The preanalysis module (or preanalyser) was developed to optimise analysis times of the FE engine. A simplified 1D problem is solved by this module to identify critical events that are

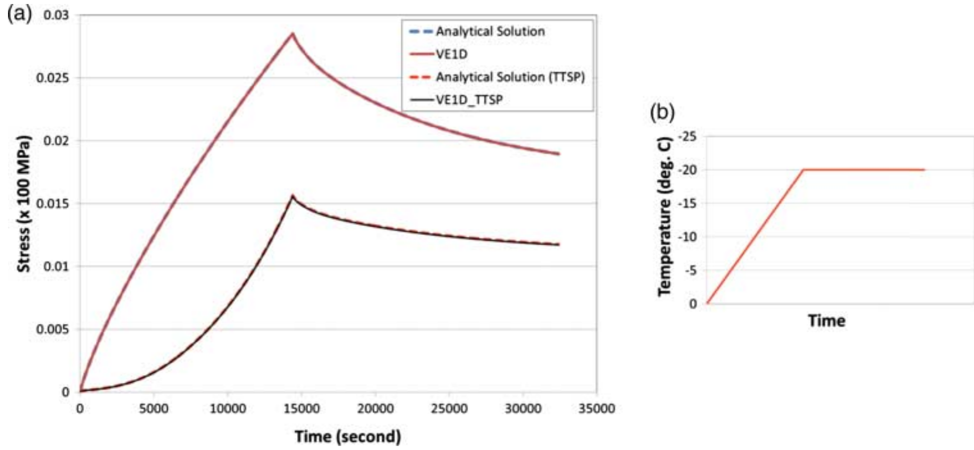

Figure 3. Verification of preanalyser. (a) Comparison of stresses obtained with the preanalyser (VE1D) and the analytical solution and (b) thermal loading.

then analysed by the FEA engine; hence, only critical cooling events are analysed with the full model. The preanalyser module solves the stresses on a restrained 1D viscoelastic body that is imposed with temperature boundary conditions representative of the temperatures at the pavement surface. The body is assumed to have same properties as the thermo-viscoelastic properties of asphalt concrete. The 1D viscoelastic solution for thermal stress can be found in Apeagyei, Dave, and Buttlar (2008); this solution is implemented in IlliTC using a recursive-incremental numerical integration method.

The results from the preanalyser were verified with the analytical solution. The stresses obtained with the preanlayser (Viscoelastic 1-D Analyser, VE1D) and the analytical solution for a 1D body imposed with thermal loading shown in Figure 3(b) are compared in Figure 3(a). The results show excellent agreement between the preanalyser results and analytical solution.

The critical cracking events are identified when the thermal stress in the 1D model exceeds 80% of the tensile strength of the asphalt mix. The thermal stress from 1D analysis is assumed to represent the stresses on the pavement surface. The threshold of 80% was selected based on previous experience of researchers in determining the stress threshold corresponding to onset of damage. The full analysis with the FE engine is performed for the 24 h surrounding the critical event.

Figure 4 shows an example of the results of the preanalyser. The winter time surface temperatures during the five-year analysis period are shown in Figure 4(a) and the resulting thermal stresses are shown in Figure 4(b). Only the time duration between October 1 and March 31 is simulated, due to significantly greater potential for thermal cracking during these months. Four critical events were identified by this analysis; the full FE simulation will be performed on these events accordingly.

5. FEA engine

The FEA method is used extensively in everyday design and analysis of civil infrastructure. The biggest strengths of FEA are its ability to simulate challenging geometries, such as pavements, and the relative ease in which it is possible to incorporate complicated material behaviour. For situations involving these characteristics, analytical solutions would be challenging and often unfeasible. The IlliTC system utilises an FE code that was developed in-house for the simulation

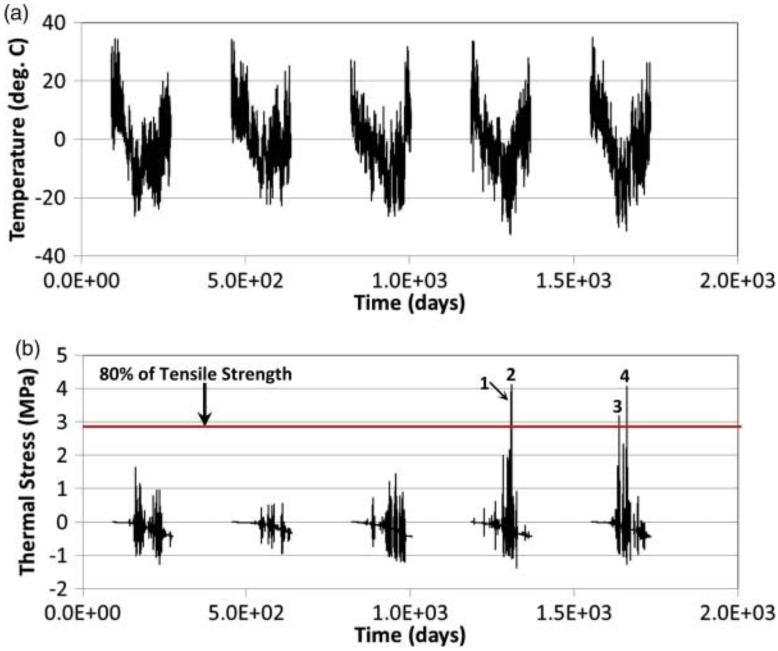

Figure 4. Results from preanalyser. (a) Winter time surface temperatures for five-year analysis period and (b) resulting thermal stresses where four critical events were identified.

of thermo-viscoelastic problems. It has the capability of simulating cracking in asphalt concrete through the use of a powerful yet computationally efficient cohesive zone fracture model. Traditional modelling approaches have not provided a direct means for the study of crack initiation and propagation in asphalt materials. The cohesive zone fracture approach provides a rational means for modelling cracking in quasi-brittle materials such as asphalt concrete, as the length scale associated with the fracturing process is accounted for. The following subsections briefly describe the cohesive zone model (CZM) and the thermo-viscoelastic implementation in IlliTC's FEA engine. This is followed by selected verification examples and brief description of the post-processing methodology to extract results from the FEA.

5.1. *Cohesive zone fracture model*

In order to correctly replicate the complex mechanisms underlying cracking in asphalt concrete, a standard 'strength of materials' type analysis is not sufficient, due to: (1) the highly non-linear behaviour in the vicinity of the crack tip and (2) the importance of the crack in the overall structural response (i.e. the need to model crack as a moving boundary value problem). For simulation of crack initiation and propagation, a CZM was selected because of its accuracy and efficiency in accounting for material response ahead of the crack tip in the fracture process zone (region of micro-cracking, crack pining, crack branching, material softening, etc.). Several researchers have used this type of approach for the simulation of cracking in asphalt materials, for example, Soares, Colares de Freitas, and Allen (2004), Song, Paulino, and Buttlar (2006), Dave and Buttlar (2010), Baek, Ozer, Wang, and Al-Qadi (2010), and Kim, Aragao, Allen, and Little (2010).

The CZM provides the relationship between the displacement jump or the opening along the crack path and the total capacity of material to transfer traction (or load) across that crack path. In the case of brittle materials, the capacity to carry load across the crack path is either 100%

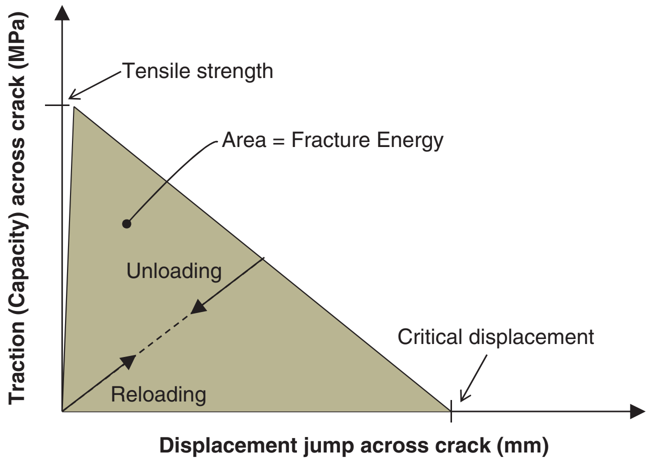

Figure 5. Bi-linear CZM used in IlliTC.

(uncracked) or 0% (cracked). The threshold of stress at which this capacity goes from 100% to 0% is the material strength and in case of mode I (opening) type of cracking, it is tensile strength. For quasi-brittle materials, the capacity to transfer load across the crack path also reduces once the material has incurred damage. However, unlike brittle materials, for quasi-brittle materials this capacity reduces gradually. The stress threshold at which the damage begins to occur is still tensile strength. Finally, when the crack separation (displacement jump) is greater than a critical value, the material no longer has bearing capacity and the traction is zero. The relationship between displacement jump and traction capacity across crack can be defined by different geometric shapes. Various shapes have been proposed for use with asphalt concrete, such as bilinear (Song et al., 2006), power law (Song, Wagoner, Paulino, & Buttlar, 2008), and exponential (Dave & Buttlar, 2010).

In this study, the bi-linear CZM described by Song et al. (2006) is being employed; the graphical representation of the model is shown in Figure 5. Note the unloading and reloading behaviour of the model is also shown in the figure. This behaviour assumes that the damage incurred by the material is permanent and is present when it is reloaded. The area under this traction and displacement jump curve is the fracture energy of the material. Thus, the CZM implementation allows for fracture representation of asphalt concrete through the use of two material properties, namely, tensile strength and fracture energy. An intrinsic cohesive zone modelling approach is used in this work; hence, a penalty stiffness (i.e. initial ascending slope) is introduced in the computational implementation. The initial penalty stiffness is determined on the basis of the numerical stability associated with the FE implementation (Roseler, Paulino, Park, & Gaedicke, 2007). The implementation of CZM in IlliTC is limited to mode I cracking. Since the thermally induced stresses in conventional asphalt pavements are along the longitudinal direction, the predominant failure occurs through tensile failure in mode I. Very small quantities of shear stresses are generated due to thermal only loading case and thus mode II or mixed mode cracking is not expected.

5.2. *FE implementation and verification*

The thermo-viscoelastic FE code was implemented in the C programming language. The code is based on the incremental-recursive formulation proposed by Yi and Hilton (1994) and Zocher, Groves, and Allen (1997). The details on the implementation and formulation have been presented by Dave et al. (2012). Due to the non-linear nature of the CZM, the FEA was implemented using

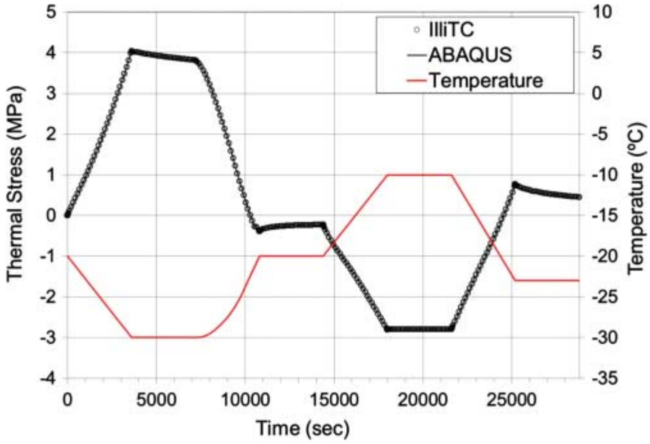

Figure 6. Comparisons for thermo-viscoelastic stress predictions using IlliTC and commercial FE programme ABAQUS.

a modified Newton–Raphson solution scheme. A simple adaptive time increment scheme was utilised that automatically increases the time increment as long as the convergence error is below the threshold.

The FE implementation was verified using analytical solutions and commercial programmes. Two verification examples are presented herein. The first example involves the simulation of a boundary value problem which resembles the thermal stress-retrained specimen test. The simulation was conducted to include both cooling and warming temperature boundary conditions to ensure the rigour of IlliTC (some CZMs have convergence problems upon crack closure). Figure 6 presents the thermal stresses predicted by IlliTC against those predicted by the commercial FE programme ABAQUS. The figure also shows the temperature boundary condition that was used in this simulation. The results show very good agreement between IlliTC and ABAQUS results, providing verification for the thermo-viscoelastic stress prediction capabilities of IlliTC.

Thermo-viscoelastic predictions coupled with CZ fracture simulation were also verified, as shown in Figure 7. The verification was conducted on a solid rectangular body as shown on the plot, where a temperature drop of 20°C over duration of 150 min was imposed. Once again, very good agreement was observed between the IlliTC and ABAQUS predictions. Note that there were minor deviations in the predicted stresses, especially during the damage and cracking process (post-peak). For this verification example, the stress variations between two programmes ranged between 1% and 2%. This level of discrepancy is not unexpected, due to differences in the implementation of the CZM and the use of different numerical solvers in the two FE engines.

5.3. *Post processing*

The output generated by the FEA contains displacements at nodal points and stress and strain predictions as element integration points. For the average user, the results require post-processing to generate a simple thermal cracking versus time output. The IlliTC screens the outputs generated by the FEA and produces a series of text files that contain information regarding crack opening widths, which is the deformation experienced by CZM elements in the longitudinal direction. By comparing the deformation of cohesive zone elements against the critical displacement in the

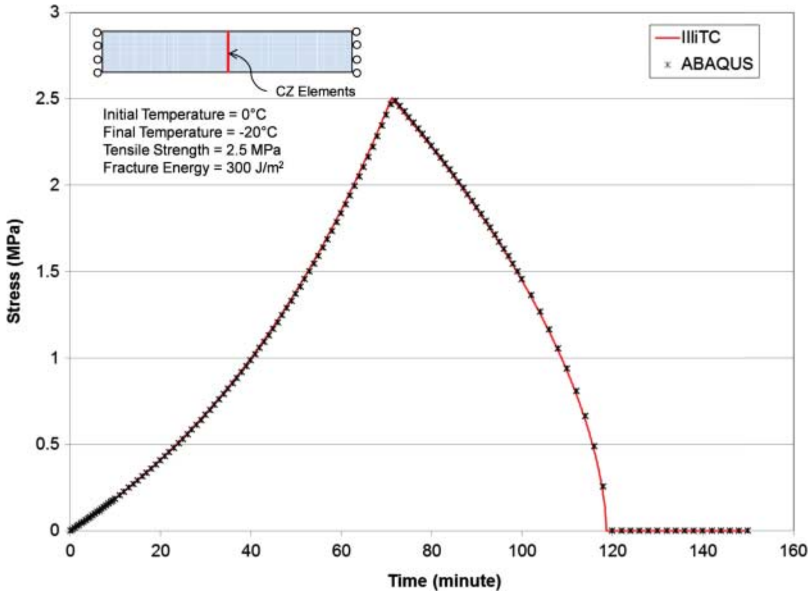

Figure 7. Comparisons of stress for a thermo-viscoelastic body with CZ fracture for predictions using IlliTC and commercial programme ABAQUS.

CZM, the location of the crack tip and location of the point beyond which the asphalt has started to soften are computed. For example, in a pavement with asphalt concrete thickness of 150 mm, if the cohesive zone elements in the top 25 mm of the pavement have experienced deformation in excess of the critical deformation, the thermal crack has formed to a depth of 25 mm. Furthermore, if the cohesive zone elements along the upper 75 mm of the asphalt layer have undergone deformations exceeding the deformation corresponding to the tensile strength in the local CZM (cf. Figure 5), the upper 75 mm of asphalt concrete has undergone softening damage. In the case of this example, the upper 25 mm of the pavement is fully cracked, while the pavement between a depth of 25 and 75 mm has undergone softening damage. In the later section on field validation, an example is provided containing more details on this topic. In addition, the field calibration section describes how the post-processed information is used to predict the extent of thermal cracking using a probabilistic crack distribution model.

6. Probabilistic crack distribution model

To avoid the modelling complexities and computation expense needed to simulate multiple thermal cracks, the scheme used in the original TCModel to translate a single thermal crack depth prediction to thermal crack density (spacing) was adopted in the current version of IlliTC (ILLITC v1.0). The modelling of multiple thermal cracks, while more exact, was not deemed as being worth the added computational expense, since the point at which crack interaction occurs is well within the range of severe cracking. Since it is unlikely that a designer would use a high cracking level as a design target, it was decided that multiple cracks would not be considered in this version of IlliTC. Rather, the model completes its execution once a high level of cracking reached (200 m of transverse cracking per 500 m of pavement, which corresponds to a crack spacing of 10 m).

This probabilistic crack distribution model converts the computed crack depth of a single modelled crack (viewed as a representative thermal crack, having a crack depth representing an average crack depth) to an amount of thermal cracking (crack frequency) with the following expression:

$$C_\mathrm{f} = \beta_1 \times P_R(\log C > \log h_\mathrm{ac}), \tag{1}$$

$$C_\mathrm{f} = \beta_1 \times N\left(\frac{\log C/h_\mathrm{ac}}{\sigma}\right), \tag{2}$$

where C_f is the predicted amount of thermal cracking (m/500 m) at a given simulation time, β_1 the multiplier representing maximum thermal cracking level, $N(x)$ the standard normal distribution evaluated at x, σ the standard deviation of the log of the depth of cracks in the pavement, C the depth of crack predicted by IlliTC at a given simulation time and h_ac the thickness of asphalt layer being simulated (generally taken as the thickness of all asphalt layers).

Crack amount (m/500 m) can be converted to thermal crack spacing by dividing predicted crack amount, C_f, by lane width (typically assumed to be 4 m), and taking the inverse of this quotient and multiplying by the unit section length (500 m). Citing the example provided earlier in this section, a crack amount of 200 m corresponds to: (1/(200/4))*500 or 10 m. This corresponds to 1000/10 or 100 full-lane-width cracks per km, which corresponds to approximately 161 thermal cracks per mile. This corresponds to the maximum thermal cracking level predicted by IlliTC in the current version. Note, similar to the approach taken in the development of TCModel, the parameters β_1 and σ were taken as model calibration parameters. Since thermal cracks are difficult to detect until they propagate completely through the pavement, it would be extremely difficult to directly measure and assess the σ parameter. Thus, its selection as a model calibration parameter is a practical means to circumvent the need to directly measure σ.

7. Field calibration

The MnROAD full-scale pavement test sections were used to calibrate IlliTC, namely, sections 03, 19, 33, and 34. Details about these sections can be found elsewhere (Marasteanu et al., 2007). A decision needed to be made with regards to the climatic files used in model calibration, since two approaches were possible: (1) use the actual time ranges corresponding to the field thermal cracking data for each section simulated or (2) use the climatic files available in IlliTC. The argument for using the actual time ranges that correspond with the field data is that predicted critical cooling events would match actual critical events in the crack history data files, leading to more accurate thermal cracking predictions for model calibration. The argument for using the climatic files included in IlliTC is that future pavement simulations conducted using IlliTC would be expected to utilise these climatic files (unless the user takes the effort to modify IlliTC to utilise alternate climatic files, which is a cumbersome process in the current version of the software). Considering that most users will likely utilise the climatic files provided in IlliTC, and also considering that the model should be re-calibrated to local conditions rather than rely on the calibration provided herein, it was decided to conduct model calibration using the climatic files provided in the current version of IlliTC.

The subsequent subsections present the results from preanalyser and the FEA engine for each of the calibration section. This is followed by the brief description on actual calibration of parameters in the probabilistic crack distribution model.

7.1. *Preanalyser runs*

Selected outputs from IlliTC's preanalyser are provided in Figure 8(a)–(e), and summarised in Table 4. By comparing Figure 8(a) with 8(b)–(e), it is clear that the days with the coldest

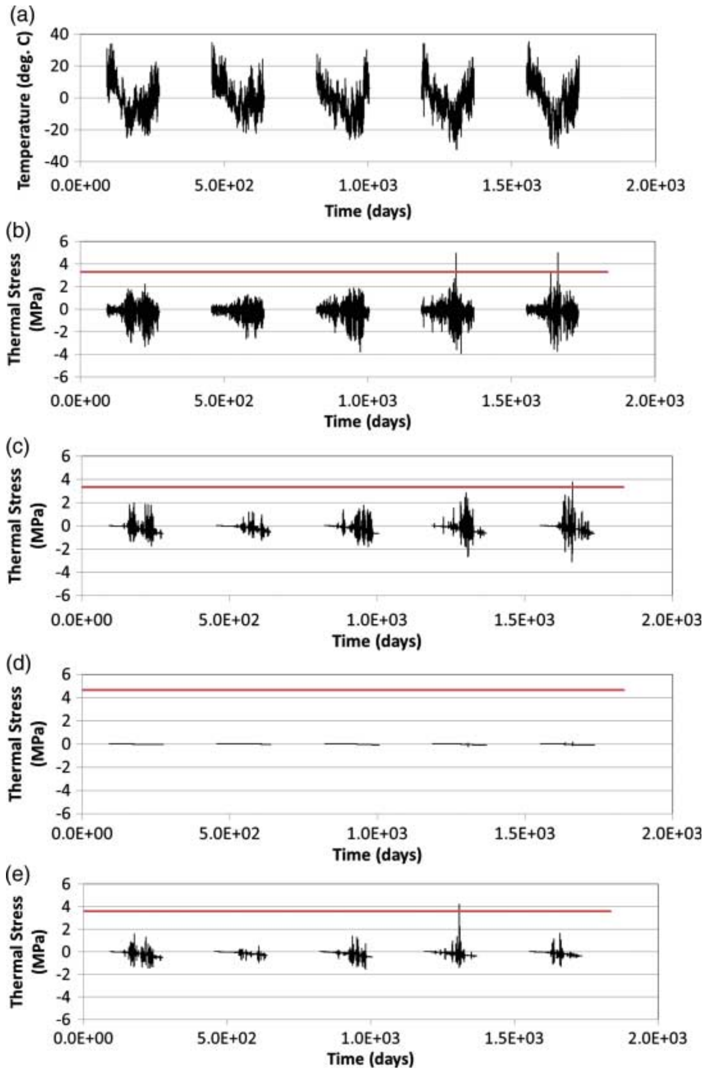

Figure 8. Results from preanalyser for calibration field sections. (a) Pavement surface temperature using default climatic files in IlliTC for MnROAD site (in the category of moderate climate within the state of Minnesota), (b) thermal stress on pavement surface for MnROAD03 from preanalyser (red line indicated 80% of tensile strength), (c) thermal stress on pavement surface for MnROAD19 from preanalyser (red line indicated 80% of tensile strength), (d) thermal stress on pavement surface for MnROAD33 from preanalyser (red line indicated 80% of tensile strength), and (e) thermal stress on pavement surface for MnROAD34 from preanalyser (red line indicated 80% of tensile strength).

temperatures correspond to the events with the highest surface tensile stress. Table 4 gives that 1 critical cooling event was computed for MnROAD section 03 during the simulated five-year analysis period, while 4, 1, and 0 critical cooling events were predicted for sections 19, 33, and 34, respectively. Comparing the number of computed critical cooling events with field cracking behaviour indicates the correlation between mixture viscoelastic behaviour (as captured by the creep compliance master curves) and cracking behaviour. The correlation between mix creep compliance and fracture behaviour was also demonstrated via a statistical analysis in a previous phase of this study (Marasteanu et al., 2007).

Table 4. Preanalyser results (number of critical events) compared with field cracking.

MnROAD cell	Number of critical events (as predicted by preanalyser)	Binder grade	Field cracking (m/500 m)
03	1	PEN 120/150 (PG 58-28)	182
19	4	AC20 (PG 64-22)	547
33	1	PG 58-28	91
34	0	PG 58-34	6

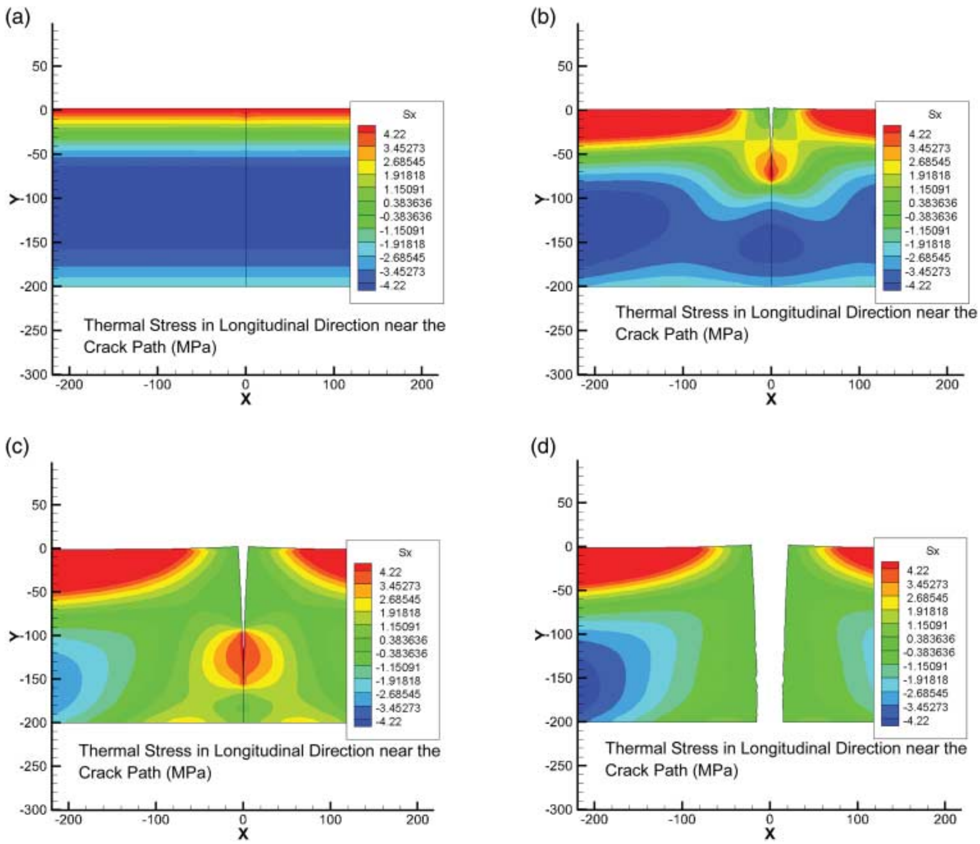

Figure 9. Results from FEA for MnROAD section 19 (calibration field section) (tensile strength = 4.22 MPa). (a) Thermal stress built-up along longitudinal direction (surface temperature = −23.3°C), (b) partial depth softening damage (surface temperature = −24.3°C), (c) partial depth crack (surface temperature = −25.4°C), and (d) fully formed crack (surface temperature = −29.6°C).

7.2. *FE runs*

Sample FE modelling results from the calibration phase of the study for MnROAD section 19 are provided in Figure 9(a)–(d). The various aspects of the cohesive zone-based FE modelling approach can be seen in these stresses and (exaggerated) deformed structure plots, which show the elevation view of the asphalt layers in the vicinity of the modelled crack. The progression of stress build-up, crack initiation, and crack propagation can be tracked as follows:

- Figure 9(a) shows high surface tensile stress (as indicated by the red colour contours), and a slight disruption in the contours at the crack interface caused by the early stages of damage (post-peak softening when tensile stress exceeded material strength at the surface of the pavement) at −23.3°C surface temperature. Mild compression is still present in the lower regions of the pavement, due to time-lag effects of heat flow.
- Figure 9(b) shows that a thermal crack has propagated partially downward through the pavement at a temperature of −24.3°C, and that a fracture process zone of about 15% of the pavement thickness exists ahead of the current crack tip location, illustrating one of the features of the cohesive zone modelling approach (length scale of fracture is directly considered). A compression zone still exists near the bottom of the asphalt layer.
- Figure 9(c) shows a later stage of crack propagation, where the fracture process zone has grown in size, and demonstrates that a compression zone no longer exists (which may partially explain the expansion of the fracture process zone) at a temperature of −25.4°C.
- Figure 9(d) shows a fully formed crack, occurring around −29.6°C. In reality, IlliTC considers the pavement section as fully cracked prior to this analysis step, as described below.

7.3. *Model calibration discussion*

Model calibration in pavement studies acknowledges the significant complexities associated with pavement materials, construction, climatic effects, traffic loading, and performance. Pavement performance model calibration is almost always needed as a result. In the case of thermal cracking, factors such as construction variability, inability to model ageing and ageing gradients with accuracy, approximate nature of tests and material models, approximate nature of climatic records and pavement temperature predictions, the presence of load-associated effects and damage on pavement, etc., exist and result in the need for model calibration.

A number of factors were available to be used for model calibration, including: fracture energy multiplier, tensile strength multiplier, thermal coefficient multiplier, crack tip definition, and beta (β_1) and sigma (σ) parameters from the probabilistic crack distribution model were readily available for use in model calibration. As a preliminary approach, it was decided to leave the material property factors as uncalibrated, and focus on the following three factors for model calibration: crack tip definition, beta (β_1) and sigma (σ) parameters. Crack tip definition refers to the fact that more than one material state can be considered as the point of crack initiation in the cohesive zone modelling technique. For instance, in Figure 5, any point along the post-peak softening curve (declining linear function in the case of the bi-linear CZM, which represents the gradual accumulation of material damage and loss of load-carrying capacity across the forming crack as the material separates) could be selected as the arbitrarily chosen location of the crack tip. The point at where the softening curve reaches zero traction (the right-hand limit of the plot shown in Figure 5) is arguably the point where the material no longer possesses the ability to heal. However, it can also be argued that an intermediate point along the softening curve may be a realistic choice for the crack tip. After examining the FE results from the MnROAD calibration FE runs, it was decided that the crack tip would be defined as the point in the pavement along the line of cohesive zone elements where a softening threshold of 75% post-post peak decay of material strength (25% traction remaining) is reached. In addition, an identical beta factor ($\beta_1 = 400$ m of cracking per 500 m of pavement section) as used in the original TCModel as calibrated in the *MEPDG* would be used. Finally, the sigma parameter in the probabilistic crack distribution model was calibrated to a value of $\sigma = 1.1$.

Table 5. IlliTC model calibration results.

MnROAD cell	Binder grade	Measured field cracking (m/500 m)	Predicted field cracking[a] (m/500 m)
03	PEN 120/150 (PG 58-28)	182	0
19	AC20 (PG 64-22)	>200 (547)	>200 (max. allowable cracking)
33	PG 58-28	91	94
34	PG 58-34	6	0

[a]Predictions are made using non-synchronised climate files.

The results of the calibrated IlliTC model, using MnROAD pavement sections, are presented in Table 5. As can be seen, reasonable modelling predictions were achieved for three out of the four sections evaluated. For instance, MnROAD section 19, which experienced very high pavement cracking due to the use of an AC-20 binder (PG 64-22) in a PG XX-34 climate was predicted to have a maximum level of cracking. Recall that when the maximum predicted crack depth is reached (crack depth = thickness of pavement), this implies that the average crack is equal to the pavement thickness. Thus, half of other pavement cracks will be less than the thickness of the pavement and, therefore, not yet counted as thermal cracks. Stated otherwise, the probabilistic crack distribution model has a maximum cracking level of 200 m of cracking per 500 m section, when = 400. MnROAD section 33 was found to have a cracking level of 94 m of cracking, as compared with a measured level of 91 m (this was the section that drove the calibration of = 1.1), and MnROAD section 34 was found to have a cracking amount of 0 m as compared with a measured cracking level of 6 m. The only poor prediction that resulted was for MnROAD section 03, where zero cracking was predicted as compared with 182 m of measured cracking. It should be noted, however, that IlliTC did indicate that softening damage had begun to occur in this section (although not enough to reach the 75% softened threshold). It is also acknowledged that the time period for the IlliTC simulation was shorter than the period of field performance reported for section 03. In addition, the master curve data used in the calibration were less-than-optimal, with data from only two test temperatures being available (three is preferred). Rather than add additional calibration factors to IlliTC, it was decided that the aforementioned calibration parameters were sufficient for the calibration of IlliTC. However, it is recommended that IlliTC be recalibrated to local conditions to arrive at better model accuracy. Model validation using an independent data set is provided in the next section of the article.

8. Field validation

Five pavement sections were constructed in Olmsted County, Minnesota, during the 2006 construction season, which were used for the analysis portion of the validation process. The mixes were sampled during the construction process and were characterised extensively in the lab. Detailed information about the pavement sections and mixture properties obtained experimentally is documented elsewhere (Marasteanu et al., 2012). The five Olmstead Co. pavement sections were simulated using IlliTC, and the results from the preanalyser and FEA are briefly presented in the following section.

Table 6 presents the results from the IlliTC preanalyser routine. As can be seen, only section 4 experienced a critical tensile stress level in the five years analysed. The preanalyser thermal stresses for this section are presented in Figure 10. Since this was the worst section in terms of field cracking, this indicates that the IlliTC programme has correctly ranked the five field sections.

Based on the preanalyser results, the validation section 4 was selected for detailed presentation herein. The thermal stresses and damage predicted by IlliTC for section 4 is presented in Figure 11.

Table 6. IlliTC preanalyser results and field cracking for the validation sections.

Validation section	Number of critical events (predicted by preanalyser)	Field performance (transverse cracking m/500 m)
1	0	23 (low)
2	0	2 (very low)
3	0	29 (low)
4	1	53 (moderate)
5	0	25 (low)

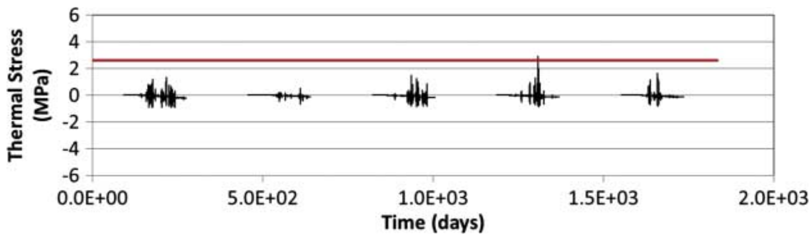

Figure 10. Preanalyser results for validation section 4.

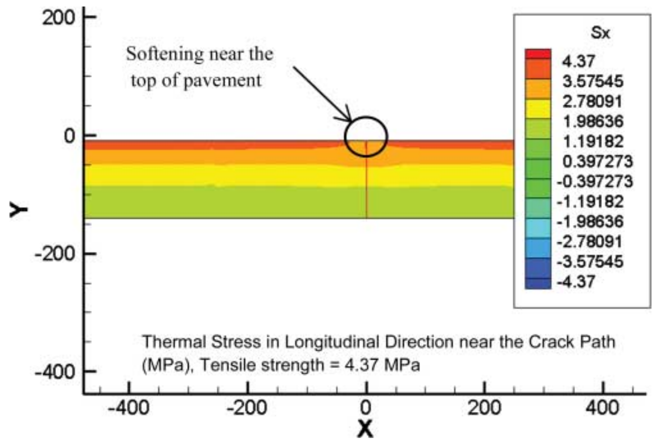

Figure 11. Thermal stresses at the end of the critical event for validation section 4.

These results show that while zero cracking was predicted, softening was activated along the cohesive zone fracture elements. As discussed previously, the IlliTC system uses the probabilistic crack distribution model to predict field cracking from the FEA. Based on the preanalyser and FEA, the IlliTC field cracking predictions for validation sections are presented in Table 7.

As evident from Table 7 under the current calibration parameters established in the previous section, zero cracking was predicted for all sections. Given the fact that most of the sections have experienced low cracking to date, it can be concluded that IlliTC under its current calibration is slightly under-predicting the cracking behaviour for these sections. It should also be noted that a limited amount of creep compliance data were available for these sections (testing at two temperatures instead of the preferred three), so errors caused by incomplete compliance data could also have contributed to the under-prediction observed. Given the fact that one of the five sections in the calibration data set was also under-predicted, the validation trials here may suggest that

Table 7. IlliTC predictions and field cracking for the validation sections using calibrations discussed in the previous section.

Validation section	Predicted crack depth (mm)	Predicted field cracking (m/500 m)	Field performance transverse cracking m/500 m)
1	0	0	23
2	0	0	2
3	0	0	29
4	0	0[a]	53
5	0	0	25

[a]Softening was predicted, indicating that thermal cracking would likely result if a longer analysis period was used.

IlliTC should be recalibrated to produce higher levels of cracking. However, given the limited validation data available and since local calibration is recommended before implementing IlliTC in a given region, further calibration of IlliTC using the current field data was not pursued herein.

9. Summary, conclusions, and future research tasks

A new TCModel called 'IlliTC' was developed as part of a recently completed pooled fund study on LTC. Various components of IlliTC model and their verification along with model calibration and preliminary validation were presented, including: a user-friendly GUI called Visual LTC; an FE modelling engine involving viscoelastic bulk material and cohesive zone fracture elements, and a probabilistic crack distribution model, identical to the one used in the original TCModel programme. Model calibration strategies, including a discussion regarding crack tip definition in the cohesive zone modelling scheme was also presented.

The model calibration was conducted using four MnROAD sections, three of which (19, 33, and 34) were found to have very good model predictions after calibration. The only unreasonable prediction that resulted was associated with MnROAD section 03, where zero cracking was predicted as compared with 182 m of measured cracking. It should be noted, however, that IlliTC did in fact indicate that softening damage had begun to occur in this section (although not enough to reach the 75% softened threshold). The time period for the IlliTC simulation in this case was shorter than the period of field performance reported for section 03. In addition, the master curve data used in the calibration were less-than-optimal, with data from only two test temperatures being available (three is preferred). Very limited model validation data were available, with only limited cracking observed to date in the four validation sections studied (Olmstead Co., MN, USA). IlliTC predictions were in general agreement with the observed field cracking (zero cracking predicted versus very low to low cracking observed in most sections). The calibration and validation sections indicated that IlliTC should probably be recalibrated to produce slightly larger cracking predictions; however, it is recommended that such calibration be performed by highway agencies, designers, or researchers using local material properties and local field performance data.

Over the course of this study several future research tasks were identified that would further enhance the capabilities and applications of the IlliTC system. Some of the future implementation tasks for IlliTC are as follows:

- The current model assumes a constant CTEC; the future version can include the bilinear thermo-volumetric trend observed by researchers from University of Wisconsin at Madison.

- The effects of oxidative ageing are not included in the current simulations; the ageing will have very pronounced effect on the thermal cracking performance due to stiffening and embrittlement of asphalt mixtures near the pavement surface. A material ageing model, such as the one used in AASHTO *MEPDG*, can be utilised in the IlliTC system to include the effect of ageing. Furthermore, at low temperatures asphalt binders may also exhibit significant non-oxidative stiffening (steric hardening), the effect of steric hardening on cracking performance should be evaluated.
- The IlliTC system does not account for multiple asphalt mix types, the extension to multiple material types requires minor modifications to the software without need for new development in FEA code. It is anticipated that the future version will have this capability.
- The current IlliTC utilises single fracture energy input at one temperature; the system should be modified to allow users to input fracture properties at multiple temperatures (or in a functional form).
- The current programme is currently calibrated using global fracture energy for FE simulations; however, researchers have shown that the use of local fracture energy improves the prediction accuracy. Local fracture energy can be determined using the raw data from fracture tests and the viscoelastic properties of the mix, along with inverse analysis by modelling the test specimen. This capability should be added to a subsequent version of IlliTC.
- An algorithm recently developed to extract mixture tensile strength from DC(T) test data should be added to IlliTC, so that users can avoid the need to directly input mixture tensile strength data.
- The effects of variability in material property inputs on the predicted thermal cracking performance should be evaluated.
- A much more comprehensive calibration and validation of IlliTC is needed; the current calibration was limited to four pavement sections at one location. With more extensive calibration, the prediction capability of IlliTC should be improved, which can be evaluated using additional validation sections.

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References

Apeagyei, A. K., Dave, E. V., & Buttlar, W. G. (2008). Effect of cooling rate on thermal cracking of asphalt concrete pavements. *Journal of Association of the Asphalt Paving Technologists*, *77*, 709–738.

Baek, J., Ozer, H., Wang, H., & Al-Qadi, I. L. (2010). Effects of interface conditions on reflective cracking development in hot-mix asphalt overlays. *Road Materials and Pavement Design*, *11*(2), 307–335.

Brown, E. R., Kandhal, P. S., Roberts, F. L., Kim, Y. R., Lee, D.-Y., & Kennedy, T. W. (2009). *Hot mix asphalt materials, mixture design and construction*. Lanham, MD: National Asphalt Pavement Association.

Buttlar, W. G., Roque, R., & Reid, B. (1995). Automated procedure for generation of creep compliance master curve for asphalt mixtures. *Transportation Research Record*, *1630*, 28–36.

Buttlar, W. G., Sahu, R., Behnia B., & Dave, E. V. (2013). *Determination of asphalt concrete tensile strength from the disk-shaped compact tension (DCT) fracture test*/ Submitted for publication.

Dave, E. V., & Buttlar, W. G. (2010). Low temperature cracking prediction with consideration of temperature dependent bulk and fracture properties. *Road Materials and Pavement Design*, *11*(SI), 33–59.

Dave, E. V., Leon, S. E., & Park, K. (2011). Thermal cracking prediction model and software for asphalt pavements. *Proceedings of the First T&DI Congress 2011, ASCE Conference Proceedings*, *398*(41167), 64–72.

Dave, E. V., Paulino, G. H., & Buttlar, W. G. (2012). Viscoelastic functionally graded finite element method with recursive time integration and applications to flexible pavements. *International Journal for Numerical and Analytical Methods in Geomechanics*, *36*(9), 1194–1219.

Deme, I. J., & Young, F. (1987). Ste. anne test road revisited 20 years later. *Proceedings of Canadian Technical Asphalt Association*, *32*, 254–283.

Islam, M. S., & Buttlar, W. G. (2012). Effect of pavement roughness on user costs. *Transportation Research Record*, *2285*, 47–55

Kim, Y.-R., Aragao, F. T. S., Allen, D. H., & Little, D. N. (2010). Damage modeling of bituminous mixtures considering mixture microstructure, viscoelasticity, and cohesive zone fracture. *Canadian Journal of Civil Engineering*, *37*(8), 1125–1136.

Lytton, R. L., Roque, R., Uzan, J., Hiltunen, D. R., Fernando, E., & Stoffels, S. M. (1993). *Performance models and validation of test results* (Final Report to Strategic Highway Research Program; Asphalt Project A-005, SHRP Report A-357). Washington, DC: National Research Council.

Marasteanu, M., Moon, K. H., Teshale, E. Z., Falchetto, A. C., Turos, M., Buttlar, W., … Buss, A. (2012). *Investigation of low temperature cracking in asphalt pavements national pooled fund study -phase II* (Report No. MN/RC 2012-23). St Paul, MN: Minnesota Department of Transportation.

Marasteanu, M., Zofka, A., Turos, M., Li, X., Velasquez, R., Li, X., … McGraw, J. (2007). *Investigation of low temperature cracking in asphalt pavements* (Report No. 776). St Paul, MN: Minnesota Department of Transportation, Research Services MS 330.

Nam, K., & Bahia, H. U. (2004). Effect of binder and mixture variables on glass transition behavior of asphalt mixtures. *Journal of Association of Asphalt Paving Technologists*, *73*, 89–119.

Nam, K., & Bahia, H. U. (2009). Effect of modification on fracture failure and thermal-volumetric properties of asphalt binders. *Journal of Materials in Civil Engineering*, *21*(5), 198–209.

Paris, P. C., Gomez, M. P., & Anderson, W. E. (1961). A rational analytic theory of fatigue. *The Trend in Engineering*, *13*, 9–14.

Roque, R., Hiltunen, D. R., & Buttlar, W. G. (1995a). Thermal cracking performance and design of mixtures using superpave (TM). *Journal of the Association of Asphalt Paving Technologists*, *64*, 718–735.

Roque, R., Hiltunen, D. R., Buttlar, W. G., & Farwana, T. (1995b). *Development of the SHRP superpave mixture specification test method to control thermal cracking performance of pavements*. STP1265 Engineering Properties of Asphalt Mixtures and the Relationship to their Performance (G. Huber and D. Decker, Eds.). ASTM Special Technical Publication. Philadelphia, PA: American Society for Testing and Materials, p. 19.

Roseler, J. R., Paulino, G. H., Park, K., & Gaedicke, C. (2007). Concrete fracture prediction using bilinear softening. *Cement and Concrete Composites*, *29*(4), 300–312.

Soares, J. B., Colares de Freitas, F. A., & Allen, D. H. (2004). Crack modeling of asphaltic mixtures considering heterogeneity of the material. *Transportation Research Record*, *1832*, 113–120.

Song, S. H., Paulino, G. H., & Buttlar, W. G. (2006). A bilinear cohesive zone model tailored for fracture of asphalt concrete considering viscoelastic bulk material. *Engineering Fracture Mechanics*, *73*(18), 2829–2848.

Song, S. H., Wagoner, M. P., Paulino, G. H., & Buttlar, W. G. (2008). δ25 Crack opening displacement parameter in cohesive zone models: Experiments and simulations in asphalt concrete. *Fatigue and Fracture of Engineering Material and Structures*, *31*(10), 850–856.

Yi, S., & Hilton, H. H. (1994). Dynamic finite element analysis of viscoelastic composite plates in the time domain. *International Journal of Numerical Methods in Engineering*, *37*(12), 4081–4096.

Zocher, M. A., Groves, S. E., & Allen, D. H. (1997). A three-dimensional finite element formulation for thermoviscoelastic orthotropic media. *International Journal of Numerical Methods in Engineering*, *40*(12), 2267–2288.