2013

KENTRACK 4.0: A RAILWAY TRACKBED STRUCTURAL DESIGN PROGRAM

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KENTRACK 4.0: A RAILWAY TRACKBED STRUCTURAL DESIGN PROGRAM

THESIS

A thesis submitted in partial fulfillment of the requirements for the degree Master of Science in Civil Engineering in the College of Engineering at the University of Kentucky

By

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2013

ABSTRACT OF THESIS

KENTRACK 4.0: A RAILWAY TRACKBED STRUCTURAL DESIGN PROGRAM

The KENTRACK program is a finite element based railway trackbed structural design program that can be utilized to analyze trackbeds having various combinations of all-granular and asphalt-bound layered support. It is applicable for calculating compressive stresses at the top of subgrade, indicative of potential long-term trackbed settlement failure. Furthermore, for trackbeds containing asphalt layer, it is applicable for calculating tensile strains at the bottom of the asphalt layer, indicative of potential fatigue cracking. The program was recently expanded to include both English and international units. A procedure has been incorporated to provide a path to save results in a text formation in post-Windows XP operating systems. More importantly, properties of performance graded (PG) asphalt binders and the Witczak E* predictive model have been incorporated in the 4.0 Version of the program. Component layers of typical trackbed support systems are analyzed while predicting the significance of layer thicknesses and material properties on design and performance. The effect of various material parameters and loading magnitudes on trackbed design and evaluation, as determined and predicted by the computer program, are presented. Variances in subgrade modulus and axle loads and the incorporation of a layer of asphalt within the track structure have significant effects on subgrade vertical compressive stresses and predicted trackbed service lives. The parameter assessments are presented and evaluated using sensitivity analysis. Recommendations for future research are suggested.

KEYWORDS: KENTRACK, Railway Asphalt Trackbed, Asphalt Binder, Asphalt Dynamic Modulus, Predicted Service Life

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December, 1, 2013

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December 2, 2013

ACKNOWLEDGEMENTS

The following thesis, while an individual work, benefited from the insights and direction of several people. First, I would like to express my sincere gratitude to my advisor Dr. Jerry G. Rose for the continuous support of my study and research, for his patience, motivation, enthusiasm, and immense knowledge. His guidance helped me during my research and writing of this thesis. Besides my advisor, I would like to thank the other members of my thesis committee, (Dr. Jerry Rose, Dr. Reginald Souleyrette, and Dr. Mei Chen), for their encouragement and insightful comments.

The contributions of previous graduate students Bei Su (MSCE, 03) and Nithin K. Agarwal are appreciated. Portions of their research were utilized as resource material for my thesis. The technical assistance provided by Yi-Hung Jen to share his precious time during the process of programming is gratefully acknowledged.

In addition to the technical and instrumental assistance above, I received equally important assistance from family and friends. My mother, Xiaolan He, and my father, Zhiping Liang, instilled in me, from an early age, the desire and skills to obtain the Master's degree.

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CHAPTER 1. INTRODUCTION

Background

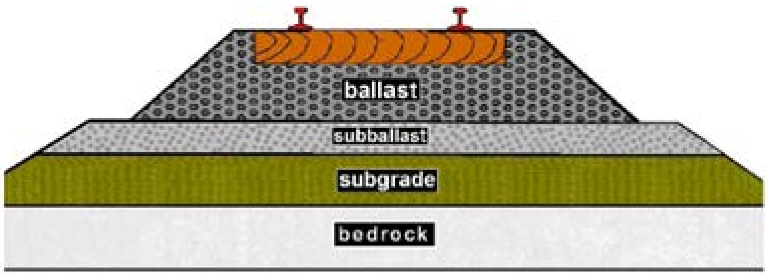
Railroads have been a mode of transportation in the United States for over 180 years. During this period, train speeds, annual gross ton-miles, and axle loads have increased significantly. On U.S. railroads, peak axle loads in common revenue service have increased to 36 tons. The aspect of 39-ton axle load is undergoing extensive research. To accommodate these changes, larger rails (i.e. RE136, RE140), and improved wood, concrete, and speciality ties are used. During the past thirty years, several new trackbed designs and support structures have been developed in several countries. In the United States, Hot Mix Asphalt trackbeds have been developed mainly for freight lines. The main attributes are — to provide increased support, to accommodate heavier axle loads, and to reduce trackbed maintenance costs, thereby favorably impacting train operations (Rose, 2013). Asphalt trackbeds are used extensively in Europe and Asia for new high-speed passenger lines to provide high quality track geometric features for safe operations at high speeds (Rose, et al., 2011).

Hot Mix Asphalt Trackbed

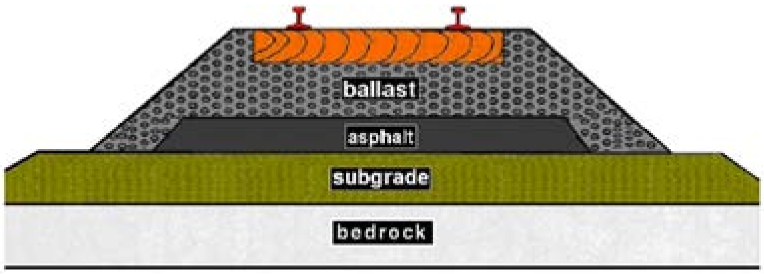
Originally, two types of asphalt trackbed designs were used for design evaluations. One is termed “underlayment” because the asphalt is used as a mat or sublayer between ballast and subgrade instead of all-granular subballast. The other is termed “overlayment” or “full depth” because the asphalt mat is placed directly on the subgrade. Ties are placed on the top of asphalt. There is no ballast layer in asphalt overlayment trackbeds.

The underlayment design is preferred by U.S. railroad engineers because the underlayment design maintains the ballast within the structure so that the track geometry can be easily adjusted. Also, the asphalt layer is maintained in a protected environment because it is buried under the ballast which provides protection, such as minimizing sunlight exposure and temperature variances. Due to its widespread use, only asphalt underlayment is documented in the thesis.

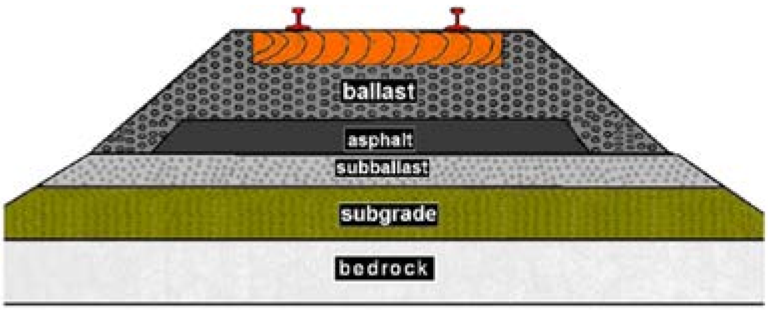
Recently, a modified asphalt underlayment design, termed "combination trackbed", containing both an asphalt layer and a subballast layer, has been evaluated. It is composed of ballast, asphalt, subballast, subgrade and bedrock. Subballast is considered as additional protection for the subgrade. The typical trackbed structures evaluated in this thesis are shown as Figure 1.1.



(a). All-Granular Trackbed



(b). Asphalt Underlayment Trackbed



(c). Combination Trackbed

Figure 1.1 Three Types of Trackbeds

Numerous test and revenue trackbeds using asphalt have been built over many types of subgrades. Thickness of the asphalt layer was purposefully varied. It has been shown that asphalt underlayment trackbeds impart the following benefits to the track structure according to performance measurements acquired from test installations (Asphalt Institute, 1998), (Rose, Su and Twehues, 2004):

1. A strengthened track support layer below the ballast to uniformly distribute reduced loading stresses to the roadbed (subgrade);

2. A waterproofing and confinement layer for the underlying roadbed that provides consistent load-carrying capability for track structures — even on roadbeds of marginal quality;

3. An impermeable layer to divert water to side ditches which essentially eliminates subgrade moisture fluctuations;

4. A consistently high level of confinement for the ballast so the ballast can develop high shear strength and provide uniform pressure distribution;

5. A resilient layer between the ballast and the roadbed is needed to reduce the likelihood of subgrade pumping without substantially increasing track stiffness; and

6. An all-weather and uniformly stable surface for placing the ballast and track superstructure.

Asphalt Dynamic Complex Modulus

Dynamic complex modulus (E*) is a measure of the stiffness of viscoelastic materials. It is one of the most import asphalt properties used to examine responses of asphalt layers such as stresses, strains, etc. Numerous E* predictive models and related equations have been developed over the past fifty years. The significant E* predictive models over the last fifty years are Shell Oil, Shook and Kallas, Hirsh, and Witczak Models. The empirical Witczak E* predictive model, developed in 1972, is currently a popular model, due to the application of MEPDG (Mechanistic-Empirical Pavement Design Guide) program for highway pavement design.

Revisions were made to the model in 1995 and 1999. (Bari and Witczak, 2006). The model is based on a sigmoidal function which is used to describe the relationship between the dynamic modulus and loading rate. Aggregate gradation, volumetric properties of mixtures and binder rheological properties are addresses in the Witczak Model. Statistical results show an R^2 of 0.96 and S_e/S_y of 0.24 indicating the model has high accuracy and is good for asphalt dynamic modulus prediction.

Superpave

The previous asphalt cement (AC) grading system, which the current KENTRACK 3.0 Version program utilizes, was primarily based on empirical tests, these being either "penetration" or "viscosity" grades. Empirical specifications rely solely on practical experience and observations without regard for asphalt performance. Therefore the specification is based on the results from a given situation. When the conditions change, the results may no longer be consistent, such as viscosity. Viscosity classification of asphalt cements was used at one temperature 140℉; however, it is known that the viscosity varies with temperature changes for different sources of asphalt cements, although they may have the same viscosity at 140℉. Another shortcoming of the AC graded system is that long-term asphalt aging is not taken into consideration. The tests are performed on un-aged or "tank" asphalt and on artificially short-term aged asphalt to simulate construction. No tests are performed to simulate in-service aging, which occurs when the asphalt reacts with oxygen in the atmosphere by oxidation. Moreover, the AC system's tests do not cover the temperature extremes that asphalt binders endure. Binders that produce similar results at the temperatures used for viscosity testing may have very different results at other temperatures experienced by the asphalt.

However, Superpave has changed asphalt manufacturing and specifications. Grading based on viscosity and penetration has been replaced with a performance graded (PG) system. No longer are the tests empirical. The PG specification uses tests to measure physical properties, such as dynamic shear modulus, creep stiffness, direct tension, etc., which can be directly related to field performance based on engineering principles. PG binders are tested under conditions that are similar to the three critical stages of a binder's life – (1) transport and storage, (2) mix production and construction, and (3) long term aging. For the third stage, long term aging, the binder is aged using a pressure aging vessel. The pressure aging vessel exposes a sample to heat and pressure to simulate years of in-service aging of asphalt. Therefore, by using the performance grading (PG) system, especially for the long term aging, asphalt test results can better simulate actual field situations.

Problem Statement

In the previous 2.0 and 3.0 KENTRACK versions, asphalt dynamic modulus was predicted using the method developed by the Asphalt Institute (Huang and Witczak, 1979), where asphalt dynamic modulus is a function of temperature, viscosity at 135 °F, loading frequency, percentage of air voids, bitumen and aggregate passing #200 sieve. A shortcoming of the model is a lack of consideration on the temperature sensitivity of viscosity. Viscosity increases when temperature decreases. Thus, in the asphalt dynamic modulus predictive equation, using a constant value for viscosity at 135 °F may lead to an underestimate of asphalt dynamic moduli. Further, since Superpave has improved the performance of asphalt with new asphalt design and PG System for asphalt binders, the old asphalt dynamic modulus predictive model existing in KENTRACK 3.0 (the latest version released) is not considered appropriate to predict dynamic modulus of PG asphalt binders.

Also, the current program has bunches of "bugs" that needed to be addressed. The 3.0 program needs to be restarted and parameters need to be reset during each run if users vary some of the parameters. Users cannot save calculated documents in post-Windows XP operating system. In addition, only the English unit system is included in the 3.0 version, which limits the user friendliness of the program for international purposes.

Objectives and Methodology

The study is focused on updating the predictive model for asphalt dynamic modulus that is appropriate for both the AC system based asphalt binders that were used in previous versions and the PG system based (Superpave) asphalt binders that are used in current design and production. The primary objectives are as follows:

1. Updating properties of asphalt binders. The chemistry of modern asphalt binders has changed significantly relative to the previous asphalt cements. Binders are not as sensitive to temperature changes and therefore their properties, such as viscosity, softness and brittleness are not as adversely affected as are asphalt cements. This change of material properties need to be incorporated into the KENTRACK Design Program. An increased design life for asphalt binders is considered to be normal today as compared to the previous AC asphalt cements.

2. Incorporating the Witczak predictive E* model to predict asphalt dynamic modulus. The Witczak predictive E* model is developed from 205 laboratory mixtures including 171 unmodified asphalt binders and 34 chemically modified binders that produced 2750 data points. The huge database with an R^2 of 0.96 and S_e/S_y of 0.24 guarantees the accuracy of the prediction (Advanced Research Associates, 2004).

3. Sensitivity analysis. Varying different parameters, such as magnitudes of axle loads, types of asphalt binders, thickness of asphalt layers, variability of subgrade modulus, types of ties, etc., to analyze the effects on mechanical behavior of the trackbeds. Compressive stresses at the top of the subgrade layer and tensile strains at the bottom the asphalt layer are calculated as well as predicted design lives for the associated layers.

4. Comparison. Compare the new calculated results according to analyses from the current 3.0 KENTRACK and the revised 4.0 KENTRACK.

5. Fix “bugs”. This includes retaining parameter values from previous runs so that users may compute results without restarting the program, providing a

location to save result documents in post-Windows XP operating system, and store files including input data for further reference.

6. Expanding the program to include international unit system.

CHAPTER 2. LITERATURE REVIEW

Finite Element Method

The Finite element method is a numerical technique for finding approximate solutions to boundary value problems. For problems involving complicated geometries, loadings, and material properties, it is generally not possible to obtain analytical mathematical solutions. Analytical solutions are those given by a mathematical expression that yields the values of the desired unknown quantities at any location in a body (total structure or physical system of interest), and thus valid for an infinite number of locations in the body. These analytical solutions generally require the solution of ordinary or partial differential equations, which, because of the complicated geometries, loadings, and material properties, are not usually obtainable. Hence, numerical methods, such as the finite element method, are used to obtain acceptable solutions. Numerical methods yield approximate values of the unknowns at discrete numbers of points in the continuum. The process of modeling a body by dividing it into an equivalent system of smaller bodies or units (finite elements) interconnected at points common to two or more elements (nodal points or nodes) and/or boundary lines and/or surfaces is called discretization. The finite element method discretizes a larger domain into many small subdomains over where many simple element equations are connected to solve structural problems (Logan, 2011).

Spring Element

A linear elastic spring is a mechanical device capable of supporting axial loading only, and the elongation or contraction of the spring is directly proportional to the applied axial load. The constant of proportionality between deformation and load is referred to as the spring constant k (Logan, 2011). The stiffness matrix of the spring can be written as Eq. 2.1.

$$[k_e] = \begin{bmatrix} k & -k \\ -k & k \end{bmatrix} \qquad \text{(Eq. 2.1)}$$

where, k_e = spring stiffness;

k = is spring constant.

Beam Element

A beam is a long slender structural member generally subjected to transverse loading that produces significant bending effects as opposed to twisting or axial effects. This bending deformation is measured as a transverse displacement and a rotation. Hence, the degrees of freedom considered per node are a transverse displacement and a rotation (Logan, 2011). The stiffness matrix of the beam element is expressed as Eq. 2.2:

$$\begin{Bmatrix} f_{1y} \\ m_1 \\ f_{2y} \\ m_2 \end{Bmatrix} = \frac{EI}{l^3} \begin{bmatrix} 12 & 6l & -12 & 6 \\ 6l & 4l^2 & -6l & 2l^2 \\ -12 & -6l & 12 & -6l \\ 6 & 2l^2 & -6l & 4l^2 \end{bmatrix} \begin{Bmatrix} d_{1y} \\ \theta_1 \\ w_{2y} \\ \theta_2 \end{Bmatrix} \qquad \text{(Eq. 2.2)}$$

Where,

E = Young's modulus;

I = moment of inertia;

l = Length of the beam;

f_{1y}, f_{2y} = vertical force at Node 1 and Node 2;

m_1, m_2 = moment at Node 1 and Node 2;

d_{1y}, d_{2y} = vertical force at Node 1 and Node 2;

θ_1, θ_2 = rotation at Node 1 and Node 2.

Multi-Layered System

Trackbeds are layered with different material on each layer and cannot be represented by a homogeneous mass. Therefore, Burmister first developed solutions for a two-layer system and then extended them to a three-layer system (Burmister, 1943). With the advent of computers, the theory can be applied to a multi-layered system with any number of layers (Huang, 1968).

The basic assumptions to be satisfied are:

1. Each layer is homogeneous, isotropic, and linearly elastic with an elastic modulus E and a Poisson's ratio.

2. The material is weightless and infinite in areal extent.

3. Each layer has a finite thickness, but the lowest layer is infinite in thickness.

4. A uniform pressure is applied on the surface over a circular area of radius.

5. Continuity conditions are satisfied at the layer interfaces, as indicated by the same vertical stress, shear stress, vertical displacement, and radial displacement. For frictionless interface, the continuity of shear stress and radial displacement is replaced by zero shear stress at each side of interface.

The detailed derivation of multilayered elastic solution can be found in references (Burmister, 1943 and Huang, 1968). Only a brief description of the method is presented herein. Each layer is described by its modulus of elasticity, Poisson's ratio, and distance from the top surface to its interface. The methodology for the solution of this system follows the classical theory of elasticity as introduced by Timoshenko (Timoshenko and Gere, 1972). An Airy's stress function, which satisfies the following governing biharmonic equation (also called equation of compatibility),

$$\nabla^4\phi = \nabla^2\nabla^2 = 0 \qquad \text{(Eq. 2.3)}$$

where, ∇^2 is the Laplacian Operator.

Eq. 2.3 is assumed for each layer. For the case of axial symmetry, as it is in this case,

$$\nabla^4 = \nabla^2\nabla^2 = (\frac{\partial^2}{\partial r^2} + \frac{1}{r}\frac{\partial}{\partial r} + \frac{\partial^2}{\partial z^2})(\frac{\partial^2}{\partial r^2} + \frac{1}{r}\frac{\partial}{\partial r} + \frac{\partial^2}{\partial z^2}) \qquad \text{(Eq. 2.4)}$$

where,

r = cylindrical coordinates for radial direction;

z = cylindrical coordinates for vertical direction;

If the ϕ function is found for a layer, the stresses and deflections in that layer can be easily obtained by using Hook's Law as Eq. 2.5.

$$\begin{aligned}
\sigma_z &= \frac{\partial}{\partial z}[(2-\upsilon)\nabla^2\phi - \frac{\partial^2\phi}{\partial z^2}] \\
\sigma_r &= \frac{\partial}{\partial z}[\upsilon\nabla^2\phi - \frac{\partial^2\phi}{\partial z^2}] \\
\tau_{rz} &= \frac{\partial}{\partial z}[(1-\upsilon)\nabla^2\phi - \frac{\partial^2\phi}{\partial z^2}] \qquad \text{(Eq. 2.5)} \\
w &= \frac{1+\upsilon}{E}[(1-2\upsilon)\nabla^2\phi + \frac{\partial^2\phi}{\partial r^2} + \frac{1}{r}\frac{\partial\phi}{\partial r}] \\
u &= -\frac{1+\upsilon}{E}[\frac{\partial^2\phi}{\partial r^2}]
\end{aligned}$$

where,

σ_z = vertical stress;

σ_r = radial stress;

τ_{rz} = shear stress;

w = vertical deflection;

u = radial deflection;

E = layer modulus of elasticity;

υ = layer Poission's ratio.

Hankel Transform

The Hankel transform is an integral transform operation first developed by the mathematician Hermann Hankel. It is also known as the Fourier-Bessel transform (Piessens, 2000). Just as the Fourier transform for an infinite interval is related to Fourier series over a finite interval, the Hankel transform over an infinite interval is related to the Fourier-Bessel series over a finite interval.

In mathematics, the Hankel transform express any function f(r) as the weighted sum of an infinite number of Bessel functions of the first kind $J_v(kr)$. The Bessel functions in the sum are all the same order v, but differ in a scaling factor k along the r-axis. The necessary coefficient F_v of each Bessel function in the sum, as a function of the scaling factor k constitute the transformed function.

The Hankel transform of a function f(r) is valid at every point at which f(r) is continuous provided that the function is define in (0, ∞), and the integral $\int_0^\infty | f(r) | r^{1/2} dr$ is finite.

The Bessel functions form an orthogonal basis with respect to the weighting factor r as Eq. 2.6.

$$\int_0^\infty J_v(kr) J_v(k'r) r dr = \frac{\delta(k-k')}{k} \qquad \text{(Eq. 2.6)}$$

where, k and k' are greater than zero.

The Hankel transform of order v of a function f(r) is given by Eq. 2.7.

$$F_v(k) = \int_0^\infty f(r) J_v(kr) r dr \qquad \text{(Eq. 2.7)}$$

where, J_v is the Bessel function of the first kind of order v with $v \geq -1/2$. The inverse Hankel transform of $F_v(k)$ is defined as Eq. 2.8 which can be readily verified using the orthogonal relationship described as Eq. 2.6.

$$f(r) = \int_0^{\infty} F_v(k) J_v(kr) k dk \qquad \text{(Eq. 2.8)}$$

Previous Railway Trackbed Design Programs

In order to develop a structural design and analysis procedure for railway trackbeds it is necessary to understand track behavior as a function of loading conditions, material properties, and track configuration. Starting in 1913 and continuing through 1942, the A.N. Talbot Joint Committee validated the basic theory of beam on elastic foundation and developed empirical equations as aids for track design (AREA, 1980). The model consists of a continuous beam representing the rail on an elastic Winkler-type foundation supported by the combined effect of ties, ballast, subballast, and subgrade. The foundation is assumed to have sufficient stiffness or track modulus to resist the imposed loadings on the rail. Later, computer models were developed utilizing combinations of finite element analysis and layered systems. These include FEARAT (Fateen, 1972), ILLITRACK (Robnett, et al., 1976), and GEOTRACK (Chang, et al., 1980).

FEARAT (Finite Element Analysis of Railway Asphalt Trackbed) Program

FEARAT was developed at the University of Maryland in 1972 (Fatten, 1972). As the name implied, it was designed for asphalt overlayment trackbeds, in which the ballast layer is replaced by hot mix asphalt. In order to simulate three dimensional characteristics of the track system, the model is divided into three stages for analysis. The trackbed is first considered as a one dimensional beam over a length of 70 tie spacings, and then it is considered as a two dimensional plate over 10 tie spacings. Finally it is considered as a two dimensional plane strain continuum over 4 tie spacings. Finite element method and linear elastic theory are applied in the model.

ILLITRACK Program

ILLITRACK was developed at the University of Illinois (Robnett, et al., 1975). It is designed especially for the analysis of all-granular trackbed systems consisting of ballast, subballast and subgrade. Two-stage of pseudo-plane strain analysis with finite element method is utilized in the model to account for the three dimensional geometry of the track system. In the analysis, trackbed materials can be considered as either linear or nonlinear (stress dependent) materials. For the two-stage analysis, the trackbed is first analyzed longitudinally as a two dimensional pseudo-plane strain continuum and then transversely as another two dimensional pseudo-plane strain continuum.

In the longitudinal analysis, the track is considered to extend over a distance of 26 or 13 tie spacings on each side of the plane of symmetry. The analysis considers concentrated loads (wheel loads) acting on a rail, which in turn rests on a tie-ballast-subgrade system. Two types of elements are applied in the analysis: (a) beam spring elements to represent the rail tie structure as a continuous beam supported on a series of linear tie springs; (b) rectangular planar elements to represent the ballast-subballast-subgrade structure. For beam-spring elements, each model point has two degrees of freedom, i.e. a rotation in the longitudinal direction and displacement in the vertical direction. For planar elements in the trackbed system, each nodal point also has two degrees of freedom, i.e. the horizontal and vertical displacements. Because a symmetrical loading is considered, it is only necessary to analyze half of the structure, and the vertical boundary at the symmetric axis is retrained from horizontal movement. There is a rigid boundary placed at the bottom of subgrade.

In the transverse analysis, a tie is directly located on the top of ballast-subballast-subgrade system. The maximum vertical displacement or force obtained from the previous longitudinal analysis is applied at the rail location. The tie can be considered either as a two dimensional body or a beam. The same rectangular planar elements are used in the trackbed system. Due to the symmetric condition, only half of the system needs to be analyzed. The boundary conditions at both vertical sides as well as at the bottom are the same as prescribed in the longitudinal analysis.

In the conventional plane strain analysis with the finite element method, the thickness, t, for all elements is assumed as a constant. This assumption limits the distribution of stress in the third direction and further restricts the diminishing of stress with depth as would be expected in a three dimension case. In order to simulate the three dimensional dissipation of stress in track system, ILLITRACK incorporates a "pseudo-plane" stress in the third direction. A parameter called "angle of distribution" is assigned for each material to determine the constant rate of increase of element thickness with depth. In the longitudinal analysis, the initial thickness of an element at the surface is equal to an effective tie bearing length, which is assumed to be the length for effectively transferring the pressure from the bottom of tie to the surface of ballast. The initial thickness of element in the transverse analysis is equal to the width of tie.

The results given by the model are: (1) moments and deflections of rail, (2) tie reactions, and (3) deformations and distribution of stresses in the trackbed system.

GEOTRACK Program

GEOTRACK model was developed at the University of Massachusetts (Chang et al., 1980). It is also designed for the analysis of all-granular trackbed system. Burmister Layered system theory, as widely used in the design of highway and airport pavement, was utilized in the model to simulate a three dimensional trackbed system. The track system is divided into two portions. One is the rail-tie structure and the other is trackbed system.

In the rail-tie structure, the rails are represented as a linear elastic beams supported by a number of concentrated rail-tie reactions (rail seat loads). The connections between rail and tie are presented by a series of linear springs with constant spring stiffness which allows the individual movement of the rail. Each axle load is distributed over 11 ties (10 tie spacings), with the axis of symmetry located at a specific tie. There are no moments at both rail ends and at each rail-tie intersection. Superposition technique is applied for calculating double axles up to of 4 axles.

The ties are also represented as linear elastic beams, but lie with a 90 degrees angle from rails. Each tie is divided into ten equal rectangular elements. The tie beam is supported by the reactions (concentrated forces) from the underlying ballast layer through the center of each element. These reactive forces are then applied to the surface of the ballast as a uniform pressure distributed over a circular area equal to the area of the rectangular tie element. The center of each area coincides with that of the tie element.

In the trackbed system, ballast, subballast and underlying subgrade soil are represented as a linear elastic multilayered system. All layers are infinite in dimension over the horizontal plane. The program allows a maximum of five layers to be analyzed. The last layer is also extended to infinity in the downward direction. Each layer may have a separate modulus of elasticity, thickness (except for the last layer), and Poisson's ratio.

The GEOTRACK model also contains two other optional features, one is to account for the nonlinear (stress dependent) properties of underlying trackbed materials and the other is to allow the separation between tie and ballast contact. An independent iterative approach is used in each feature.

The output information from the model includes: rail seat loads, tie-ballast reaction, deflections of rail and tie, tie and rail bending moments, and the complete three dimensional stress state at specified locations in the trackbed system.

In summation, FEARAT was designed to analyze asphalt overlayment trackbed. ILLITRACK was developed for all-granular ballast trackbed and only contains longitudinal and transverse two-dimensional models. However, research has shown that asphalt underlayment service as a better waterproof layer than asphalt overlayment, which improves subgrade moisture control. This program uses a two-dimensional model to simulate a three dimensional situation. GEOTRACK only can be used for the analysis of all-granular ballast trackbeds and is not applicable to asphalt trackbeds and slab trackbeds. The need for a program that is able to analyze stress distributions for both asphalt and all-granular trackbeds led to the development of KENTRACK.

KENTRACK Program

KENTRACK, initially developed at the University of Kentucky, is a layer elastic finite element based computer program that can be utilized for a performance-based structural design and analysis of railroad trackbeds (Huang, et al., l984). It was initially developed to analyze traditional all-granular layered trackbeds and asphalt layered trackbeds, and to predict service lives of the associated layers. The initial version utilized a Disk Operating System (DOS) and was coded in FORTRAN language.

Later, the program known as KENTRACK 2.0.1, was moved from the DOS-based version to a user friendly Window's based interface – Graphical User's Interface (GUI). It contained four descriptive forms (or screens), and allowed users to enter varying values for the track structure components. (Rose and Konduri, 2006). In order to compare stress levels at various vertical locations in railroad trackbeds, in-situ earth pressure measurements were conducted on both heavy-haul CSX Transportation revenue service trackbeds and on the Association American Railroads Transportation Technology Center test trackbeds (Rose et al., 2004). The predictive values of subgrade compressive stress and asphalt compressive stress computed from KENTRACK were similar to the actual measurement. The conclusion demonstrated that the KENTRACK program is capable of analyzing both all-granular trackbeds and asphalt trackbeds; in-track measurements confirmed the predictive values from KENTRACK thus providing this program a measure of credibility. Although the KENTRACK 2.0.1 was made more user friendly, it had several limitations. The program did not have a default set of values and the coding was done in FORTRAN which restricts any further developments since the FORTRAN language is not highly used among software engineers. The program did not carry out validations for the input parameters which often resulted in abrupt termination of the program. There were no options for the analysis of separate trackbeds, and users were required to enter all values irrespective of the analysis.

In order to make the program more user friendly, KENTRACK 3.0 was released. It is developed entirely on .Net framework using C# (Rose, et al., 2010). The core structure of KENTRACK 3.0 is similar to that of KENTRACK 2.0.1. It has a similar GUI as the previous version, but with additional features and benefits. KENTRACK 3.0 has a built-in default set of parameters that is displayed once the user starts the program. The user is given the task to select minimum options from the drop down menu in limited places. User can also enter any values desired other than the default numbers. A "help" button was also added. Additionally, the versatility was expanded to analyze trackbeds containing a combination of granular and asphalt layers. However, calculated documents could not be saved in post-Windows XP operating system. Users have to restart the program, resetting all the parameters they have entered into the program. Moreover, properties of PG asphalt binders were not taken into consideration in KENTRACK 3.0.

Current KENTRACK 4.0 Version Program

The recently developed KENTRACK 4.0, (Witczak) Model, incorporates the functions of asphalt binders, mix properties, viscosity and loading rates. Viscosity temperature susceptibility method is used to estimate viscosity of different asphalt binders/cements. Detailed discussions of these aspects follow:

Dynamic Modulus

The complex dynamic modulus is a complex number that relates to strain for linear viscoelastic materials subjected to continuously applied sinusoidal loading in the frequency domain. The absolute value of the complex modulus, |E*|, is commonly referred to as the dynamic modulus. (Yoder and Witczak, 1975) (Witczak et al., 2002b).

HMA mixtures can be considered as a linear viscoelastic material under small strain levels (around 100με, (Schwartz, 2005)). Thus, the HMA stress-strain relationship under continuous sinusoidal loading in the linear viscoelastic region can be defined by the complex dynmaic modulus.

The complex modulus is defined as the ratio of the amplitude of the sinusoidal stress (at any given time and load frequency) and sinusoidal strain (at the same time and frequency) that results in a stready state response (Dougan et al., 2003), as shown in Eq. 2.9.

$$E^* = \frac{\sigma}{\varepsilon} = \frac{\sigma_0 e^{i\omega t}}{\varepsilon_0 e^{i(\omega t - \delta)}} = \frac{\sigma_0 \sin(\omega t)}{\varepsilon_0 \sin(\omega t - \delta)} \qquad \text{(Eq. 2.9)}$$

where,

E*= complax modulus;

σ_0= peak (maximum) stress;

ε_0= peak (maximum) strain;

δ= phase angle, degrees;

ω= angular velocity;

t= time, seconds;

i = imaginary component of the complex modulus.

Thus, the dynamic modulus is defined as Eq. 2.10

$$|E^*| = \frac{\sigma_0}{\varepsilon_0} \qquad \text{(Eq. 2.10)}$$

For pure elastic materials, $\delta = 0$; and for pure viscous materials, $\delta = 90°$.

E Predictive Model*

(1) Asphalt Institute Model

The mechanical behavior of an asphalt mix, which is a composite material, is primarily governed by the viscoelastic nature of the asphalt binder and the volumetric properties of the mixture. During the past few decades, researchers have developed empirical equations to convert common consistency parameters such as penetration and viscosity. Models are also available to convert viscosity data to modulus data.

The Asphalt Institute developed an empirical formula to predict the dynamic modulus of hot mix asphalt based on the mix properties, temperature and loading conditions. The formula, which is utilized in KENTACK 3.0, is shown in Eq. 2.11 (Huang and Wiczak, 1979):

$$E = 10^5 \times \beta_1$$

$$\beta_1 = \beta_3 + 0.000005\beta_2 - 0.00189\beta_2 f^{-1.1}$$

$$\beta_2 = \beta_4^{0.5} T^{\beta_5}$$

$$\beta_3 = 0.483V_b + 0.028829P_{200}f^{-0.1703} - 0.03476V_v + 0.070377\eta + 0.831757f^{-0.02774}$$

$$\beta_4 = 0.483V_b$$

$$\beta_5 = 1.3 + 0.49825 log f \qquad \text{(Eq. 2.11)}$$

where,

$\beta_1, \beta_2, \beta_3, \beta_4, \beta_5$ = temporary constants;

E = dynamic modulus of hot mix asphalt in psi;

T = temperature in °F;

f = load frequency in Hz;

P_{200} = aggregate passing no.200 sieve in %;

V_b = volume of bitumen in %;

V_v = volume of air vioids in %;

However, shortcomings of the previous Asphalt Institute model are that viscosity is extremely sensitive to temperature. Viscosity increases when temperature decreases. Thus, using a constant value for viscosity at 135°F may lead to an underestimation of the asphalt dynamic moduli. Furthermore, since Superpave has improved the performance of asphalt with new asphalt design and PG System for asphalt binders was used, the old asphalt dynamic modulus predictive model existing in KENTRACK 3.0 (the latest version released) may not be accurate enough to predict dynamic modulus of PG asphalt binders.

(2) Witczak E* Predictive Model

The Witczak E* predictive model is an empirical model based on volumetric mixture properties and binder characteristics. The initial Witczak E* predictive model was developed in 1972. It was based on non-linear polymonial regression of laboratory E* values. The early model was established from 29 mixtures with 87 total data points. Several revisions were made during the following 20 years. The current MEPDG uses the 1999 Witczak model developed for E* estimation. The 1999 Witczak model is developed from 205 laboratory mixtures including 171 unmodified asphalt binders and 34 modified binders that produced 2750 data points (Garcia and Thompson, 2007).

This model is capable of predicting mixture stiffness over a range of temperatures, rate of loading, and aging conditions from information that is readily available from material specifications, or the volumetric design of the mixture as shown in Eq. 2.12. This model can predict the dynamic modulus of mixture using both modified and conventional asphalt cements/binders (2002 Design Guide, 1999). Table 1 shows summary statistics for this equation.

$$\log E^* = -1.249937 + 0.02923\rho_{200} - 0.00167\left(\rho_{200}\right)^2$$
$$-0.002841\rho_4 - 0.058097V_a - 0.802208\frac{V_{beff}}{V_{beff}+V_a}$$
$$+\frac{3.971977 - 0.0021\rho_4 + 0.003958\rho_{38} - 0.000017\left(\rho_{38}\right)^2 + 0.00547\rho_{34}}{1+e^{-0.603313-0.31335\log(f)-0.393532 lof\,(\eta)}}$$

(Eq. 2.12)

where,

$|E^*|$ = dynamic modulus, 10^5 psi;

η = binder viscosity at the age and temperature of interest, 10^6 Poise;

f =loading frequency, Hz;

V_a = air of void content, %;

V_{beff} = effective binder content, % by volume;

ρ_{34} = cumulative % retained on 3/4 in (19mm) sieve;

ρ_{38} = cumulative % retained on 3/8 in (9.5mm) sieve;

ρ_4 =cumulative % retained on #4 (4.76 mm) sieve;

ρ_{200} = % passing #200 (0.075mm) sieve.

Table 2.1 Summary Statistics for the Witczak Dynamic Modulus Predictive Equation ((Garcia and Thompson, 2007))

Statistic	Value
Goodness of fit	$R^2 = 0.96$ $S_e/S_y = 0.24$
Data points	2750
Temperature range	0 to 130°F
Loading rates	0.1 to 25 Hz
Mixtures	205 Total 171 With unmodified asphalt binders 34 With modified binders
Binders	23 Total, 9 Unmodified, 14 Modified
Aggregates	39
Compaction methods	Kneading and gyratory
Specimen sizes	Cylindrical 4 in by 8 in or 2.75 in by 5.5 in

Viscosity Temperature Susceptibility Method (VTS)

Viscosity is used in the predictive equation model to describe the temperature effects and to express the shift factors. For un-aged binders, the viscosity at the temperature of interest can be determined from the ASTM viscosity temperature relationship as Eq. 2.13 (ASTM D 2493-01, 2009):

$$\log\log\eta = A + VTS\log T_R \qquad \text{(Eq. 2.13)}$$

where,

η = binder viscosity, cP;

A, VTS = regression parameters;

T_R = temperature, Rankine.

This linear relationship is characterized by its unique A and VTS parameters for the original asphalt cement/binder condition. It allows a continuous binder viscosity characterization over a wide range of temperature (Garcia and Thompson, 2007).

CHAPTER 3. THEORY

Trackbed Types

Three types of trackbeds are included in KENTRACK: All-Granular trackbed, Asphalt Underlayment trackbed and Combination trackbed.

The all-granular trackbed is a traditional trackbed which is composed of ballast, granular subballast, subgrade and bedrock from top to bottom.

In the asphalt underlayment trackbed, an asphalt layer is used in place of the subballast layer in the all granular trackbed. The asphalt underlayment trackbed has been widely accepted – not only because it reduces subgrade stresses, but it also serves as a waterproofing layer to control subgrade moisture contents and provides a high level of confinement for the ballast enhancing the shear strength of the ballast (Anderson and Rose, 2008), (Rose and Lees, 2008) (Rose and Bryson, 2009).

The combination trackbed has both an asphalt layer and a subballast layer, which is composed of ballast, asphalt, subballast, subgrade and bedrock. Subballast is considered as an improved subgrade.

Trackbed Model

For analysis, the track is divided into rails, springs, ties, and layered support systems, as shown in Figure 3.1.

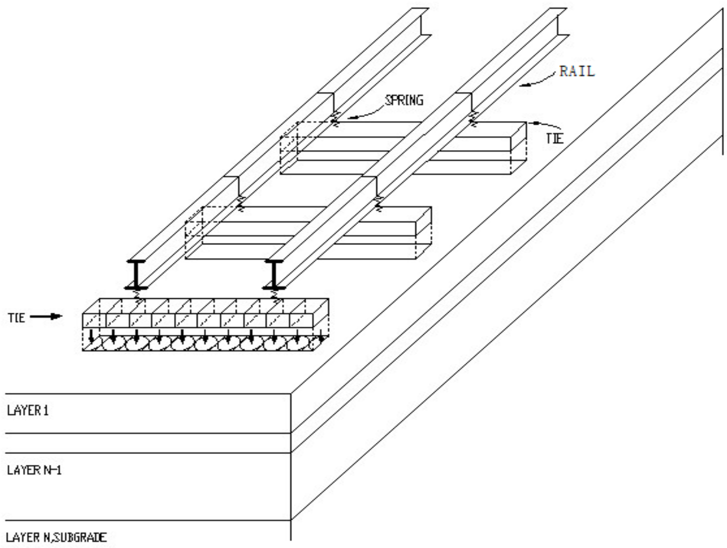


Figure 3.1 Trackbed Structural Model

Rails and ties, considered as beam elements, are orthogonal to each other. Figure 3.2 shows a beam element for a rail and a tie. The element stiffness matrix for the rail is shown in Eq. 3.1.

$$\frac{EI}{l^3}\begin{bmatrix} 12 & 6l & -12 & 6 \\ 6l & 4l^2 & -6l & 2l^2 \\ -12 & -6l & 12 & -6l \\ 6 & 2l^2 & -6l & 4l^2 \end{bmatrix}\begin{Bmatrix} w_i \\ \theta_{yi} \\ w_j \\ \theta_{yj} \end{Bmatrix} = \begin{Bmatrix} P_i \\ W_{yi} \\ P_j \\ W_{yj} \end{Bmatrix} \qquad \text{(Eq. 3.1)}$$

Where,

E = Young's modulus;

I = moment of inertia of beam;

l = the distance of beam between node i and j;

w_i = vertical deflection at node i;

θ_{yi} = rotation about y axis at node i;

P_i = vertical force applied at node i;

M_{yi} = moment about y axis at node i.

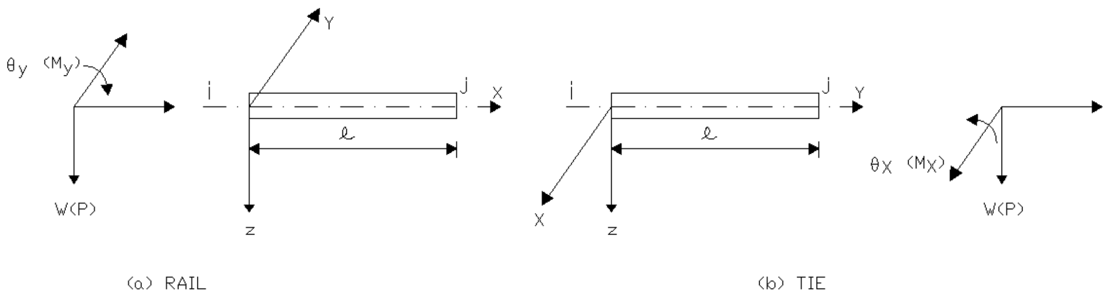


Figure 3.2 Beam Element for Rail and Tie

The same equation applies to tie except that subscript y is replaced by x.

Figure 3.3 shows the one dimensional element for a spring to connect rails and ties. The element stiffness matrix is expressed in Eq. 3.2.

$$\begin{bmatrix} C_s & -C_s \\ -C_s & C_s \end{bmatrix}\begin{Bmatrix} W_i \\ W_j \end{Bmatrix} = \begin{Bmatrix} P_i \\ P_j \end{Bmatrix} \qquad \text{(Eq. 3.2)}$$

where,

C_s = spring constant.

W_i, W_j = vertical deflection at Node i and Node j respectively.

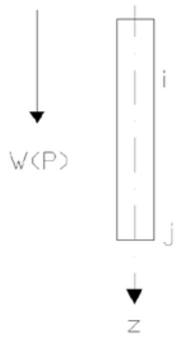


Figure 3.3 Spring Element

Based on the calculations, loads below ties can be determined. Burmister's multi-layered system theory (Burmister, 1945), can be applied to calculate stresses and strains in the trackbed. However, due to the necessity of the multi-layered theory, the loads are simplified as intent circular loads. By using the general equation of Burmister's multi-layered theory as shown in Eq. 2.4 in Chapter 2, the stresses in vertical, tangential and radial directions, the shear stresses, and the strains (displacements) in radial and vertical directions can be expressed and calculated. However, these values are not the actual stresses and displacements resulting from a uniform load q distributed over a circular area. To find the actual stresses and displacements under a uniform load distributed over a circular area, the Hankel transform method should be used. Then, the results obtained from the above equations can be converted to actual stresses and displacements by using Eq. 3.3:

$$R = \frac{qa}{H}\int_0^\infty \frac{R^*}{m} J_1\left(\frac{ma}{H}\right) dm \qquad \text{(Eq. 3.3)}$$

where,

R* = the stress or displacement due to the loading;

R = the stress or displacement due to load q;

J = Bessel function;

m = a parameter.

Therefore, the analysis of the multi-layered system can be summarized as follows:

1. Assign successive value of m, from 0 to a rather large positive number until R converges;

2. Determine the constants of integration according to the boundary conditions for each value of m;

3. Calculate R* by using these constants of integration;

4. Obtain R from the equation.

In the layered system, the foundation reactive forces are applied to the bottom of the ties. The overall equilibrium equations can be written as Eq. 3.4 (Huang, et al, 1984):

$$[K]\begin{Bmatrix}\delta_{rail}\\ \delta_{tie}\end{Bmatrix}=\begin{Bmatrix}F_{rail}\\ 0\end{Bmatrix}-\begin{Bmatrix}0\\ F_{tie}\end{Bmatrix} \qquad \text{(Eq. 3.4)}$$

To determine the reactive forces at the tie, F_{tie}, the vertical deflection at a point on the surface of the layered system due to a unit load over a circular area, applied at a given distance r from the point, is determined by the Burmister's layered theory and the flexibility matrix of the foundation is formed as Eq. 3.5 (Huang, et al, 1984).

$$[G]\begin{Bmatrix}P_1\\ \vdots\\ P_i\\ \vdots\\ P_n\end{Bmatrix}=[H]\begin{Bmatrix}W_1\\ \vdots\\ W_i\\ \vdots\\ W_n\end{Bmatrix} \qquad \text{(Eq. 3.5)}$$

where,

[G] = flexibility matrix of foundation;

P_i = vertical force at Node i;

W_i = vertical deflection at Node i;

Eq. 3.5 can be inverted to (Huang, et al, 1984):

$$F_{tie} = \begin{Bmatrix} P_1 \\ \vdots \\ P_i \\ \vdots \\ P_n \end{Bmatrix} = [H] \begin{Bmatrix} W_1 \\ \vdots \\ W_i \\ \vdots \\ W_j \end{Bmatrix} \quad \text{(Eq. 3.6)}$$

where,

F_{tie} = the reactive forces at the tie;

Substituting Eq. 3.6 into Eq. 3.4, a set of simultaneous equations is obtained, as shown in Eq. 3.7 (Huang, et al, 1984).

$$[K]\{\delta\} = \{F\} \quad \text{(Eq. 3.7)}$$

where,

[K] = overall stiffness of the structure;

$\{\delta\}$ = overall displacement;

$\{F\}$ =overall reactive forces.

Superposition Principle

Because KENTRACK is a linear elastic model with the layered foundation extended infinitely in the horizontal direction, the superposition principle can be applied. A standard case with a single wheel load P is always analyzed first. Stresses, strains and deflections in the track system due to multiple wheel loads are obtained by superposing the results of the single wheel load. This is illustrated in Figure 3.4 The variable S_i is the deflection in the i^{th} tie due to load P. The deflections in the ties due to each of loads P_1 to P_4 are shown in the figure. After superposing, the deflection at the first tie can be calculated as Eq. 3.8 (Huang, et al, 1984).

$$S_1 = S_2 \frac{P_1}{P} + S_4 \frac{P_2}{P} \quad \text{(Eq. 3.8)}$$

Figure 3.4 shows that the load P is distributed over four ties, but it is reasonable to assume that the wheel load P is distributed over six ties and it gives reliable results for rails and ties.

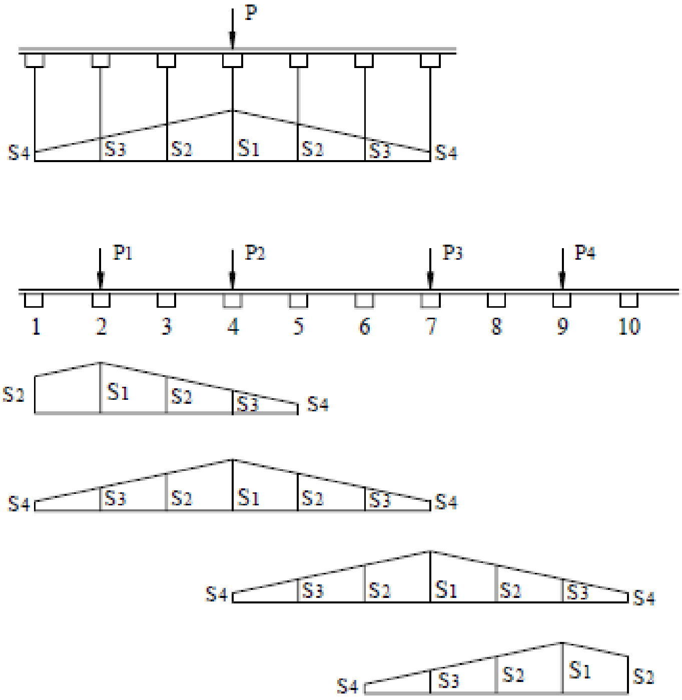


Figure 3.4 Superposition of Multiple Loads

Material Properties

Ballast in newly constructed trackbeds behaves non-linearly while it behaves linearly when considered in an aged trackbed since it is well compacted. The elastic modulus of nonlinear materials is determined as Eq. 3.9, (Huang, et al, 1984):

$$E = K_1 \theta^{K_2} \qquad \text{(Eq. 3.9)}$$

Where, E= resilient modulus;

θ=bulk stress, or the sum of three principal stresses including geostatic stresses;

K_1, K_2=regression constants reflecting material properties.

A lateral stress ratio of K_0 must be specified to compute the geostatic bulk stress. When $K_2 = 0$, the material behaves linearly. Because of its small effect on stress and strain, a constant Poisson's ratio, independent of the state of stresses, is assumed for each layer.

Witczak E* predictive model was incorporated into KENTRACK 4.0 to calculate asphalt dynamic modulus. The Witczak E* predictive equation is expressed as Eq. 3.10 (Andrei, et al., 1999).

$$\begin{aligned}\log E^* = &-1.249937 + 0.02923\rho_{200} - 0.00167(\rho_{200})^2 \\ &-0.002841\rho_4 - 0.058097V_a - 0.802208\frac{V_{beff}}{V_{beff}+V_a} \\ &+\frac{3.971977 - 0.0021\rho_4 + 0.003958\rho_{38} - 0.000017(\rho_{38})^2 + 0.00547\rho_{34}}{1+e^{-0.603313-0.31335\log(f)-0.393532 lof(\eta)}}\end{aligned} \qquad \text{(Eq. 3.10)}$$

where,

$|E^*|$ = dynamic modulus, 10^5 psi;

η = binder viscosity at the age and temperature of interest, 10^6 Poise;

f =loading frequency, Hz;

V_a = air of void content, %;

V_{beff} = effective binder content, % by volume;

ρ_{34} = cumulative % retained on 3/4 in (19mm) sieve;

ρ_{38} = cumulative % retained on 3/8 in (9.5mm) sieve;

ρ_4 =cumulative % retained on #4 (4.76 mm) sieve;

ρ_{200} = % passing #200 (0.075mm) sieve.

Viscosity, as one of the parameters in Eq. 3.10, could be either input by users or determined by A-VTS relationship if data for viscosity is unavailable (ASTM D

2493-01, 2009). The relationship between viscosity and temperature is established by Eq. 3.11.

$$\log \log \eta = A + VTS \log T_R \qquad \text{(Eq. 3.11)}$$

where,

η = binder viscosity, cP;

A, VTS = regression parameters;

T_R = temperature, $\circ$ Rankine.

Typical grades of asphalt cements and binders that have been used in trackbeds are AC-10, AC-20, AC-40, PG 64-22, PG 70-28 and PG 76-34. For each type of asphalt cement/binder, A and VTS values are shown in Table 1:

Table 3.1 Relationship between Asphalt Binder Grade and Viscosity Parameters (NCHRP 1-37A, 2004)

Grade	AC-10	AC-20	AC-40	PG 64-22	PG 70-28	PG 76-34
A	11.0134	10.7709	10.5338	10.9800	9.7150	8.5320
VTS	-3.6954	-3.6017	-3.5047	-3.6800	-3.2170	-2.7850

Subballast and subgrade were always considered as linear elastic materials. The bedrock at the bottom of the trackbed, was assumed incompressible with a Poisson's ratio of 0.5.

Damage Analysis

Two types of damage analysis are performed in KENTRACK, tensile strains at the bottom of asphalt layer and compressive stresses at the top of subgrade. The former controls fatigue cracking and the latter controls excessive permanent deformation.

The fatigue cracking of asphalt is governed by the horizontal tensile strain at the bottom of asphalt layer. The relationship between tensile strain and the allowable number of load repetitions in asphalt was expressed in Eq. 3.12 (Asphalt Institute, 1982):

$$N_a = 0.0795\varepsilon_t^{-3.291}E_a^{-0.853} \qquad \text{(Eq. 3.12)}$$

where,

N_a = the allowable number of load repetitions in asphalt;

ε_t = horizontal tensile strain at the bottom of asphalt;

E_a = dynamic modulus of asphalt, psi.

The excessive permanent deformation is governed by the compressive stress at the top of subgrade. The relationship between compressive stress and allowable number of load repetitions in subgrade is expressed in Eq. 3.13 (Huang, et al., 1984):

$$N_d = 4.837\times10^{-5}\sigma_c^{-3.734}E_s^{3.583} \qquad \text{(Eq. 3.13)}$$

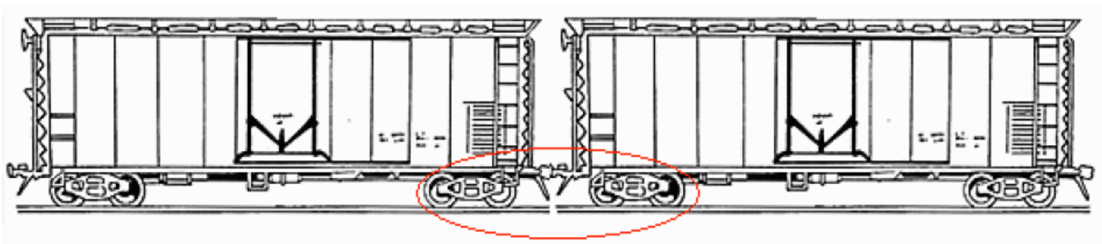
N_d = the allowable number of load repetitions in subgrade;

σ_c = compressive stress at the top of subgrade, psi;

E_c = subgrade modulus, psi.

Eq. 3.12 and Eq. 3.13 were developed for highway pavement where loadings and environments were quite different from railroad. Load pressures and temperature extremes on highway pavements are more severe than on trackbeds, thus the life prediction for trackbeds tends to be conservative when using equations developed for highway pavements.

The passage of one car of a train is equivalent to one load repetition. The predicted number of repetitions varies with the traffic that the trackbed is subjected. Assume the predicted number of load repetitions each season is 200,000 and wheel load is 36,000 lbs, the traffic would be 28.6 MGT. An illustration and calculations for the load predictions are shown in Figure 3.5.



Wheel load	= 36,000 lb/wheel
For one car the total weight	= 36,000 lb/wheel x 8 = 286,000 lb/rep
	= 143 ton/rep
The number of repetitions assumed per year	= 200,000 rep/yr
The traffic per year	= 200,000 rep/yr x 143 ton/rep
	= 28,600,000 GT/yr
	= 28.6 MGT/yr

Figure 3.5 Million Gross Tons per Year Calculation

With loading conditions and material properties varying throughout the year, damage may be analyzed seasonally for one year to determine the service life. Based on the maximum stress or strain in a given period, the allowable number of load repetitions during that period is obtained from Eq. 3.14 for each mode of distress. The repetition ratio, which is the ratio between the predicted and the allowable numbers of repetitions, is computed for each period and summed to obtain the repetition ratio for the entire year. The design life is the reciprocal of the repetition ratio. If a year is divided into four seasons, the design life for each distress mode can be written as:

$$L = \frac{1}{\sum_{i=1}^{4} \frac{N_p}{N_a}} \qquad \text{(Eq. 3.14)}$$

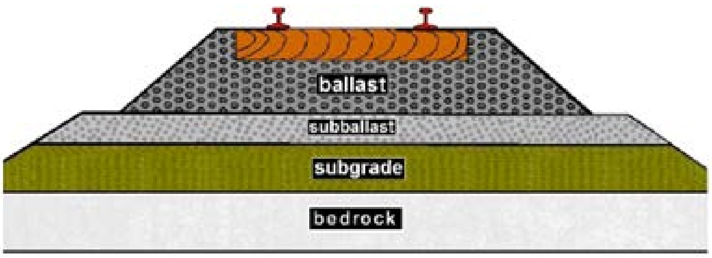
where,

L = service life in a specific layer, years;

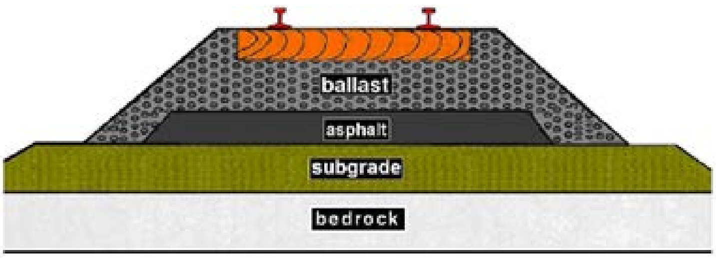
N_p = predicted number of load repetitions each season.

CHAPTER 4. METHODOLOGY

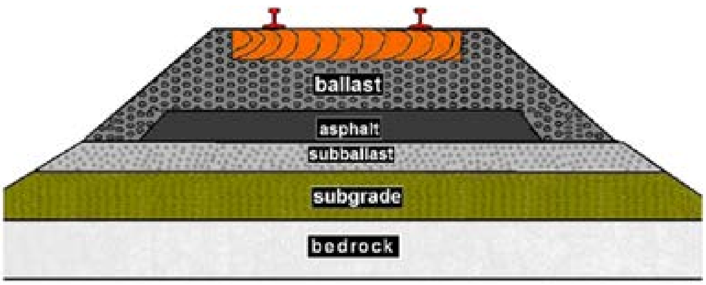
In order to develop a rational structural design method for railroad trackbeds, it is necessary to understand the effects of the various track components on trackbed performance. These factors include axle load, rail size, tie type and configuration, ballast modulus, ballast layer thickness, asphalt properties, asphalt layer thickness, subballast, and subgrade modulus. The traditional all-granular railroad trackbed, shown in Figure 4.1(a), was also evaluated for performance comparisons with the asphalt underlayment trackbeds shown in Figure 4.1(b) and 4.1(c). The components and factors used in both of the models are the values for a typical trackbed.



(a). All-Granular Trackbed



(b). Asphalt Underlayment Trackbed



(c). Combination Trackbed

Figure 4.1 Three Types of Trackbeds

Standard scenarios for typical designs, using asphalt binders PG64-22, are established and the series of default parameters are utilized, as shown in Tables 4.1 and 4.2. Table 4.1 presents the standard design parameters for All-Granular, Asphalt Underlayment, and Combination trackbeds used in the United States. The asphalt modulus varies depending on the season. The moduli in each season is calculated by Witczak E* predictive model. Table 4.2 shows the detail information about the track model and properties of the asphalt binders.

Table 4.3 records all the parameters that are varied in the sensitivity analysis. When analyzing the three types of trackbeds, two aspects are of primary interest. One aspect is to discuss the effect of varying parameters such as rail size, axle load, subgrade modulus, asphalt properties, asphalt binder grades, ballast thickness and modulus, and the relationship between two parameters (i.e., rail size and axle load, subgrade modulus and axle load, ballast modulus and subgrade modulus, etc.) which can help to explain the behavior of the trackbeds. The other aspect is the evaluation of the significance of the parameters on the performance of trackbeds. The parameters that may significantly affect the performance of trackbeds should be stressed when a structural design for a trackbed is evaluated. More details of analysis of the variation of parameters will be discussed in Chapter 5.

Table 4.1 Layer Properties for Standard Case

Layers	All-Granular trackbed		Asphalt Underlayment Trackbed			Combination Trackbed		
	Thick-ness (in)	Modulus (psi)	Thick-ness (in)	Modulus (psi)		Thick-ness (in)	Modulus (psi)	
Ballast	10	18,000	8	18,000		8	18,000	
Asphalt	n/a		6	Spring	1.86E+06	6	Spring	1.86E+06
				Summer	9.10E+05		Summer	9.10E+05
				Fall	3.39E+06		Fall	3.39E+06
				Winter	4.84E+06		Winter	4.84E+06
Subballast	4	31,000	N/A	N/A		4	31,000	
Subgrade	200	12,000	200	12,000		200	12,000	
Bedrock	N/A	1.00E+19	N/A	1.00E+19		N/A	1.00E+19	

Table 4.2 Standard Input Parameters

Rail Type	RE 132
Type of Tie	Wood Tie (102 in * 7 in * 9 in)
Tie Spacing (in)	20
Temperature for Asphalt (°F)	Spring 50
	Summer 67
	Fall 33
	Winter 20
Wheel Load (lbs)	2 @ 36,000
% Passing #200 Sieve	4
Cumulative % Retained on 3/4 Sieve	16
Cumulative % Retained on 3/8 Sieve	40
Cumulative % Retained on #4 Sieve	56
% Air Voids	4
% Effective Binder Content by Volume	10
Loading Frequency (Hz)	1

Table 4.3 Details of Varied Parameters in Sensitivity Analysis

Tie Type	Wood Tie (20 in spacing) Concrete Tie (25 in spacing)
Rail Size	RE 100, RE 115, RE 132, RE 140
Axle Load (tons)	33, 36, 39
Subgrade Modulus (psi)	6,000, 12,000, 18,000, 24,000
Ballast Modulus (psi)	12,000, 18,000, 24,000, 3,0000
Ballast Thickness (in)	6, 8, 10, 12
Asphalt Thickness (in)	2, 4, 6, 8
Aggregate Passing #200 Sieve (%)	0, 2, 4, 6, 8, 10, 12
Aggregate Retained on #4 Sieve (%)	30, 40, 50, 60, 70
Aggregate Retained on #3/4 Sieve (%)	0, 10, 20, 30
Aggregate Retained on #3/8 Sieve (%)	20, 30, 40, 50, 60
Air Voids (%)	0, 2, 4, 6, 9
Effective Binder Content (%) by Volume	5, 10, 15, 20, 25
Viscosity (10^6 Poise)	10, 100, 1,000, 10,000, 100,000
Asphalt Binder Grade	PG 64-22, PG 64-28, PG 64-34, PG 70-28, PG 76-28

The critical outputs for the standard all-granular trackbeds, asphalt underlayment trackbeds and combination trackbeds under standard scenarios are listed in Table 4.4 including subgrade compressive stresses, subgrade service lives, asphalt layer tensile strains and asphalt layer service lives. The advantage of trackbeds with asphalt can be noted. For example, the all-granular trackbed provides the highest calculated subgrade compressive stress and shortest predicted service life. The combination trackbed has the lowest predicted subgrade compressive stress and asphalt tensile strain and longest predicted service lives. This type of output data is described in detail in Chapter 5.

Table 4.4 Critical Outputs for All-Granular, Asphalt Underlayment, and Combination Trackbeds with Wood Ties

Trackbed Type	Subgrade Compressive Stress (psi)	Subgrade Service Life (yrs)	Asphalt Tensile Strain	Asphalt Service Life (yrs)
All-Granular Trackbed	13.52	6.0	n/a	n/a
Asphalt Underlayment Trackbed	10.84	21.4	1.48E-04	25.0
Combination Trackbed	9.82	28.5	1.29E-04	34.0

The failure criterion of subgrade and asphalt used in KENTRACK program is based on highway pavement performance. The failure of highway pavements is governed by either the permanent deformation of the subgrade or fatigue cracking in the asphalt layer. However, the environmental conditions in asphalt trackbeds are less severe than pavements.

For the subgrade, the stress level in the asphalt trackbed is lower as a result of stress distribution in the asphalt layer. Further, the settlement of a trackbed normally results from subgrade weakening or softening due to water infiltrating the structure, which is very common for the all-granular trackbeds. The asphalt layer provides an impermeable layer to protect the subgrade from water infiltration from above, thus the asphalt trackbed is less likely to be adversely affected by weaken or soften subgrade. Moreover, the subgrade in a highway pavement is normally subjected to varying moisture contents, but this is not the case in railway trackbed subgrades.

For the asphalt layer, the temperature data used to perform viscoelastic analysis in asphalt trackbeds was obtained on highway asphalt pavements, where the asphalt surface was exposed to the atmosphere and sunlight. Under these conditions, asphalt binders tend to harden and oxidize fairly rapidly, which will adversely affect life of asphalt binders. This will cause the asphalt layer to crack and deteriorate fairly rapidly, resulting in shorter life expectancy. The asphalt layer in the trackbed underlayment is submerged, thus insulated, by the thickness of the tie and ballast, or about 15 to19 inches. The oxygen level is much lower and the temperature extremes in the asphalt underlayment layer are much less in the insulated environment than on a highway pavement surface where the asphalt layer is exposed to larger temperature extremes from winter to summer. The temperature extremes and insulation are different for highway and railroad applications.

CHAPTER 5. SENSIVITY ANALYSIS

Sensitivity analysis evaluates how the uncertainty in the output of a mathematical model or system can be apportioned to different sources of uncertainty in its inputs (Saltelli, et al., 2008).

Sensitivity analysis can be useful for a range of purposes, including (Pannell, 1997):

1. Testing the robustness of the results of a model or system in the presence of uncertainty.

2. Increased understanding of the relationships between input and output variables in a system or model.

3. Uncertainty reduction: identifying model inputs that cause significant uncertainty in the output and therefore should be the focus of attention if the robustness is to be increased (perhaps by further research).

4. Searching for errors in the model (by encountering unexpected relationships between inputs and outputs).

5. Model simplification – fixing model inputs that have no effect on the output, or identifying and removing redundant parts of the model structure.

6. Enhancing communication from modelers to decision makers (e.g. by making recommendations more credible, understandable, compelling or persuasive).

7. Finding regions in the space of input factors for which the model output is either maximum or minimum or meets some optimum criterion.

The KENTRACK model is defined by a series of equations, input variables and parameters aimed at characterizing the track structure being evaluated. Increasingly, the model is highly complex, and as a result the input/output relationships may be poorly understood. Therefore, the model can be viewed as a black box. Parameters, including rail size, axle load, subgrade modulus, asphalt properties and thickness, ballast modulus and thickness, etc. are investigated as factors of sensitivity analysis.

Effect of Rail Size and Axle Load

The unit weight of a rail per length is an important factor in determining rail strength and acceptable axle loads. Larger rails are desirable for heavier wheel loads and tonnages. On a given railroad, one rail size is not generally used for all purposes. A typical application observed on the NS in Atlanta, was RE 136 rail for the main line. Once clear of the turnout into an industry, the rail was reduced to RE 115 for one rail length, and then further reduced to 85-lb to serve the industry. Yard tracks are much the same, typically of a smaller rail size than the main line associated with it.

In this analysis, a 36-ton axle load is considered as a standard design value. However, in the United States, a 33-ton axle load is also very common on many freight railroad lines, and a 39-ton axle load is undergoing research and testing. For the rail evaluation, four different rail sizes are used: RE100, RE115, RE132, and RE140. The axle load is varied from 33 to 39 tons. Three types of trackbeds are evaluated. A constant subgrade of 12,000 psi is used. Figures 5.1-5.3 show the effect of four different rail sizes and axle load variations from 33 ton to 39 ton on subgrade compressive stress in the all-granular trackbed, asphalt underlayment trackbed, and combination trackbed respectively. Figures 5.4 and 5.5 show the effect of varying rail sizes and axle load on asphalt tensile strains for the three types of trackbeds. Table 5.2 shows the effect of rail size and axle load on the predicted service lives of associated layers.

It is obvious for all three types of trackbeds that a heavier axle load results in greater subgrade compressive stresses, as shown in Figures 5.1-5.3, and greater asphalt tensile strains due to large deformations of asphalt layers, as shown in Figures 5.4-5.5. As respected, large subgrade stresses and asphalt tensile strains reduce the service lives of the associated layers, as shown in Table 5.2. Consider in an asphalt underlayment trackbed, the subgrade compressive stress and asphalt tensile strain are increased by 16 percent while the service lives are reduced by around 40 percent when axle load increases from 33 tons to 39 tons for RE 132.

Therefore, axle loads have a significant effect on the service lives of trackbeds. Heavy wheel loads and tonnage require a strong trackbed foundation to support it due to increases in subgrade compressive stress and asphalt tensile strain. Meanwhile, controlling the magnitude of axle load is beneficial for the trackbed service life.

The rail sizes have a positive effect on the track performance behavior. Heavier rails with larger sizes reduce subgrade compressive stresses and asphalt tensile strains, increasing the service lives. It is interesting to note the effect of heavy axle loads on large rails is identical to the effect of small axle loads on light rails. For example, the asphalt service life in the asphalt underlayment trackbed under a 33-ton axle load on RE100 (24.4 years) is similar to that under a 36-ton axle load on RE 132. Also, the subgrade service life in the asphalt underlayment under a 36-ton axle load on RE 100 (23.4 years) is close to that under a 39-ton load on RE 139. Therefore, a conclusion could be made that large rail size is desirable for heavy main lines with greater axle loads. Large size rail is used for heavy main lines.

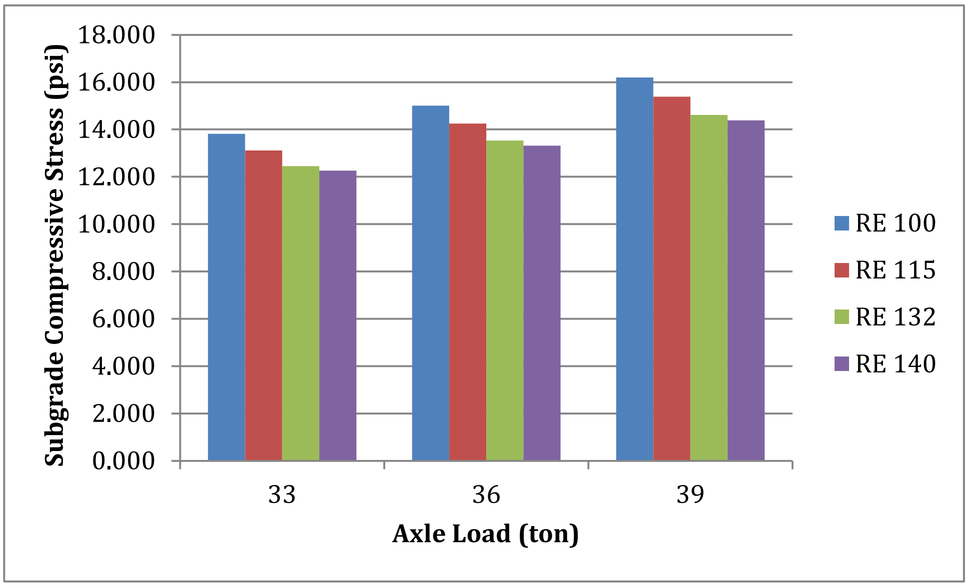


Figure 5.1 Effect of Rail Size and Axle Load on Subgrade Compressive Stress in the All-Granular Trackbed

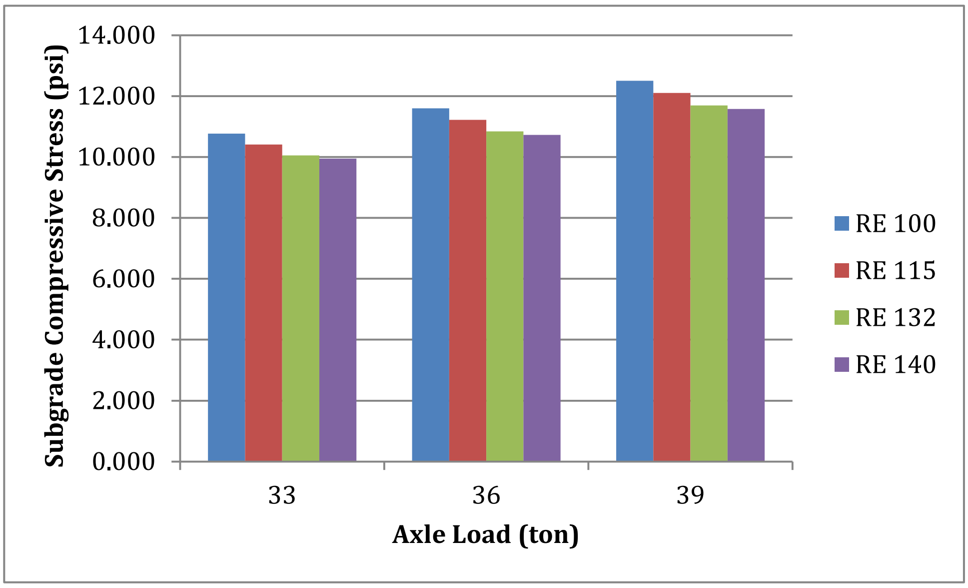


Figure 5.2 Effect of Rail Size and Axle Load on Subgrade Compressive Stress in the Asphalt Underlayment Trackbed

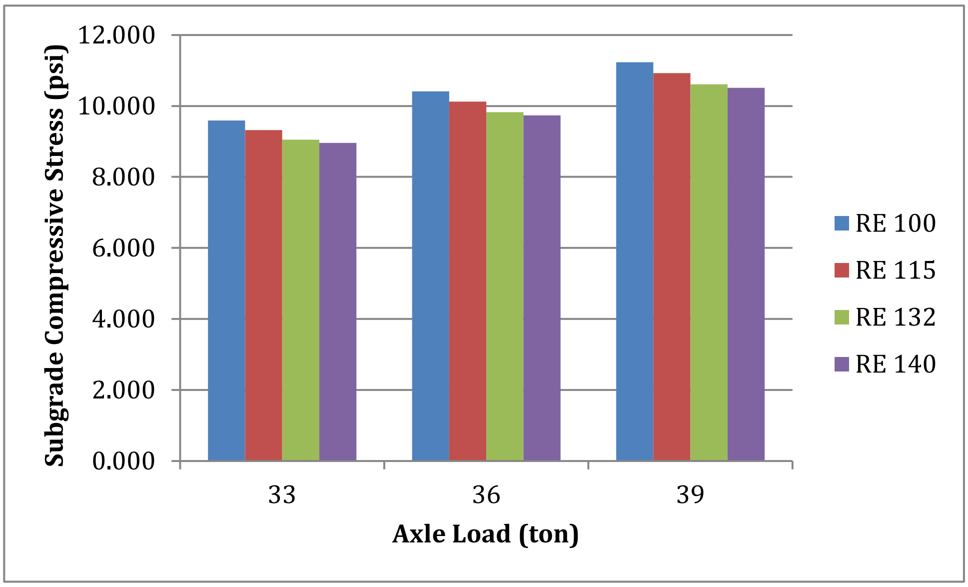


Figure 5.3 Effect of Rail Size and Axle Load on Subgrade Compressive Stress in the Combination Trackbed

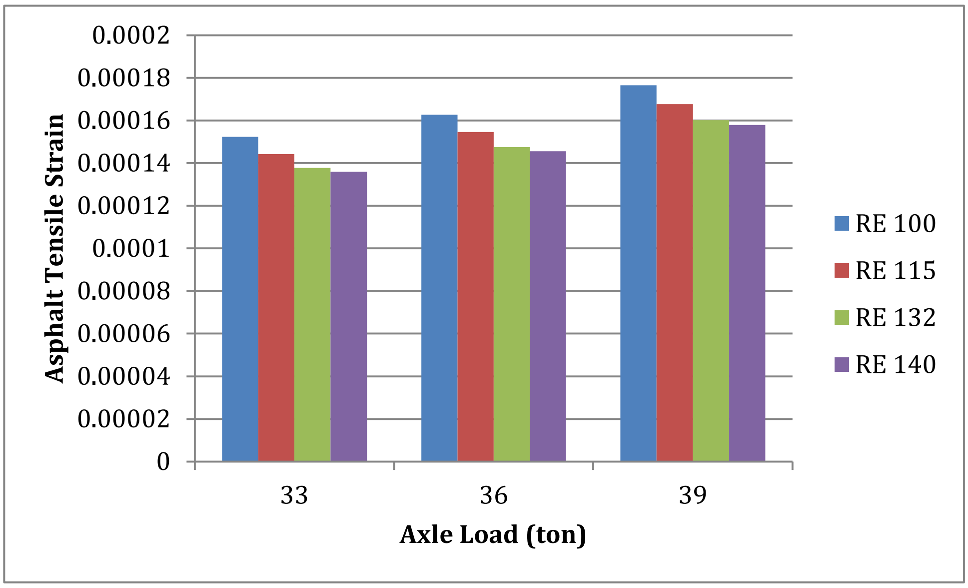


Figure 5.4 Effect of Rail Size and Axle Load on Asphalt Tensile Strain in the Asphalt Underlayment Trackbed

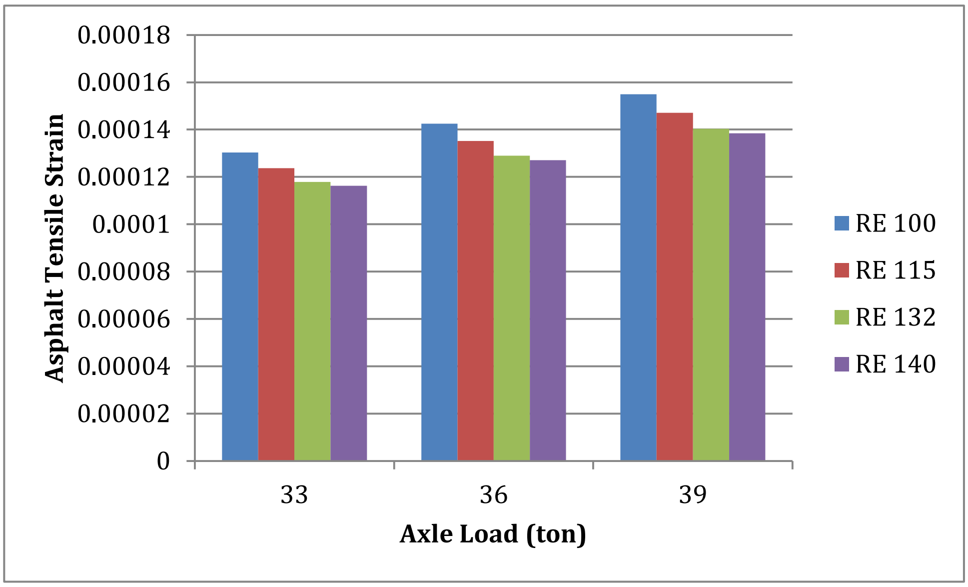


Figure 5.5 Effect of Rail Size and Axle Load on Asphalt Tensile Strain in the Combination Trackbed

Table 5.1 Effect of Rail Size and Axle Load on Subgrade Compressive Stresses and Asphalt Tensile Strains

Rail Size	Axle Load (ton)	All-Granular Trackbed	Asphalt Underlayment Trackbed		Combination Trackbed	
		Subgrade Stress (psi)	Subgrade Stress (psi)	Asphalt Strain	Subgrade Stress (psi)	Asphalt Strain
RE 100	33	13.82	10.76	0.000152	9.59	0.000130
	36	15.01	11.60	0.000163	10.41	0.000142
	39	16.19	12.50	0.000177	11.23	0.000155
RE 115	33	13.11	10.41	0.000144	9.32	0.000124
	36	14.25	11.22	0.000155	10.12	0.000135
	39	15.38	12.10	0.000168	10.92	0.000147
RE 132	33	12.45	10.06	0.000138	9.05	0.000118
	36	13.54	10.84	0.000148	9.82	0.000129
	39	14.61	11.70	0.000160	10.61	0.000140
RE 140	33	12.26	9.95	0.000136	8.96	0.000116
	36	13.32	10.73	0.000145	9.73	0.000127
	39	14.38	11.57	0.000158	10.51	0.000138

Table 5.2 Effect of Rail Size and Axle Load on Service Lives of Subgrade and Asphalt Layers

Rail Size	Axle Load (ton)	All-Granular Trackbed	Asphalt Underlayment Trackbed		Combination Trackbed	
		Subgrade Life (yrs)	Subgrade Life (yrs)	Asphalt Life (yrs)	Subgrade Life (yrs)	Asphalt Life (yrs)
RE 100	33	5.5	23.4	24.4	32.5	34.0
	36	4.1	17.3	18.7	23.8	25.5
	39	3.1	13.1	14.3	17.9	19.5
RE 115	33	6.7	25.9	28.5	35.5	39.6
	36	4.9	19.2	21.8	26.0	29.6
	39	3.7	14.5	16.7	19.6	22.6
RE 132	33	8.1	28.9	32.7	39.0	45.5
	36	6.0	21.4	25.0	28.5	34.0
	39	4.5	16.1	19.1	21.4	25.9
RE 140	33	8.6	29.9	34.1	40.1	47.5
	36	6.3	22.1	26.1	29.4	35.4
	39	4.7	16.6	19.9	22.1	27.0

Effect of Asphalt Properties and Thickness

Sensitivity Analysis of the E Model*

The important parameter relative to evaluating asphalt properties is asphalt dynamic modulus. Properties of asphalt binders and mixes are incorporated into Wictzak predictive E* model to determine asphalt dynamic moduli. A sensitivity analysis is an integral step in model evaluation and validation (Bari and Witczak, 2006). In the sensitivity analysis, the maximum, minimum, and average values of each basic predictor variable at specific combinations of temperature and loading frequency are summarized.

It is noteworthy that variables related to aggregate gradation $(\rho_{200}, \rho_{4,} \rho_{38}, \rho_{34})$, mix volumetrics -- volume of air voids (V_a) and volume of effective bitumen (V_{beff}), and viscosity have the most influence on the E* stiffness of asphalt mixes. Figure 5.6 shows the best-fit trend line plots of both average observed E* data and predicted E* data versus aggregate gradation ($\rho_{200}, \rho_{4,} \rho_{38}, \rho_{34}$) for the full range of each variable at f=1Hz and T=20°F, 33°F, 50°F, and 67°F. Similarly, Figure 5.7 and Figure 5.8 show constructed plots for the mix volumetrics (V_a, and V_{beff}), and binder stiffness respectively. The E* are affected even more greatly at a high temperature than a low temperature. In Figure 5.6(a), the E* tends to increase when the percentage of aggregate passing #200 sieve increases. In Figure 5.6, the x-axis values indicate the percentage of aggregate retained on a specific sieve. The E* has a peak value at around 2 percentage of air voids by volume, as shown in Figure 5.7. It is evident that the E* increases with the increase in asphalt viscosity due to greater stiffness, as shown in Figure 5.8 (Bari and Witczak, 2006).

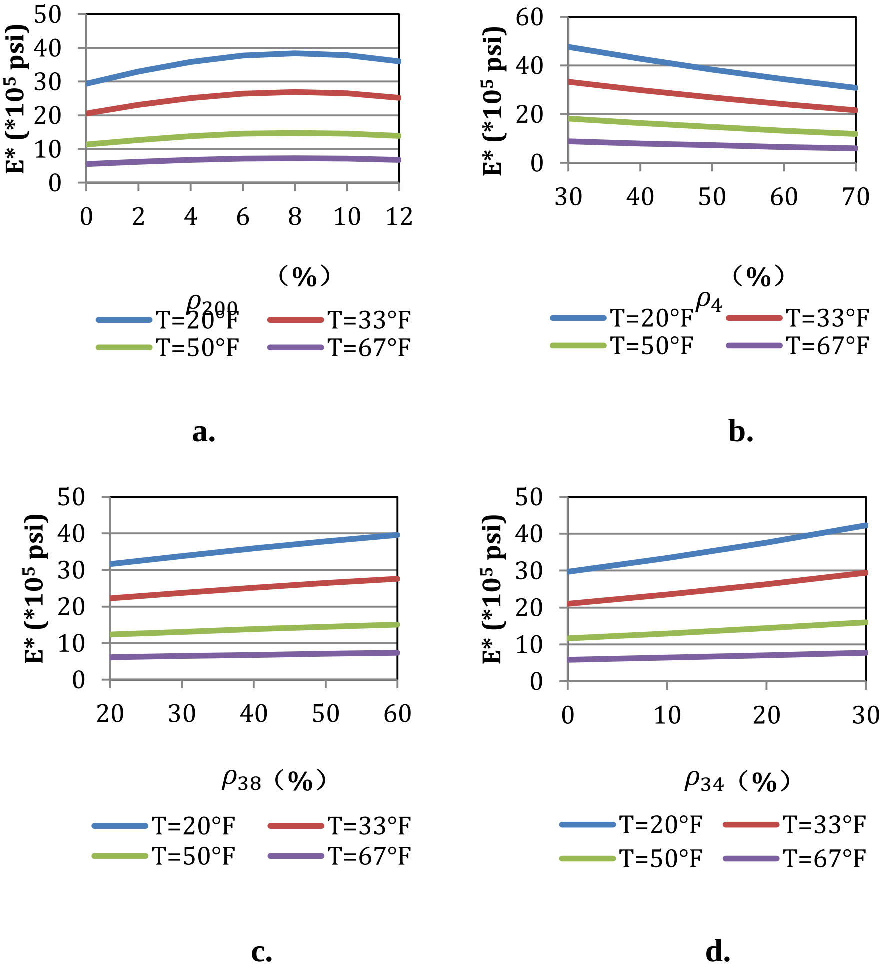


Figure 5.6 E* Model Sensitivity to Aggregate Gradation

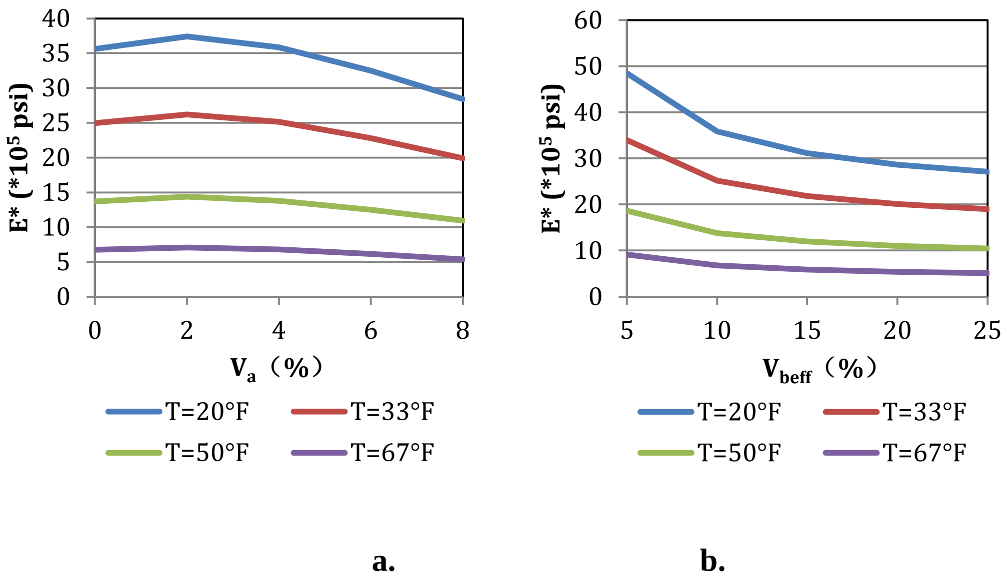


Figure 5.7 E* Model Sensitivity to Mix Volumetrics

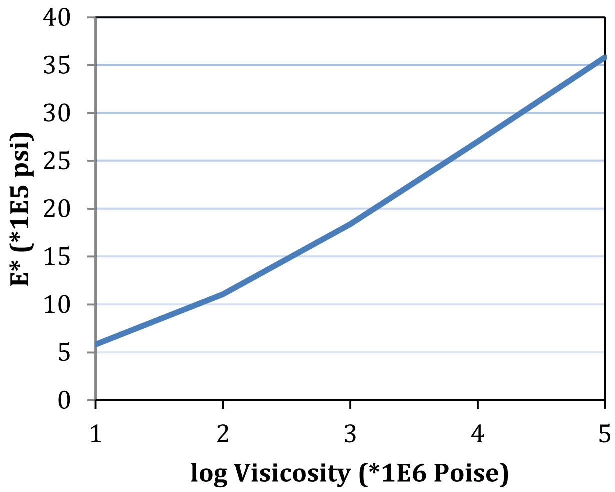


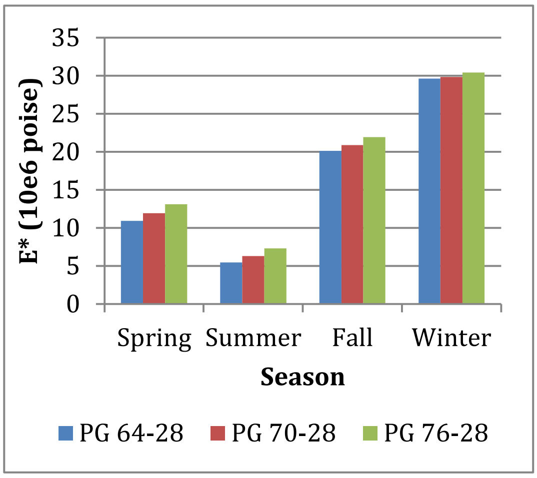
Figure 5.8 E* Model Sensitivity to Binder Stiffness

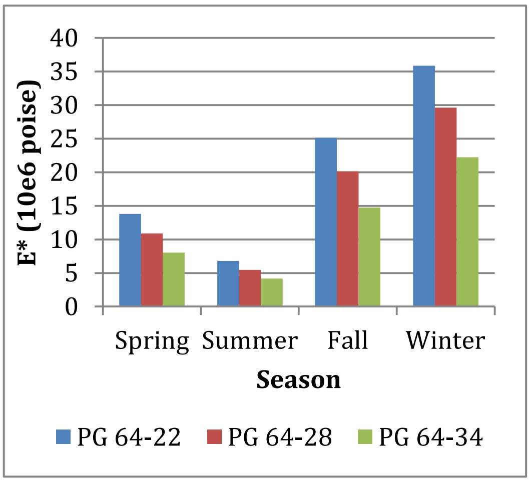
Effect of Asphalt Binder Grade

Asphalt binders are graded based on their physical properties. An asphalt binder's physical properties directly describe how it will perform as a constituent in an asphalt mixture. The PG grading system is based on climate, so the grade notation consists two portions: high and low service temperature. The major concern for high temperature performance is rutting, which typically takes time to develop. Therefore, an average of 7-day maximum pavement temperature is used to describe the high temperature climate. For the low temperature consideration, thermal cracking can happen during one really cold night; therefore the minimum pavement temperature experienced is used for describing the low temperature climate. For both high and low temperature grades, PG grades are graded in increments of 6°F. The average seven day maximum pavement temperature typically ranges from 46°F to 82°F, and minimum pavement temperature typically ranges from -46°F to -10°F. A binder identified as PG 64-22 must meet performance criteria at an average seven day maximum pavement temperature of 64°F and also at a minimum pavement temperature of -22°F.

To evaluate changes of binder grades on E*, the upper and lower grades of the binders are changed. The upper grade represents the highest temperature in which the binder can operate, and it is mainly consider for rutting. On the other hand, the lower grade represents the lowest temperature a binder can operate, and is mainly considered for thermal cracking. Therefore, it is expected that the stiffness of a higher upper grade binder should be higher than a lower upper grade. E* results for PG 64-28, PG 70-28 and PG 76-28, as presented in Figure 5.9(a), show that at high temperatures, during summer for example, the higher binder grade yielded higher E* values, and the binder yielded nearly the same values at low temperature.

Figure 5.9(b) represents E* results for PG 64-22, PG 64-28 and PG 64-34 with low temperature binder grades. It is speculated at higher temperatures, E* values will be similar since the upper grade has the same. E* values at low temperature, such as the temperature experienced during winter, are anticipated to vary due to differing low temperature binder grades. Results show E* values vary at high temperatures with the same upper binder grade and are similar at low temperatures with varying lower binder grades (Adbo, A.A., et al, 2009).





a. Changes in Upper Binder Grade **b. Changes in Lower Binder Grade**

Figure 5.9 E* with Different Asphalt Binders

Table 5.3 shows calculated results of asphalt underlayment and combination trackbeds with various asphalt binders by KENTRACK program. Increasing the upper grade brings decreases in subgrade compressive stresses and asphalt tensile strains in both trackbeds that utilize asphalt. As expected, the service lives of subgrade and asphalt layers are increased. These changing trends are opposite to varying the lower grade. When the lower grade increases, increases in subgrade compressive stresses and asphalt tensile strains lead to a reduction in the subgrade and asphalt tensile strains lead to a reduction in the subgrade and asphalt service lives. Also, it is should be noted that varying the lower asphalt binder grade has a more significant effect on the service lives than varying the upper grade. With improvement of chemical and physical properties of asphalt binders, new asphalt PG binders have become less sensitive to the high temperature than old AC asphalt cements. The upper grade of asphalt binders controls the highest temperature that asphalt can operate, therefore, even if the upper grade increases, the asphalt performance, such as viscosity, dynamic modulus, etc., would not vary significantly in a trackbed environment.

Table 5.3 Effect of Asphalt Binders on Asphalt Underlayment and Combination Trackbeds

Asphalt Binders	Asphalt Underlayment Trackbed				Combination Trackbed			
	Subgrade Compressive Stress (psi)	Subgrade Life (yrs)	Asphalt Tensile Strain	Asphalt Life (yrs)	Subgrade Compressive Stress (psi)	Subgrade Life (yrs)	Asphalt Tensile Strain	Asphalt Life (yrs)
PG 64-28	11.039	19.4	1.59 E-04	22.9	9.949	26.4	1.37 E-04	32.8
PG 70-28	10.847	20.3	1.50 E-04	23.9	9.825	27.3	1.31 E-04	33.0
PG 76-28	10.723	21.1	1.44 E-04	24.5	9.706	28.3	1.25 E-04	33.9
PG 64-22	10.410	26.0	1.32 E-04	29.9	9.500	33.3	1.16 E-04	39.8
PG 64-28	11.039	19.4	1.59 E-04	22.9	9.949	26.4	1.37 E-04	32.8
PG 64-34	11.330	16.7	1.76 E-04	19.8	10.203	23.3	1.49 E-04	29.7

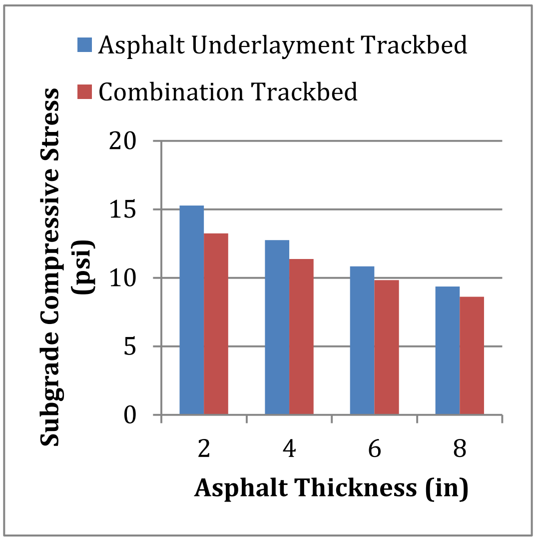
Effect of Asphalt Dynamic Modulus (E)*

The predicted service life of asphalt layers is determined by Eq. 3.10 in Chapter 3 as a function of asphalt dynamic modulus and asphalt tensile strain. Higher asphalt dynamic modulus values lead to lower asphalt tensile strains and longer asphalt service lives. This is shown in Figure 5.9 and Table 5.2, where the asphalt binder with higher dynamic moduli produces a smaller asphalt tensile strain and greater service life of asphalt layer. In Figure 5.9, when the upper grade of asphalt binders increases or the lower grade decreases, the asphalt dynamic modulus is increased, thus, the asphalt tensile strains decrease and asphalt predicted service lives increase. As asphalt layers stiffen, the asphalt dynamic modulus becomes large, reducing the asphalt tensile strains due to less amounts of deformation in the asphalt layer. Considering increased values of asphalt tensile strain and dynamic modulus with Eq. 3.12 in Chapter 3 indicates an increase in the allowable load repetitions for asphalt layer. The increased allowable load repetitions can be converted to longer predicted asphalt service life in Eq. 3.14.

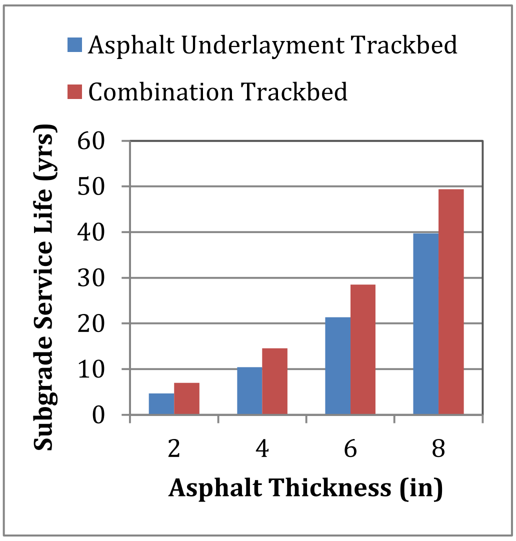
Effect of Asphalt Thickness

The asphalt layer, serving as a waterproofing layer and a stress distributing layer from the top to bottom, exists in both asphalt underlayment trackbeds and combination trackbeds. Research has also demonstrated that asphalt layers used in trackbeds are beneficial for reduction of subgrade moisture contents and fluctuations. The layers also provide a consistently high level of confinement for the ballast which enhances its shear strength (Anderson and Rose, 2008), (Rose and Lees, 2008), (Rose and Bryson, 2009). Designing an asphalt layer with the appropriate thickness is important because a thick asphalt layer can help improve the performance of the trackbed, but increases costs. A balance must be reached between the added cost and the performance enhancements.

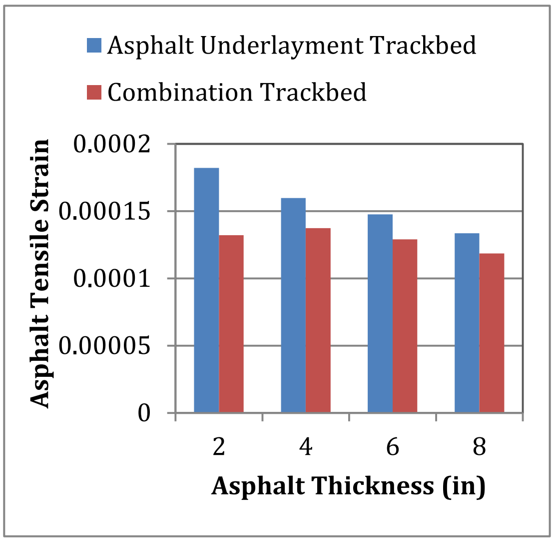
In order to design the asphalt layer that is both economical and efficient, the thicknesses from 2 to 8 inches are varied in 2-inch intervals for the standard design; ballast thickness was maintained at 8 inches. The results by KENTRACK are shown as Figure 5.10. The subgrade compressive stresses and asphalt tensile strains are reduced when the asphalt layer thickness increases. Meanwhile, the associated service lives are increased greatly. The service life of the trackbed is determined by the minimum service life of the subgrade layer and the asphalt layer. Therefore, when the asphalt layer is 2 inches thick in the trackbed, the subgrade service life is much shorter than asphalt service life (5 years versus 12 years). This means the asphalt is not useful for a long enough period of time so the trackbed would be destroyed due to the subgrade damage at the fifth year of the usage. Thus, it is suggested selecting the thickness of asphalt layer, such as 6 to 8 inches, so that the service lives of subgrade and asphalt are similar, so that both subgrade and asphalt can maximize their lifespans.



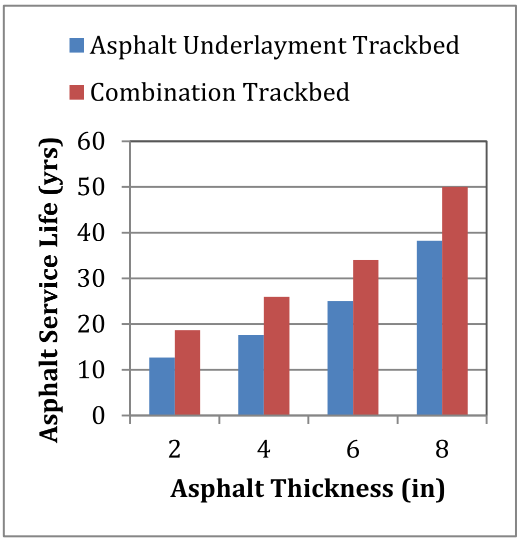
a. Subgrade Compressive Stress



b. Subgrade Service Life



c. Asphalt Tensile Strain



d. Asphalt Service Life

Figure 5.10 Effect of Varying Asphalt Thickness in Trackbeds

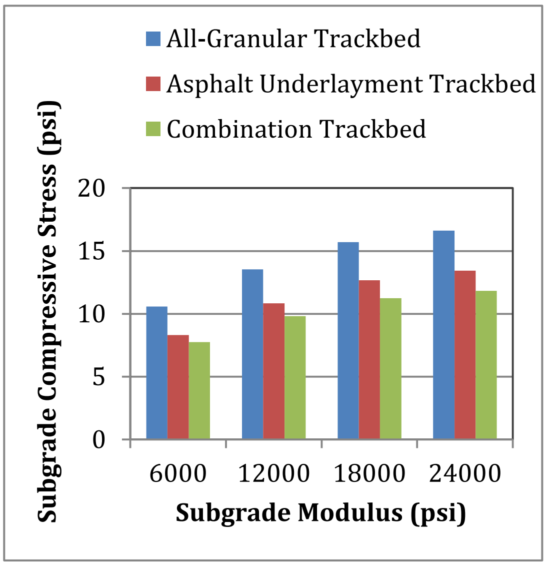
Effect of Subgrade Modulus

Subgrade modulus is a primary input for trackbed design using the KENTRACK program. It estimates the support provided by the layers. Subgrade modulus is a very critical parameter influencing the quality and load carrying capability of the track structure. Subgrade with higher moduli provides a stiffer foundation that has a greater bearing capacity to increase load carrying capability.

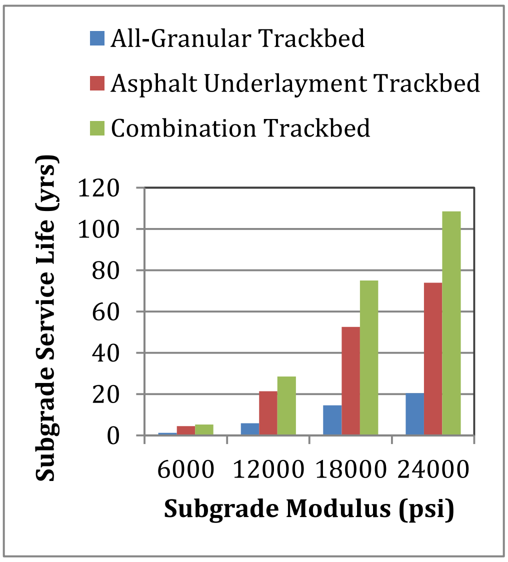
Figure 5.11 shows the effect of subgrade modulus for the different types of trackbeds. An interesting finding is that as the subgrade modulus increases, the subgrade compressive stress also slightly increases, as shown in Figure 5.10(a). However, the subgrade predicted service life is also increased significantly, as shown in Figure 5.11(b). This is because the increase in subgrade modulus leads to a higher bearing capacity. Additionally, the increment of the bearing capacity of the subgrade is always greater than the increment of the subgrade compressive stress. Therefore, even if the pressure on the top of subgrade increases, it should still perform well for an extended period.

Figure 5.11(c) shows the effect of subgrade modulus on the tensile strain at the bottom of the asphalt layer. The tensile strain decreases as the subgrade modulus increases. For the low subgrade moduli, the subgrade cannot adequately support the asphalt layer. In this case, with the same load acting on the asphalt layer, the deformation of the asphalt is increased on the soft subgrade, producing higher tensile strains on the bottom of the asphalt layer due to excessive bending strains.

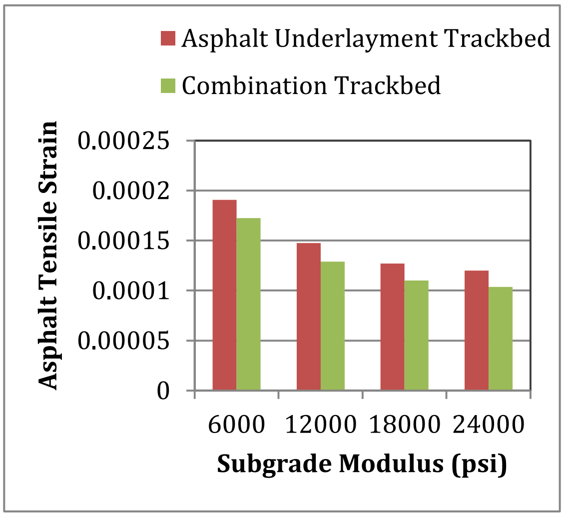
The predicted service lives for asphalt and subgrade for different subgrade moduli are presented in Figure 5.11(b) and Figure 5.11(d). Note that subgrade modulus has a significant effect on the predicted service lives for all the trackbeds. For example, increasing the subgrade modulus from 6,000 psi to 24,000 psi (a 350% increase) under 36-ton axle load, will increase the predicted asphalt service life by a factor of 5 and the subgrade life by a factor of 18. Also, comparing the effects on the asphalt trackbeds to the all-granular trackbed, the subgrade service life for an asphalt trackbed is typically increased by 100% over that of an all-granular trackbed. A combination trackbed has longer service lives of the subgrade and asphalt layers than asphalt underlayment trackbed.



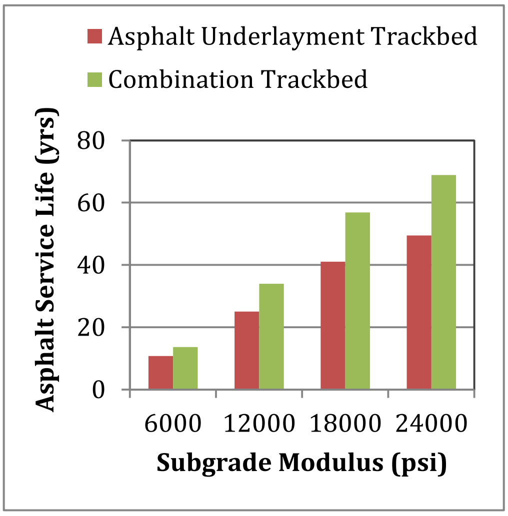
a. Subgrade Compressive Stress



b. Subgrade Service Life



c. Asphalt Tensile Strain



d. Asphalt Service Life

Figure 5.11 Effects of Subgrade Modulus

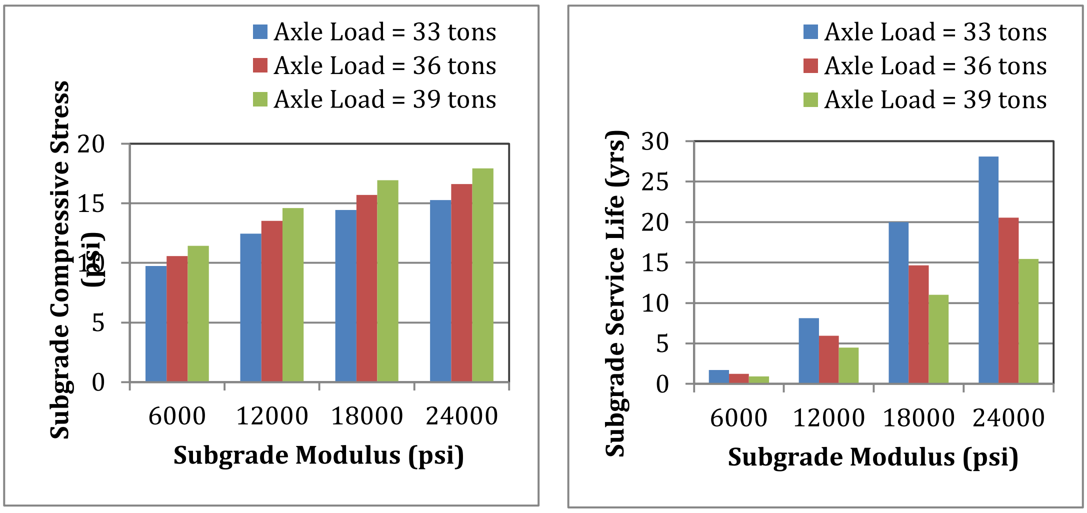
Subgrade serves as the foundation of the railroad trackbed. Loads are ultimately transmitted to the subgrade soil. A subgrade's performance generally depends on two interrelated characteristics.

1. Load bearing capacity; the subgrade must be able to support loads transmitted from upper structure. A subgrade that can support a high amount of loading without excessive deformation is desired.

2. Volume changes; most soils undergo some amount of volume change when exposed to excessive moisture of freezing condition. Some clay soil shrink

and swell depending upon their moisture content, while soils with excessive fines may be susceptible to frost heave in freezing areas. The subgrade modulus also changes when the soil volume changes.

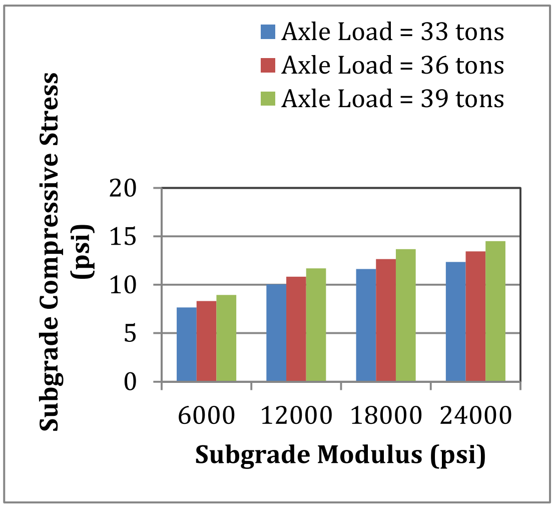
The effect of axle load and subgrade modulus has been discussed in the previous section. In order to understand how the two important interrelated characteristics affect the performance of subgrade, three trackbed models with axle loads varying from 33 tons to 39 tons and subgrade modulus values varying from weak (6,000 psi) to strong (24,000) psi are evaluated. The results are shown in Figures 5.12-5.14.

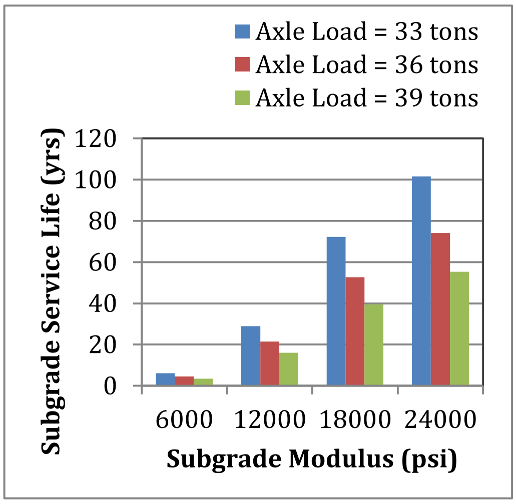


a. Subgrade Compressive Stress **b. Subgrade Service Life**

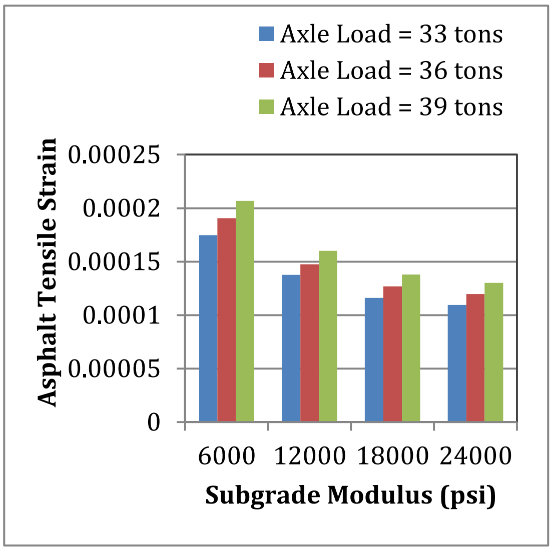
Figure 5.12 Effects of Axle Load and Subgrade Modulus in All-Granular Trackbed

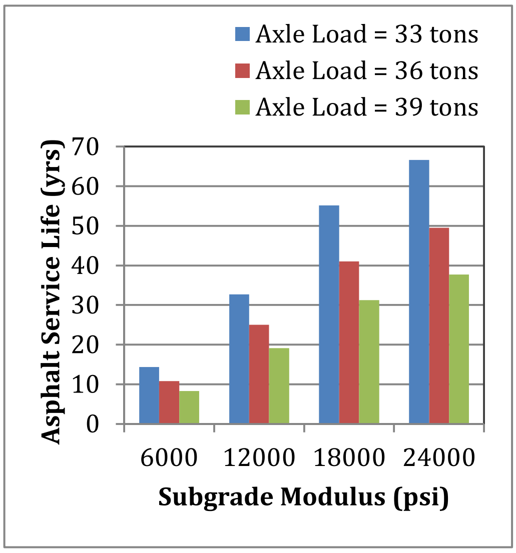
As was demonstrated in the previous section, heavy axle loads can be detrimental to the structural aspects of trackbeds. Consider an asphalt underlayment trackbed. If a heavy axle load (39 tons) is applied on the weak trackbed with a subgrade modulus of 6,000 psi, the subgrade service life is reduced to 3 years, as shown in Figure 5.13(b). Comparing that the strong trackbed with subgrade modulus of 24,000 psi and an applied 39-ton load, the subgrade compressive stress is reduced by a half and the subgrade service life is increased by 15 times. Also, the same trend is apparent relative to asphalt service life. Heavy loads applied on a strong subgrade with high modulus can help reduce asphalt tensile strain and promote longer service life compared to heavy loads applied on a weak subgrade. Therefore, it is implied that if the trackbed is subjected to heavy wheel loads, it is desirable to strengthen the subgrade and maintain a high subgrade modulus.





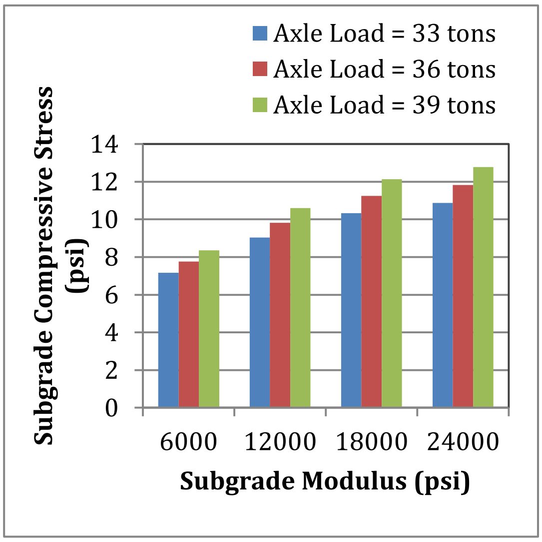
a. Subgrade Compressive Stress **b. Subgrade Service Life**



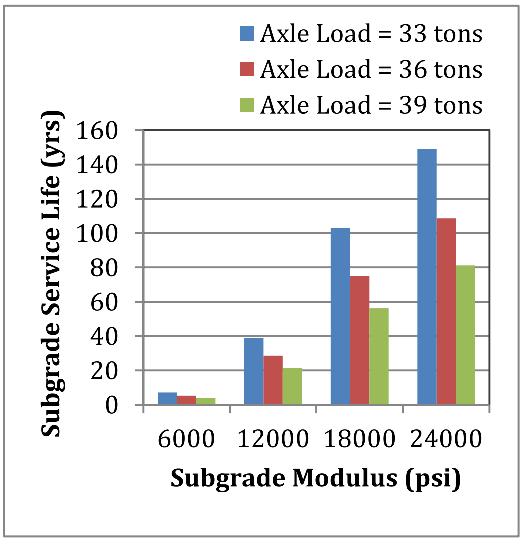


c. Asphalt Tensile Strain **d. Asphalt Service Life**

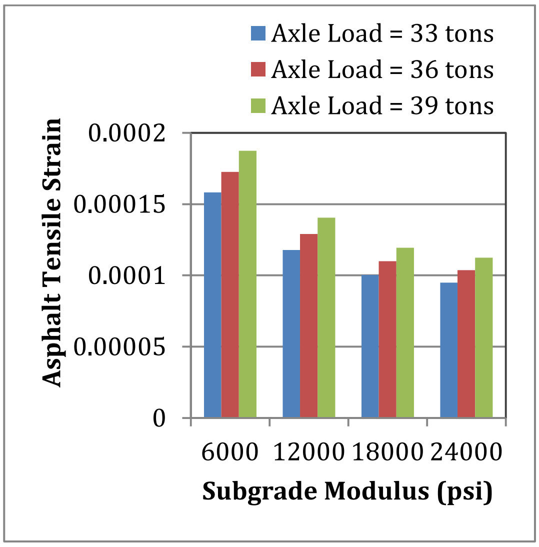
Figure 5.13 Effects of Axle Load and Subgrade Modulus in Asphalt Underlayment Trackbed



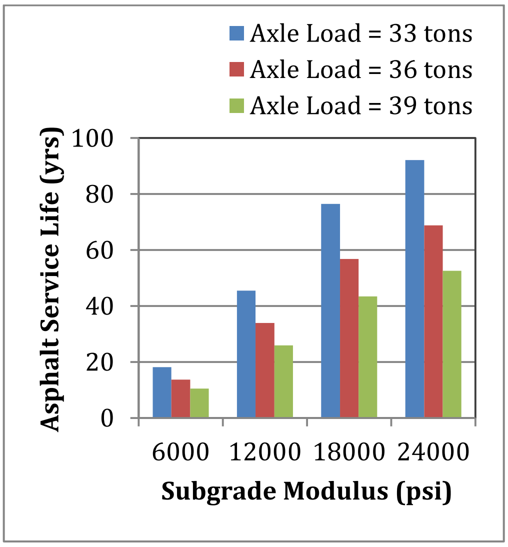
a. Subgrade Compressive Stress



b. Subgrade Service Life



c. Asphalt Tensile Strain



d. Asphalt Service Life

Figure 5.14 Effects of Axle Load and Subgrade Modulus in Combination Trackbed

Effect of Ballast Modulus and Thickness

Ballast distributes the pressures to the subgrade at reduced intensity, provides proper resilience to the rail track structure, assists in draining water from the trackbed, resists the movement of ties, and permits the adjustment of track geometry. However, ballast also has deficiencies. Due to its size composition, it is an open-graded granular material and its original porosity is very high. When loads are applied to it, large deformations can be developed. Herein the effect of two aspects of ballast characteristics, ballast modulus and ballast thickness, are discussed.

Effect of Ballast Modulus

Ballast modulus is a very important factor affecting the resilience of typical all-granular trackbeds. For asphalt underlayment and combination trackbeds, it has an increased capability to resist load induced deformations of the trackbed. However, ballast modulus changes with the density status of the ballast. When the ballast is newly constructed, it is usually very loose with a high porosity, and the modulus is low. Eventually the ballast is well compacted with load repetitions, obtaining a higher modulus. After years, large particles crush into small rounded particles, ballast degrades due to cyclic loading. Therefore, the ballast modulus decreases. Ballast degradation is detrimental to shear strength and rail track structure performance.

The three types of trackbeds were analyzed by KENTRACK to evaluate the effect of varying ballast modulus from 12,000 psi to 30,000 psi. The parameters of models are based on the standard case presented in Chapter 4 except the ballast modulus is varied. Tables 5.4-5.6 show the effect of varying ballast modulus on subgrade and asphalt layers in all-granular, asphalt underlayment, and combination trackbeds.

Generally, the service lives of subgrade and asphalt are increased for a track support consisting of a stronger subgrade. From the tables, it can be noted that ballast modulus has little effect on the pressure distribution on the top of subgrade when the subgrade modulus is low. However, the ballast modulus affects subgrade compressive stress as well as subgrade service life more significantly when the subgrade modulus is higher. In all-granular trackbeds, unbound granular layers (ballast and subballast) are between the ties and subgrade soils. If the ballast modulus is very high, high pressures will develop on the top of subgrade. The asphalt tensile strains at the bottom of asphalt layers increase, especially with a strong subgrade and increasing ballast modulus, which indicates that asphalt layers can withstand the loading and reduce the pressure distributed from the top of subgrade. Another interesting founding is that with the compaction of the ballast, subgrade service lives increase while asphalt service lives decrease. When the ballast is well compacted, the ballast has more contact area to distribute the load, thus subgrade compressive stress is reduced, leading to an increase in subgrade service life. Meanwhile, asphalt layers have to withstand more deformation due to higher moduli of the well compacted ballast, thus, asphalt tensile strains are increased, resulting in decreases in asphalt service lives. However, this does not imply that loose ballast should be preferred in the trackbed. Loose ballast will result in large vertical deformations of the track and cannot provide enough resistance to prevent the movement of the track panels, which is essential since most railroad track in the U.S. has continuously welded rail.

Table 5.4 Effect of Ballast Modulus in the All-Granular Trackbed over Weak and Strong Subgrades

Subgrade Modulus (psi)	Ballast Modulus (psi)	All-Granular Trackbed	
		Subgrade Compressive Stress (psi)	Subgrade Service Life (yrs)
Weak Subgrade Modulus = 6,000	12,000	10.83	1.1
	18,000	10.59	1.2
	24,000	10.36	1.3
	30,000	10.09	1.5
Strong Subgrade Modulus = 24000	12,000	17.59	26.8
	18,000	17.45	27.6
	24,000	17.08	29.9
	30,000	16.73	32.3

Table 5.5 Effect of Ballast Modulus in the Asphalt Underlayment Trackbed over Weak and Strong Subgrades

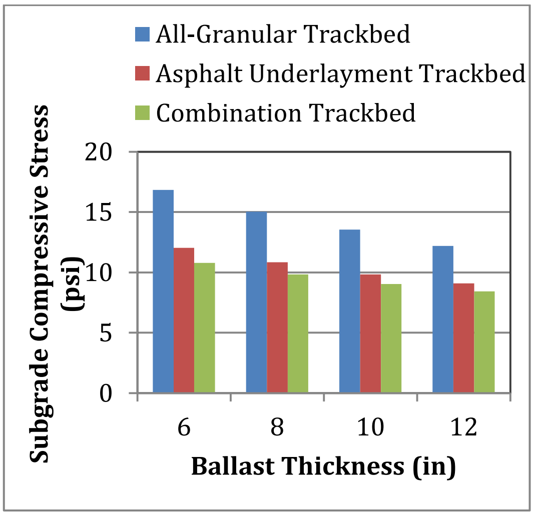
Subgrade Modulus (psi)	Ballast Modulus (psi)	Asphalt Underlayment Trackbed			
		Subgrade Compressive Stress (psi)	Subgrade Service Life (yrs)	Asphalt Tensile Strain	Asphalt Service Life (yrs)
Weak Subgrade Modulus = 6,000	12,000	8.82	3.7	0.000190	10.9
	18,000	8.31	4.5	0.000191	10.8
	24,000	7.93	5.4	0.000191	10.7
	30,000	7.60	6.4	0.000191	10.2
Strong Subgrade Modulus = 24,000	12,000	14.48	95.6	0.000111	65.8
	18,000	14.15	99.4	0.000114	58.3
	24,000	13.82	106.2	0.000116	54.7
	30,000	13.41	115.8	0.000117	53.4

Table 5.6 Effect of Ballast Modulus in the Combination Trackbed over Weak and Strong Subgrades

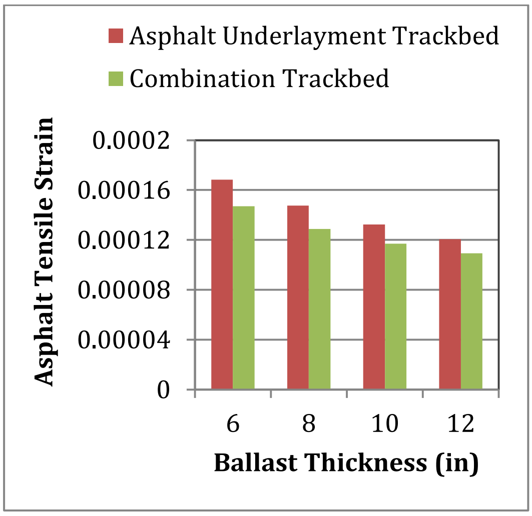
Subgrade Modulus (psi)	Ballast Modulus (psi)	Combination Trackbed			
		Subgrade Compressive Stress (psi)	Subgrade Service Life (yrs)	Asphalt Tensile Strain	Asphalt Service Life (yrs)
Weak Subgrade Modulus = 6,000	12,000	8.21	4.4	0.000167	14.5
	18,000	7.76	5.3	0.000172	13.7
	24,000	7.41	6.4	0.000175	13.5
	30,000	7.13	7.5	0.000177	13.2
Strong Subgrade Modulus = 24,000	12,000	12.56	144.9	0.000093	97.0
	18,000	12.36	149.6	0.000099	81.0
	24,000	12.07	160.3	0.000102	75.8
	30,000	11.78	173.6	0.000104	72.2

Effect of Ballast Thickness

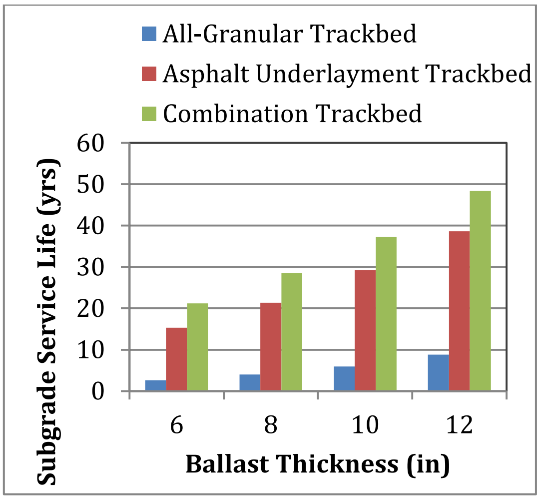
The thickness of ballast affects stress distribution and drainage. For traditional all-granular trackbed, increasing the thickness of ballast will decrease the subgrade compressive stress. However, in asphalt trackbeds, the ballast thickness also affects the deformation and load distribution in the asphalt and subgrade layers. In order to assess the magnitude of this influence, the thickness of ballast was varied from 6 inches to 12 inches in all-granular, asphalt underlayment and combination trackbeds; asphalt thickness was maintained at 6 inches, subballast thickness was maintained at 4 inches. The calculated results are shown in Figure 5.15.



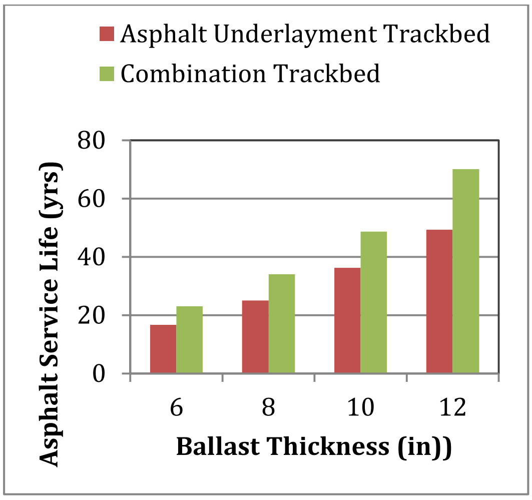
a. Subgrade Compressive Stress



b. Subgrade Service Life



c. Asphalt Tensile Strain



d. Asphalt Service Life

Figure 5.15 Effect of Varying Ballast Thickness in Trackbeds

Figure 5.15(a) shows an increase in the thickness of ballast decreases the subgrade compressive stress, which also leads to an increase in subgrade service life, as shown in Figure 5.15(b). Additionally, asphalt tensile strains are reduced, as shown in Figure 5.15(c), and asphalt service lives, as shown in Figure 5.15(d), are increased with increases in ballast thickness. Loads, stresses and deformations have more space to transmit and distribute with a thicker ballast layer. It also can be noted for a given thickness of ballast, the subgrade compressive stresses in asphalt underlayment and combination trackbeds are much lower than those in all-granular trackbed. Further, the subgrade service lives in asphalt trackbeds are increased in all-granular trackbed for a given ballast thickness. When the ballast thickness increases, the subgrade service lives increase significantly in asphalt trackbeds. Therefore, it is obvious that asphalt layers assist in distributing loads and stresses from the top and reduce subgrade compressive stresses.

Effect of Tie Type

A railroad tie provides the transverse support for the rails in railroad tracks. It is laid perpendicular to the rails. The purpose of installing ties is that they can (1) hold the two rails transversely secure to correct gage, (2) bear and transmit axle loads to ballast with decreased pressure, and (3) anchor the track laterally, longitudinally and vertically. Wood ties and concrete ties are the two most two common types used for railroad tracks.

Wood ties have widespread use in North America due to rich lumber resources. Wood ties are mainly made from hardwoods, which are much stronger compared to soft woods. They are able to withstand the mechanical pressures and forces of the rail/tie plate in the bearing area, and provide resistance to the spikes so that they will stay tight and not loosen. Wood ties usually have creosote treatment to prevent wood ties preservation. The usage life of the wood ties is about 20 years under heavy traffic.

Concrete ties are popular in China, India, and Europe countries. They are cheaper and easier to obtain than wood, better able to carry higher axle-weights and will sustain higher speeds due to higher stiffness. The higher weight ensures improved retention of track geometry especially when installed with continuous-weld rail. Concrete ties have longer service lives and require less maintenance than wood ties, mainly due to their greater weight which helps them remain in the correct position longer.

Table 5.7 contains data for the standard all-granular and asphalt-bounded trackeds with wood and concrete ties. The same design parameters are used to obtain the data of the standard trackbeds with wood ties in the previous chapter.

Table 5.7 Typical Asphalt-Contained and All-Granular with Concrete Ties

Trackbed	Type of Ties	Subgrade Compressive Stress (psi)	Subgrade Service Life (yrs)	Asphalt Tensile Strain	Asphalt Service Life (yrs)
All-Granular Trackbed	Wood	13.52	6.0	n/a	n/a
	Concrete	10.92	13.3	n/a	n/a
Asphalt Underlayment Trackbed	Wood	10.84	21.4	1.48E-04	25.0
	Concrete	9.49	31.6	1.10E-04	61.5
Combination trackbed	Wood	9.82	28.5	1.29E-04	34.0
	Concrete	8.75	38.9	9.40E-05	84.5

Substituting concrete ties for wood ties provides a reduction in subgrade compressive stress and asphalt tensile strain for asphalt trackbeds. The subgrade compressive stress is reduced by as much as about 10 percent and the asphalt strain is reduced by 25 percent. As a result, the service lives of subgrade and asphalt are increased significantly using concrete ties. These benefits occur because the stiffness of concrete ties is much greater than wood ties. The Young's modulus of concrete ties is three times higher than wood ties, thus as loads are applied on rails, concrete ties have higher capability to bear the loads and have less deformation. Therefore, the loads transmitted below the ties become smaller compared to wood ties, leading to decrease in subgrade compressive stresses and asphalt tensile strains.

Comparisons of KENTRACK 3.0 and KENTRACK 4.0 Versions

In order to compare results calculated by Version 4.0 with Version 3.0, a series of calculations using various asphalt binders for the two versions were made. The calculation results are presented in Table 5.8 and Figure 5.16. The calculation results using AC-20 in KENTRACK 4.0 were compared to AC-20 in KENTRACK 3.0. Various asphalt binders, i.e. AC-20, PG 64-22 and PG 76-22 were compared in KENTRACK 4.0. All of the calculations were performed using the standard trackbed designs, i.e. typical values for material parameters and an axle load of 36 ton, as described in Tables 4.1 and 4.2.

The core of the revised program is incorporating the asphalt E* predictive model, however, for the all-granular trackbed, no asphalt layers are included, so all the analyses for all-granular trackbed by KENTRACK 3.0 and 4.0 should be identical, as is indicated in Table 5.8 and Figure 5.16.

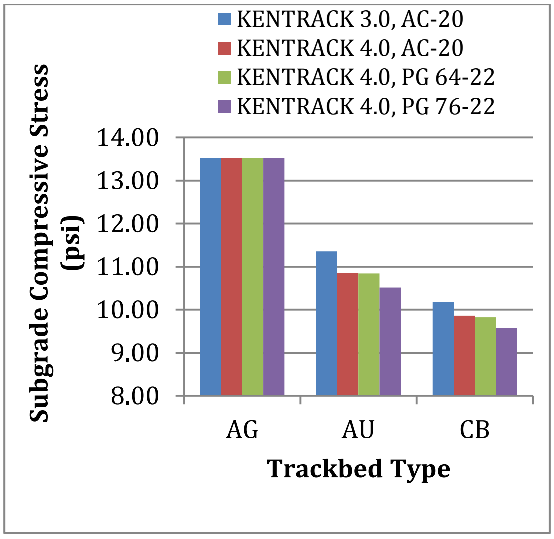
Evaluating the same asphalt binder, AC-20, for the two versions reveals the following. The subgrade compressive stresses differ insignificantly when calculated by KENTRACK 3.0 and KENTRACK 4.0, but the service lives of subgrade layers are increased by 23 percent for asphalt underlayment trackbed and 17 percent for combination trackbed as a result of a stiffer asphalt layer above the subgrade. Meanwhile, the asphalt tensile strains are reduced by 13 percent in the asphalt underlayment trackbed and 10 percent in the combination trackbed. The predictive service lives of asphalt layers are increased by 23 percent for asphalt underlayment and 12 percent for combination trackbed. In KENTRACK 3.0, the dynamic modulus predictive model tends to be conservative due to viscosity determined only at the temperature of 135 °F. By incorporating the Witczak E* predictive model into KENTRACK 4.0, the procedure for predicting asphalt dynamic modulus is more accurate and the moduli using the new model are increased compared to KENTRACK 3.0. Therefore, the asphalt tensile strains decrease and the service lives increase with an increase in asphalt dynamic modulus.

Considering the various asphalt binders in KENTRACK 4.0, it is apparent that the differences of trackbed performance using AC-20 and PG 64-20 are subtle because the properties of AC-20 and PG 64-20 are essentially identical. When the grade of asphalt binder increases, from PG 64-22 to PG 76-22, the subgrade compressive stresses and asphalt tensile strains decrease, and the service lives of asphalt and subgrade increase as expected. The grade of asphalt binder represents the severity of the weather. The larger the number of the upper/lower grade, the more severe weather the asphalt binder can endure, thus the asphalt tensile strains are decreased and the service lives of the asphalt are increased.

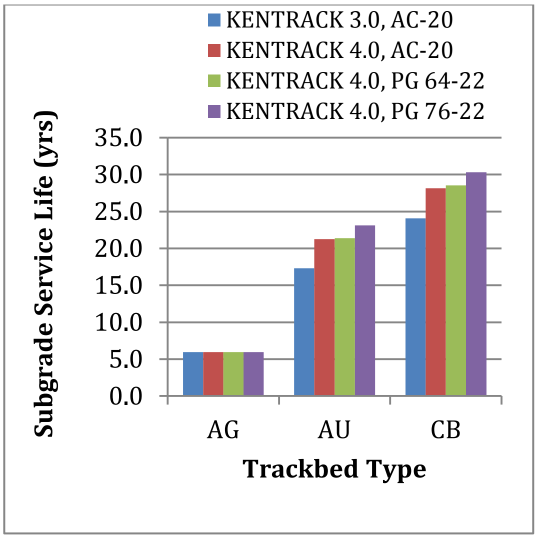
Table 5.8 Comparisons of Calculation Results for Different Asphalt Binder Grades in Different KENTRACK Versions*

Trackbed Type	KENTRACK Version	Asphalt Binder Grade	Subgrade Compressive Stress (psi)	Subgrade Service Life (yrs)	Asphalt Tensile Strain	Asphalt Service Life (yrs)
All-Granular Trackbed	3.0	AC-20	13.52	6.0	n/a	n/a
	4.0	AC-20	13.52	6.0	n/a	n/a
		PG 64-22	13.52	6.0	n/a	n/a
		PG 76-22	13.52	6.0	n/a	n/a
Asphalt Underlayment Trackbed	3.0	AC-20	11.36	17.3	1.73E-04	20.4
	4.0	AC-20	10.86	21.3	1.50E-04	24.9
		PG 64-22	10.84	21.4	1.48E-04	25.0
		PG 76-22	10.52	23.1	1.34E-04	26.5
Combination Trackbed	3.0	AC-20	10.18	24.1	1.47E-04	30.1
	4.0	AC-20	9.86	28.2	1.33E-04	33.6
		PG 64-22	9.82	28.5	1.29E-04	34.0
		PG 76-22	9.58	30.3	1.18E-04	35.6

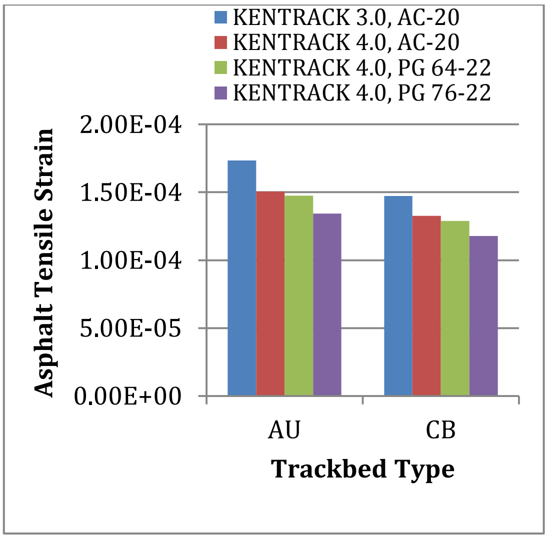
** Subgrade modulus = 12,000 psi*



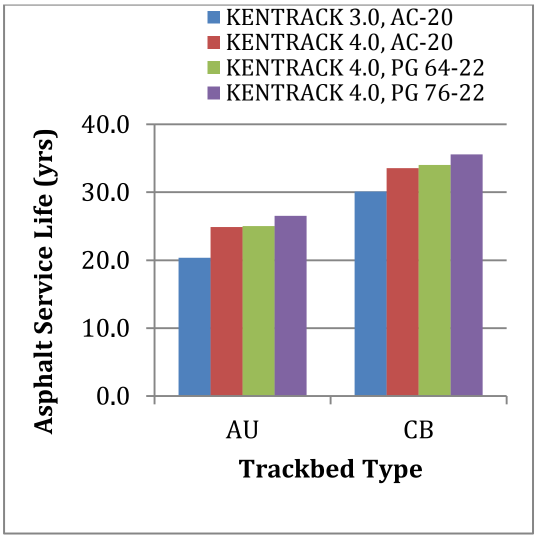
a. Subgrade Compressive Stress



b. Subgrade Service Life



c. Asphalt Tensile Strain



d. Asphalt Service Life

Note:

AG: All-Granular Trackbed; AU: Asphalt Underlayment Trackbed; CB: Combination Trackbed

Figure 5.16 Comparisons of Calculation Results for Different Asphalt Binder Grades in Different KENTRACK Versions

CHAPTER 6. OTHER UPDATES ON KENTRACK 4.0 VERSION

KENTRACK 4.0 version addressed a myriad of "bug" problems. Most notably, in the previous versions, after each calculation, an "Information" window popped up to ask whether the program was to be restarted or closed. If users wanted to vary some parameters to evaluate the effect on the trackbeds, they had to restart the program and reset all the parameters. This does happen in KENTRACK 4.0. Users may now compute results without restarting the program. After each run, users are able to go back to previous steps to vary the value they would like to change, thereby retaining parameter values from previous runs. After values are changed, the program recalculates the results according to the newly defined parameters.

Furthermore, user could tweak parameters and reprocess the data. Analysis of all-granular trackbed keeps the same structure as in the previous versions. For asphalt-contained trackbeds, in Pre-KENTRACK 4.0, the properties of asphalt cement AC 10 or AC 20 was used as a typical grade of asphalt cement. However, After Superpave, asphalt grading system and their properties have changed significantly. For the past several years trackbed construction has used the Superpave PG system based asphalt binders. Therefore, the program maintains the previous asphalt grading system and incorporates information of the new asphalt grading system for comparison purposes.

Figure 6.1 shows the input interface of asphalt binder selection. The most common asphalt cements/binders are incorporated in the program. Once the grade is selected, the program will automatically calculate the value for viscosity using A-VTS relationship, as Eq. 3.11. The viscosity is a variable for asphalt dynamic modulus prediction. The "other" option provides users the ability to define the viscosity in four seasons.

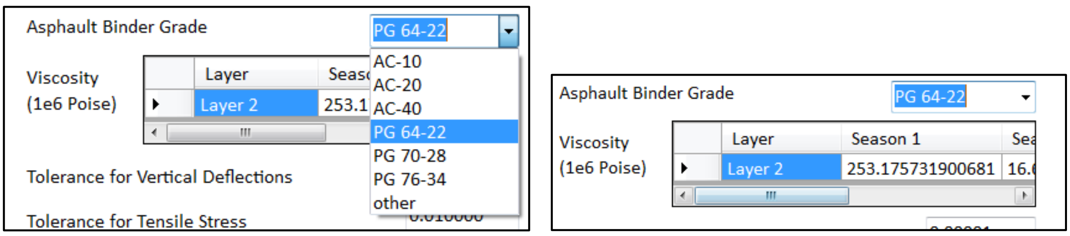


Figure 6.1 Input Interface of Asphalt Binders

Moreover, users can define a path to save calculated results. A big shortcoming in the previous versions is that the program was designed on the basis of Window XP and pre-Windows XP operating systems. In any post-Windows XP system, i.e. Vista, Windows 7 and Windows 8, an error window “unhandled exception has occurred in your application”, popped up in the last step, leading to the input parameters and calculated results not being saved. In KENTRACK 4.0, the program addresses the problem by prompting users for a path to save results in a text format. Saved files include user input data for future reference, as shown in Figure 6.2.

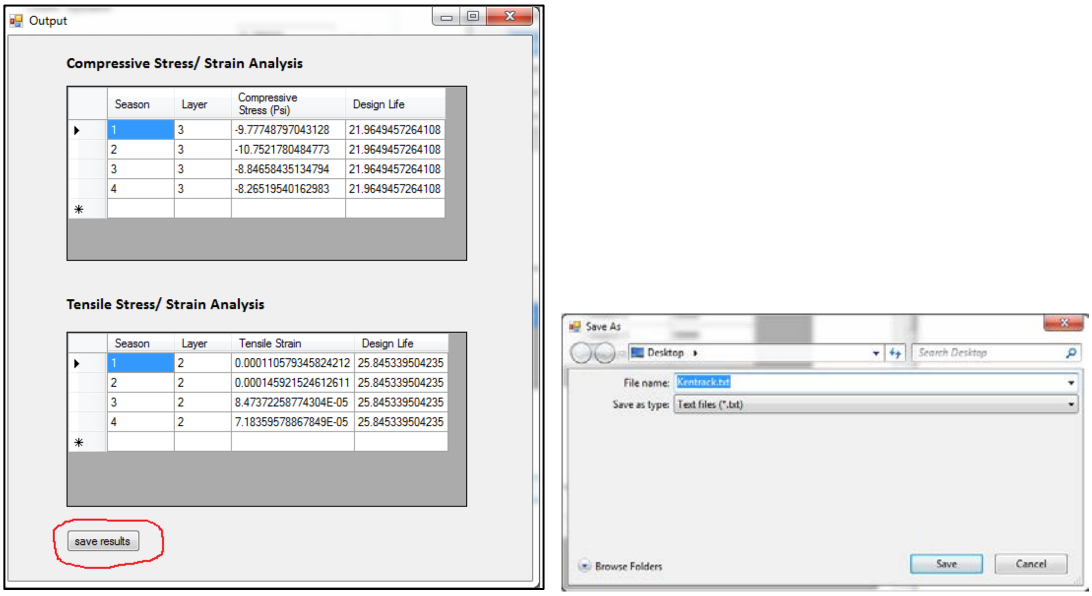


Figure 6.2 Output Interface

Lastly, in addition to English unit system, this update includes the international unit system as an alternative to input parameters. All defaults as well as associated unit labels are adjusted accordingly. Calculated outputs provide results based on the unit system selected at configuration time. Table 6.1 shows the comparison of critical results based on two unit systems. All the parameters are for the standard case, as shown in Table 4.1 and Table 4.2. The values indicate the results based on the two unit systems are identical to each other, which validates the accuracy of the transformation.

Table 6.1 Comparisons of Results of KENTRACK 3.0 VS. KENTRACK 4.0 Versions

Type of Trackbed	Unit System	Subgrade Compressive Stress	Subgrade Service Life (yrs)	Asphalt Tensile Strain	Asphalt Service Life (yrs)
All-Granular Trackbed	English Unit	13.52 psi	6.0	n/a	n/a
	SI Unit	0.093 MPa	6.0	n/a	n/a
Asphalt Underlayment Trackbed	English Unit	10.84 psi	21.4	0.000148	25.0
	SI Unit	0.075 MPa	21.4	0.000148	25.0
Combination Trackbed	English Unit	9.82 psi	28.5	0.000129	34.0
	SI Unit	0.068 MPa	28.5	0.000129	34.0

Note: 10 psi =0.069 MPa

CHAPTER 7. SUMMARY AND DISCUSSION

The 4.0 version of the KENTRACK computer program, utilizing finite element method and multi-layered theory, has been described and utilized throughout this thesis. The Witczak E* predictive model for asphalt was specifically incorporated in the updated version. Asphalt binders classified in either AC system or PG system can be used in KENTRACK 4.0. The program was expanded to include the SI international unit system. The recently incorporated Witczak E* predictive model has been specifically evaluated.

The effects of a series of variables on trackbed design are calculated and analyzed by the program. The variables that have the most significant effect on the predicted railroad trackbed service life is subgrade modulus and axle load. The importance of initially designing for and maintaining high subgrade moduli within the track structure cannot be over emphasized. It is critical to control the magnitude of axle loads. Heavy load is detrimental to trackbed structural adequacy and shortens service lives of the subgrade layer and the asphalt layer, if included.

Overall, the combination trackbed, utilizing both asphalt and granular subballast layers, has the longest service lives for subgrade and asphalt. The all-granular trackbed has the shortest service lives for the same loading conditions. The waterproofing asphalt layer, not only protects subgrade from the damaging effects of water assuring high subgrade modulus, but it also can strengthen subgrade support by reducing subgrade stress.

All damage analyses for the subgrade and asphalt within the track structure are based on damage equations developed for highway pavements. The critical outputs are vertical compressive stress on the subgrade and horizontal tensile strain on the bottom of the asphalt layer. However, the actual service environment for the asphalt and subgrade in railroad trackbed environments are likely to be less severe than highway pavements due to lower magnitudes of subgrade loading, less sunlight exposure, and minimal temperature fluctuations in the insulated trackbed environment. The predicted lives for railroad applications would thus be conservative.

The findings and conclusions emanated from the computer calculated stresses, strains and the associated service lives in the track structure calculated by the KENTRACK 4.0 program are covered in the following discussions.

Varying Rail Size

As expected, increasing rail size can reduce subgrade compressive stresses in all trackbeds and asphalt tensile strains in asphalt trackbed because rails with larger size are stiffer, producing less rail bending stresses when loads are applied.

Furthermore, the effect of heavy axle loads applied on the larger size rails is equivalent to the effect of light axle loads applied on the smaller size rails. A heavy axle load has a detrimental effect on the performance of subgrade and asphalt, but using large size rail may help alleviate this effect. Therefore, for a main line having heavy traffic, using large size rail is desirable. Small size rail is appropriate for the branch lines with light traffic.

Varying Axle Load

Increasing axle load will increase subgrade compressive stresses and asphalt tensile strains if an asphalt layer is included. As expected, service lives of subgrade and asphalt layer decrease. Overall, the subgrade compressive stresses and asphalt tensile strains are increased by 20 percent in asphalt underlayment trackbed but the service lives of the subgrade and asphalt are reduced greatly, about 50 percent, when axle loads increase from 33 tons to 39 tons..

Axle loads have significant effect on the service lives of trackbeds. Heavy haul lines require a strong trackbed foundation support due to increases in subgrade compressive stress and asphalt tensile strain in asphalt trackbeds. Therefore, controlling the magnitude of axle load is beneficial for trackbed service life.

Varying Asphalt Properties

The effect of varying asphalt properties depends on the way how the asphalt properties affect the Witczak E* predictive model. Higher value for E* will produce less asphalt tensile strain due to higher stiffness and increase asphalt service life as expected.

The sensitivity analysis on Witczak E* predictive model regarding asphalt properties is shown in Figure 5.8. The effect of increasing the percentage of aggregate passing #200 Sieve on asphalt dynamic modulus simulates a parabola and the peak value is around 8 percent. Also, the effect of percentage of air voids has a peak value of 2 percent air. The conclusion is more obvious when the temperature is low. At the high temperature, the dynamic moduli is stable as aggregate passing #200 Sieve increases. Moreover, the dynamic modulus increases with decreases in the percentage of aggregate retained on the No.4 and 3/4-inch Sieves and an increase in the percentage of aggregate retained on the 3/8-inch Sieve.

When the percentage of bitumen increases, the dynamic modulus decreases due to the viscoelasticity of the bitumen. If the percentage of the bitumen increases, the asphalt mixture will have less stiffness, leading to reduction in asphalt dynamic modulus.

Viscosity has a direct effect on asphalt dynamic modulus. Increasing viscosity will increase the asphalt dynamic modulus. Dynamic modulus is the highest in Winter and the lowest in Summer, because viscosity is extremely temperature sensitive.

When the asphalt upper binder grade increases, the asphalt dynamic modulus is predicted to be greater, especially under the higher temperature as a result of upper binder grade controlling the maximum temperature that asphalt works. Adversely, the asphalt dynamic modulus increases more greatly under lower temperature than higher temperature when the asphalt lower binder grade increases because the lower binder grade takes care of the minimum temperature that asphalt layer undergoes. Therefore, for those weather severe area, increasing the asphalt binder grade either upper or lower as needed can improve the performance of asphalt contained trackbeds because of the increase in asphalt dynamic modulus.

Varying Asphalt Layer Thickness

Increasing asphalt layer thickness from 2 inches to 8 inches in asphalt underlayment trackbed and combination trackbed, while maintaining constant ballast thickness, reduces subgrade compressive stress by 38 percent in asphalt underlayment trackbed and 35 percent for combination trackbed. Asphalt tensile strains are reduced by 26 percent in asphalt underlayment trackbed and 10 percent in combination trackbed. The service lives of asphalt and subgrade are both increased by increasing asphalt layer thickness.

Varying Subgrade Modulus

Increasing the subgrade modulus leads to incremental increases in subgrade vertical compressive stress. The range is from 10.5 psi to 16.5 psi when subgrade modulus increases from very weak 6,000 psi to very stiff 24,000 psi. Stresses are typically 15% higher in the all-granular trackbed for a given subgrade modulus as compared to asphalt trackbed.

However, the increases in stress levels with increasing subgrade modulus are insignificant. In fact, the stronger subgrade with higher modulus has large capability to bearing higher stresses. Therefore, although the subgrade compressive stress level increases, but the subgrade bearing capability increases more greatly than the stress level. Thus, the service life of subgrade is still increased with the increases in subgrade modulus.

Asphalt tensile strains are reduced as subgrade modulus increases. Once the subgrade become stiffer, the asphalt layer deflects less, thus the asphalt tensile strains decrease, extending the fatigue life of the asphalt layer.

It is crucial to have a stiffer trackbed foundation with high moduli. A soft subgrade may have issues on track geometry maintenance such as excessive settlement, deflection, component wear, etc.

Varying Ballast Modulus

Increasing ballast modulus has a minimal effect on the subgrade compressive stresses and asphalt tensile strains, especially with weak subgrade. If the subgrade is strong, increasing ballast modulus can slight decrease subgrade compressive stresses as well as asphalt tensile strains resulting in a slight increase the associated service lives.

Varying Ballast Thickness

Increasing ballast thickness reduces subgrade compressive stresses for all these types of trackbeds. The predicted service life of the subgrade increases as the ballast thickness increases. The asphalt tensile strains in asphalt underlayment trackbed and combination trackbed are also decreased. The increase in asphalt service live is very pronounced and is typically three times longer for the thicker ballast sections.

CHAPTER 8. FUTURE RESEARCH RECOMENDATIONS

The contents of this thesis by no means represent the conclusion of the research topic. Recommendations for the further research follow:

1. The temperature of each season used for predicted asphalt dynamic modulus is assumed as the same as the ground temperature. The soil temperature at 12-inch depth around the world in January, April, July and October is used to represent the average temperature at the bottom of the asphalt layer each season. Tests to determine the actual temperature in the asphalt layer should be conducted.

2. As emphasized in the study, the failure criteria based on highway experience appear to be conservative. It is likely that the asphalt has a much longer predicted service life when it is used in the insulated environment of a railroad trackbed. A particular study should be made based on previous constructed asphalt sections to validate these criteria. The tentative design scheme can be used to estimate the allowable number of repetitions for each site based on the thickness of each layer and its materials properties at the site. A subsequent comparison between these estimated repetitions and actual numbers of repetition obtained from railroad records should be able to verify the validity of the tentative criteria. To establish the true criteria, documented information on more asphalt test sections with different thickness designs should be collected.

3. The long-term aging behavior of asphalt binder should be considered. The asphalt layer in trackbeds is buried under ballast and not exposed in the sunlight, the temperature in trackbeds is not as high or as low as in highway pavements and oxidative rate is slower in trackbeds. The environment in asphalt trackbed is less severe than in asphalt highway pavement, the asphalt aging behavior in trackbeds is different from highway pavement and the effect needs to be investigated. This involves drilling asphalt cores from the trackbed and analyzing the aging of the asphalt based on the laboratory test. These results can be compared to aging test results for highway materials.

4. Ballast materials forming part of railway structures are subjected to severe cyclic loading environment. As a result of these loads, ballast densification, aggregate degradation, and lateral spread of the ballast material underneath the ties takes place,

inducing permanent deformations on the railroads. Maintenance and rehabilitation costs of tracks due to problems related with ballast performance are substantial. Understanding the crushable behavior of track ballast could lead to the design of better railroads. It is necessary to conduct ballast degradation research, attempting to obtain contact forces between ballast and tie by the discrete element method. Sensors can be placed between ballast and ties in the field, thus the actual contact forces can be investigated to verify the results of numerical simulation. Ballast failure criteria should be established to predict the service life of ballast before degradation.

5. The economic feasibility is an important aspect that is directly linked to the popular applications of asphalt trackbeds but has not been studied in detail in this or other research. There are many factors that affect the economic feasibility, such as geographical and climatic situations, prices for material and labor, and subgrade conditions. The construction of asphalt trackbed may cost more as compared to the all-granular trackbed. However, the difference varies locally and the overall costs considering maintenance over the life of the track may be substantially less. For those areas where good quality ballast is rare and expensive, the difference in cost will be small because lower quality aggregates can be used in asphalt mix. The reduction of maintenance is necessary for verifying this benefit. Proving the effectiveness of asphalt trackbeds depends not only on the structural advantage but also on the economic feasibility. A detailed study of this aspect is definitely necessary.

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Abstract accepted

2. Rose, J.G., Liu, S.S., Souleyrette, R.R. (2013) KENTRACK 4.0: A Railway Trackbed Structural Design Program. The Joint Rail Conference 2014, Colorado Springs, CO.