Benefit of Lime Treatment for Controlling Longitudinal Pavement Cracking Due to Expansive Subgrade

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ABSTRACT

Dry-land longitudinal cracking is one of the most prevalent pavement distresses caused by expansive clay in pavement subgrade. In the State of Texas, using lime to stabilize expansive subgrade soil is one of the most effective methods to control pavement longitudinal cracking due to the shrinkage of expansive subgrade. However, the use of lime treatment is mostly based on empirical engineering experience. The mechanism of lime controlling dry-land cracks has not been addressed in depth. The benefit of lime treatment needs to be quantified.

This paper develops finite element models to investigate the mechanism of lime-treated layer preventing shrinkage cracks from propagating to pavement surface. The lime treatment can significantly change the properties of the natural expansive soils through the processes of modification and stabilization. Finite element modeling results show that the tensile stress developing in the lime-treated layer is less than the tensile strength of the lime-stabilized soil. Therefore, shrinkage cracks are less likely to initiate in the lime-stabilized layer. The possible location of the shrinkage crack initiation is in the untreated soil close to the bottom of the lime-treated layer. The theoretical calculation shows that, when the initial crack propagates into the lime-treated soil, the magnitude of the stress intensity factor at the upper crack tip is in the same order of the estimated fracture toughness of the lime-treated soil. The shrinkage crack is not likely to develop through the lime-treated soil with increased fracture toughness. The occurrence of multiple shrinkage cracks reduces the stress concentration at each crack tip, which further reduces the probability of shrinkage crack propagating toward the pavement surface.

I. INTRODUCTION

Expansive soils, widespread in 20 percent of the United States, experience significant volumetric changes when subjected to moisture variation (*1*). Expansive soils usually have a high plasticity index (PI). If expansive soils are present in pavement subgrade, their volumetric changes can produce serious damage to highways and streets. Longitudinal cracking is one of the most prevalent pavement distresses caused by the volumetric change of the expansive subgrade. As the weather becomes dryer, the expansive subgrade loses moisture. When the water content of the subgrade soil decreases to a certain level, cracks initiate in the drying subgrade, reflect from the highly plastic subgrade through the pavement structure, and then generate longitudinal cracks at the pavement surface (*2, 3*). This type of longitudinal crack is termed "dry-land crack", and most of these cracks are close to the pavement shoulder.

The State of Texas has the most severe degree of expansive soil occurrences. The extensive network of surface-treated pavements in Texas has suffered from the detrimental effects of expansive soils in the subgrade for decades. Pavement and geotechnical engineers have attempted a number of methods to eliminate the dry-land cracking. Lime treatment is the most extensively used chemical alteration method to modify the expansive properties of subgrade soils. Using lime to stabilize expansive subgrade soils is one of the most effective methods to reduce longitudinal cracking of pavements due to subgrade shrinkage. Engineering practice has shown that there is little development of shrinkage cracks in well designed lime-treated expansive clays. For example, Figure 1 shows the cross section of an in-service pavement with a lime-stabilized layer. The material comprising the bottom layer of this section is high PI clay; the middle section is a lime-treated layer; and the top layer is a hot mix asphalt (HMA) layer. This picture clearly indicates that the shrinkage cracks developed in the shrinking soil while the lime-treated layer still remained intact. In this example, the lime-stabilized layer showed tensile strength high enough to resist shrinking stress produced by suction change.

However, the use of lime treatment is mostly based on empirical engineering experience. The mechanism of lime controlling dry-land cracks has not been addressed in depth. The benefit of lime treatment needs to be quantified. The investigation of mechanism and benefit of lime treatment will provide theoretical support to the mechanistic design of pavements over expansive clays.

This paper develops finite element models to investigate why and how lime-treated layer prevents shrinkage cracks from propagating to pavement surface. The next section reviews the properties of the lime-treated soil, which is followed by finite element modeling of pavement with a lime-treated layer. The subsequent section studies the development of shrinkage cracks. The final section summarizes the main findings in this research.

FIGURE 1 Shrinkage Cracks in High PI Clay Covered by Lime-Treated Layer (Courtesy of Lytton and Freeman at Texas Transportation Institute).

II. BACKGROUND OF LIME TREATMENT

Lime treatment of expansive soils can essentially provide two methods of improving soil engineering properties: modification and stabilization.

A. Lime Modification

Modification occurs because of two chemical processes: i) exchange of calcium cations supplied by the lime ($Ca(OH)_2$ or hydrated lime) for the normally present cation adsorbed on the surface of the clay mineral, and ii) reaction of the hydrated lime with the clay mineral surface in the high pH environment promoted by the lime-water system (*4*). Modification can successfully provide substantial reduction in expansive soil's plasticity, moisture holding capacity, and swelling property.

The plasticity of expansive soils is reduced most effectively by the first increments of added lime. The soil often becomes non-plastic with lime treatment. Different percentages of lime are required to reduce plasticity to desired levels for different soils. Generally, a larger percentage of lime added to the soil provides additional reduction of the plastic index. This fact indicates that a larger percentage of lime is needed when treating soils with higher PI and higher clay content in order to achieve the non-plastic condition, if it can be achieved. Figure 2 shows the reduction in plastic indices of four soil samples: a Texas soil, a California soil, and two South

Dakota soils (*5*). This figure reflects the immediate effect of lime treatment without long curing time.

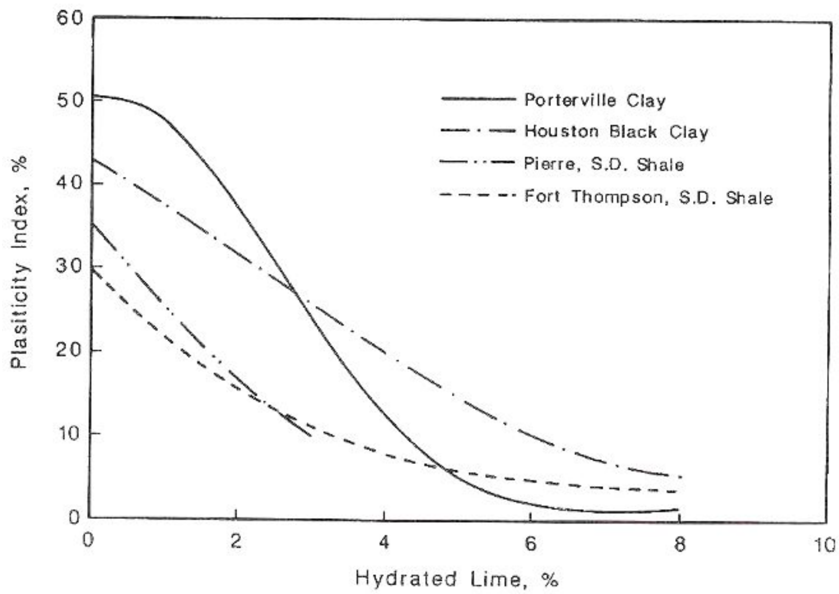


FIGURE 2 Reduction in Plasticity Index by Lime Treatment (*5*).

The reduction of plastic index by lime treatment is a significant indication of the reduction of soil swell potential and swelling pressure. Seed et al. (*6*) developed a relationship between PI and swell potential as follows:

$$\text{Percent Swell} = 0.00216 \times PI^{2.44} \tag{1}$$

This relationship is graphically illustrated in Figure 3, in which the percent swell was defined as volume change of the soil when the moisture content increased from the optimum moisture content to the saturation moisture content. With the reduction in percent swell or volume change, the swell pressure was considerably decreased by adding lime to the soil. Basma and Tuncer (*7*) found that lime percentage and curing time profoundly reduce the swell potential and swell pressure of expansive soils. Figure 4 demonstrates the influence of the amount of lime and the influence of curing time on the reduction of swell pressure of lime-treated clay. In their one-dimensional swell tests, the swell potential of a high PI clay with a swell pressure of 2,600 kPa was reduced to 1,700 kPa with 10 percent hydrated lime (immediately) and was further reduced to 0 kPa with 28 days of cure at only 4 percent hydrated lime.

Associated with a reduction in the swell pressure and swell potential is the decrease in matric suction compression index of the soil. Matric suction, one of the two components of soil suction which is a measure of the affinity of the soil for water, is derived from the negative water pressure according to the capillary phenomenon. Another component of soil suction is osmotic suction, which arises from the soluble salts in the soil water that produced the osmotic repulsion forces. Matric suction compression index was introduced by Lytton et al. (*8, 9, 10*) in their empirical model (Equation 2) as a parameter which relates the change of matric suction and the volumetric change of expansive soil. This empirical model was developed to estimate the volumetric strain of expansive soil and will be used in the following sections for the finite element modeling purpose. If other parameters in Equation 2 remain unchanged, a decrease of the matric suction compression index will result in the decrease of volumetric strain. Lime treatment can significantly reduce the matric suction compression index. For example, if a subgrade soil with an original matric suction compression index (γ_h) of 0.0313 is treated by 8 percent of lime, γ_h may decreases to 0.0156 (*11*). When subjected to the same matric suction change and model constraints, the soil with smaller γ_h will have smaller maximum tensile stress developing in the soil body than the soil with higher value of γ_h.

$$\frac{\Delta V}{V} = -\gamma_h \log_{10}\left(\frac{h_f}{h_i}\right) - \gamma_\sigma \log_{10}\left(\frac{\sigma_f}{\sigma_i}\right) - \gamma_\pi \log_{10}\left(\frac{\pi_f}{\pi_i}\right) \qquad (2)$$

in which:

$\frac{\Delta V}{V}$ = volumetric strain;

h_i = initial value of matric suction;

h_f = final values of matric suction;

σ_i = initial value of mean principle stress;

σ_f = finial value of mean principle stress;

π_i = initial value of osmotic suction;

π_f = finial value of osmotic suction;

γ_h = matric suction compression index;

γ_σ = mean principal stress compression index; and

γ_π = osmotic suction compression index.

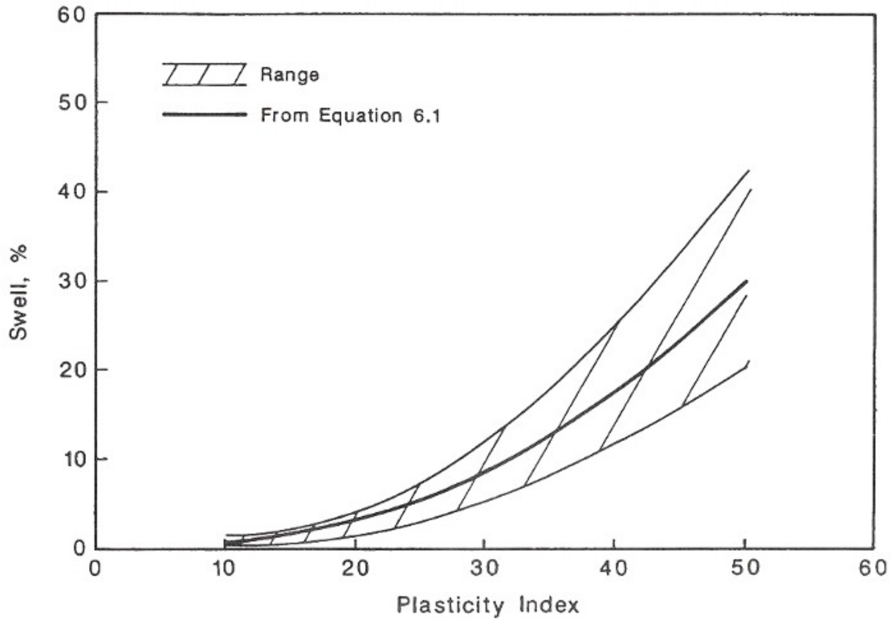


FIGURE 3 Relationship between Plastic Index and Swelling (*6*).

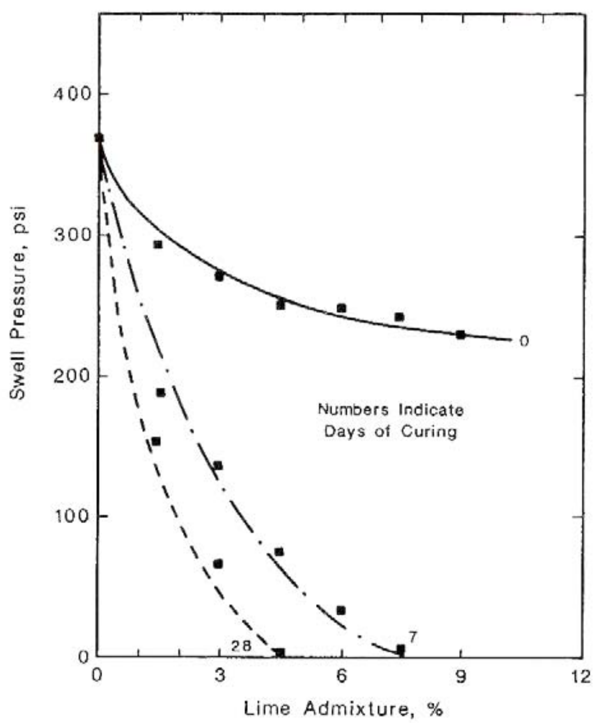


FIGURE 4 Swell Pressure as a Function of Lime Content and Period of Curing for Irbid, Jordan, Clay (*7*).

B. Lime Stabilization

Besides modification, another improvement provided by lime treatment is stabilization. Stabilization offers the soil long-term strength through a long-term pozzolanic reaction. In the pozzolanic reaction process, the calcium from the lime reacts with the aluminates and silicates solubilized from the clay mineral surface to produce the formation of calcium silicate hydrates and calcium aluminate hydrates. The pozzolanic reaction may take many years. The long-term pozzolanic reaction provides substantial increases in resilient modulus and tensile strength. The typical modulus of lime treated soil falls in a range between 210 MPa and 3,500 MPa (*12*). The increase of soil stiffness is not beneficial to the shrinkage problem in this study because the increase of the elastic modulus results in the increase of the tensile stresses when the model is subjected to the same suction change and constraints.

However, lime stabilization also increases the unconfined compressive strength and tensile strength of the soil in addition to the stiffness. Evans (*13*) used 8 percent lime to treat highly plastic Queensland black clays with original PI's of near 40. The lime treatment reduced the PI to below 8 while increasing the 26-week unconfined compressive strength to over 4.5 MPa. Little (*12*) stated that the ultimate unconfined compressive strength of lime-soil mixtures can be as high as 7 to 10 MPa or higher. The stabilization may result in a 400 to 1,500 percent tensile strength increase. Considerable research has been conducted to estimate the correlation between the unconfined compressive strength and tensile strength of lime-stabilized soil. Metcalf et al. (*14*) found that the tensile strength was between one-twelfth and one-tenth of the unconfined compressive strength for the lime-treated soils. Thompson (*15, 16*) reported an approximate 0.13 ratio of indirect tensile strength to the unconfined compressive strength of the lime-stabilized soils, and this ratio exhibited little variation between test samples. Tulloch et al. (*17*) developed an equation based on experimental results and regression analysis to predict the indirect tensile strength of the lime-treated material if the unconfined compressive strength was known. Therefore, if a lime-soil mixture has an unconfined compressive strength of 10 MPa, its tensile strength may be higher than 1 MPa.

C. Summary of Lime Treatment

In summary, both modification and stabilization of the lime treatment can significantly improve the engineering properties of expansive soils. First, the modification process decreases the potential expansiveness of the soils from very high to low. The addition of lime decreases the liquid limit and plasticity index of the expansive soils while increasing their shrinkage limit. When subjected to the same suction change and constraints, the lime-treated soil experiences lower tensile stress development in the soil body than the untreated expansive soils. In addition, lime stabilization considerably increases the tensile strength of the expansive soils. The stabilized soil with higher tensile strength is able to resist larger tensile stress developing in the soil. If the tensile stress induced within the stabilized layer does not exceed the tensile strength, the shrinkage crack will not initiate in the soil. Therefore, lime treatment provides the most desirable benefit for the expansive subgrade: decreasing the tensile stress induced by suction change as well as increasing the soil's tensile strength at the same time.

Poisson's ratio of lime-treated soil is a stress dependent property. Poisson's ratio falls in the range of 0.1 to 0.2 when the stress level is less than 50 percent of the ultimate compressive strength. At higher stress levels, Poisson's ratio is within the range of 0.2 to 0.3. A typical value of between 0.15 and 0.2 is usually used for Poisson's ratio of lime-stabilized soil (*18*).

III. FINITE ELEMENT MODELING OF PAVEMNET

A two-dimensional plane strain pavement model is constructed in a computer program, ABAQUS. The proposed pavement model has an asphalt layer, a granular base, a lime-treated layer, and a multi-layer subgrade. Each pavement layer is assumed to be homogenous, isotropic, linearly elastic, weightless, and bonded to the underlying layer. Because of symmetry, a half-wide (4 m) pavement is studied to reduce computation effort. Figure 5 illustrates the details of this model. This model is similar to the model recently constructed during research that investigated the mechanism of dry-land longitudinal crack development (*19*). The only difference between the two models is that a lime-treated layer is included in the model proposed in this paper. In other words, a part of the subgrade soil and field soil in the previous model is treated with 8 percent lime in the current model. Therefore, the benefit of lime treatment can be identified by comparing the two models: one without a lime-treated layer, and the other with a lime-treated layer.

The lime-treated layer has a depth of 0.75 m and width of 6 m. According to available data in the literature (*11*), the matric suction compression index decreases on average from 0.0313 to 0.0156 because of the addition of 8 percent lime to the subgrade. The lime-treated layer is assumed to have an elastic modulus of 250 MPa and a tensile strength of 1 MPa. Poisson's ratio used for the lime stabilized layer is 0.2 in this model.

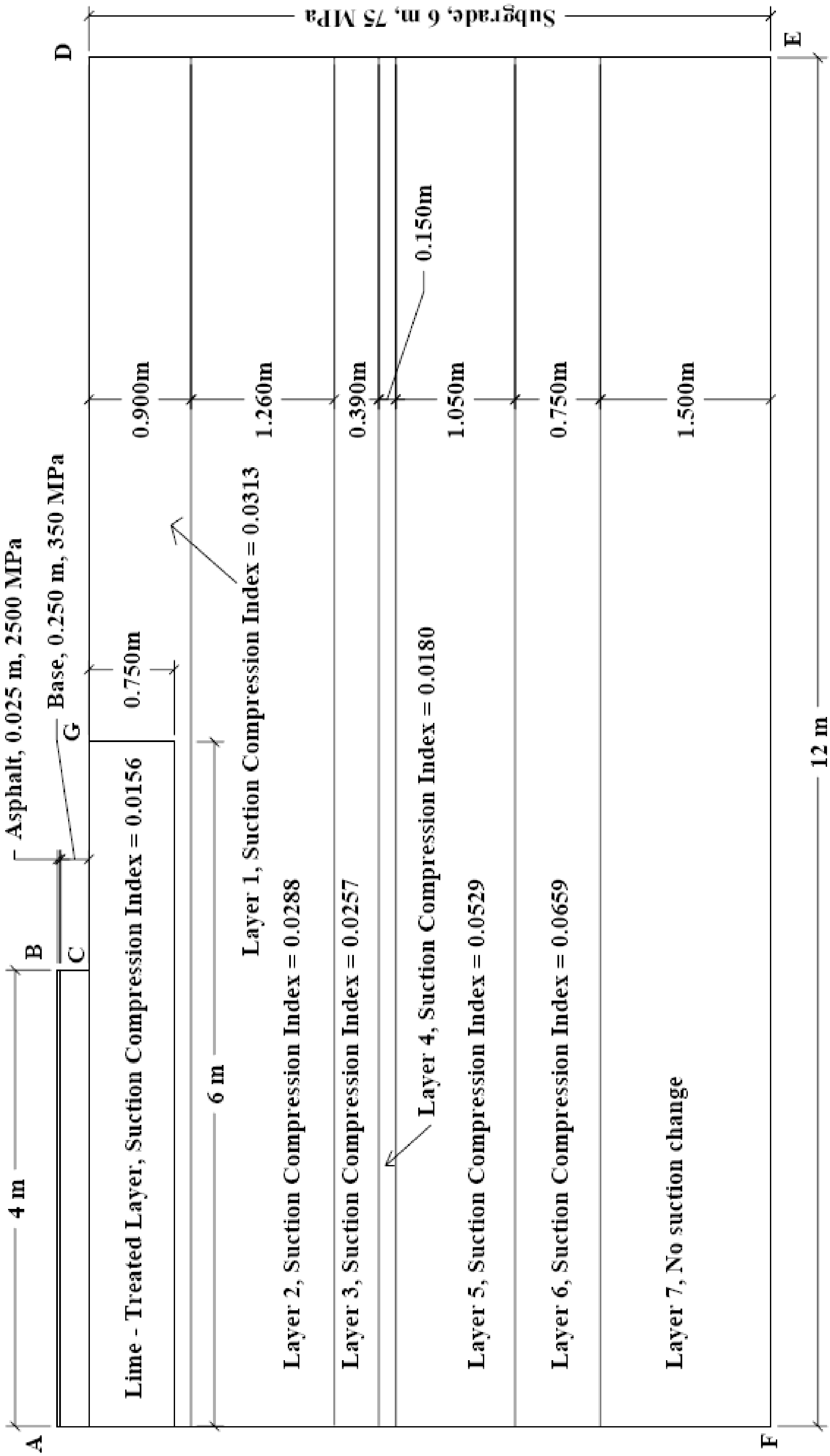


FIGURE 5 Model of Pavement with Lime-Treated Layer.

The boundary conditions of this model are:

- Zero horizontal displacement at edge AF in Figure 5;
- Zero displacement at edge FE in Figure 5;
- Zero horizontal displacement at edge DE in Figure 5; and
- Edge AB, BC and CD in Figure 5 are free to deform.

These boundary conditions were also applied to the pavement model without a lime-treated layer in the previous research (*19*). In addition, the matric suction changes of the subgrade soil in this model are also the same as those in the model without a lime-stabilized layer. The matric suction change in the subgrade is calculated based on the matric suction data in the literature (*11*). The determination of suction change has been detailed in previous research (*3, 19*). The logarithm of suction change is simulated by an equivalent temperature change using a thermal expansion model in ABAQUS since there is analogy between the soil shrinkage problem and the temperature change problem (*19*). The thermal expansion coefficient used in the thermal expansion model is negative one third of the suction compression index of each layer. Since the suction change varies in the subgrade soils, the subgrade is partitioned into a number of grids. The initial temperature of each grid is assigned to a value of zero; the final temperature of every grid is the logarithm of suction change at that location.

The purpose of using the same model constraints and loading in both models with and without lime treatment is to determine how the lime treatment changes the distribution and magnitude of tensile stresses induced by the same suction change. In the lime-treated layer, the increased elastic modulus (from 75 MPa to 250 MPa) results in increased tensile stress. On the other hand, the decrease of suction compression index contributes to the decrease of tensile stress. The combined effect of increased elastic modulus and decreased suction compression index is of special interest in this study.

To clearly illustrate the distribution of transverse tensile stresses in the upper part of the subgrade soil and the field soil, the calculation results are displayed in Figure 6 and Figure 7, respectively. As can be seen from Figure 6, the largest transverse tensile stress in the lime-treated layer, $\sigma_{11\max}$, develops at the interface of base and lime-treated layer close to point C. The magnitude of $\sigma_{11\max}$ is found to be approximately 0.38 MPa. Figure 7 shows another area with large transverse tensile stress in the field soil. This area is within the lime-stabilized layer and close to point G at the field soil surface. The largest tensile stress in this area is around 0.60 MPa. Since the tensile strength of the lime-stabilized soil can be as high as over 1 MPa, the shrinkage crack has a low probability of initiation in the lime-stabilized layer under the pavement when it is subjected to the same suction change.

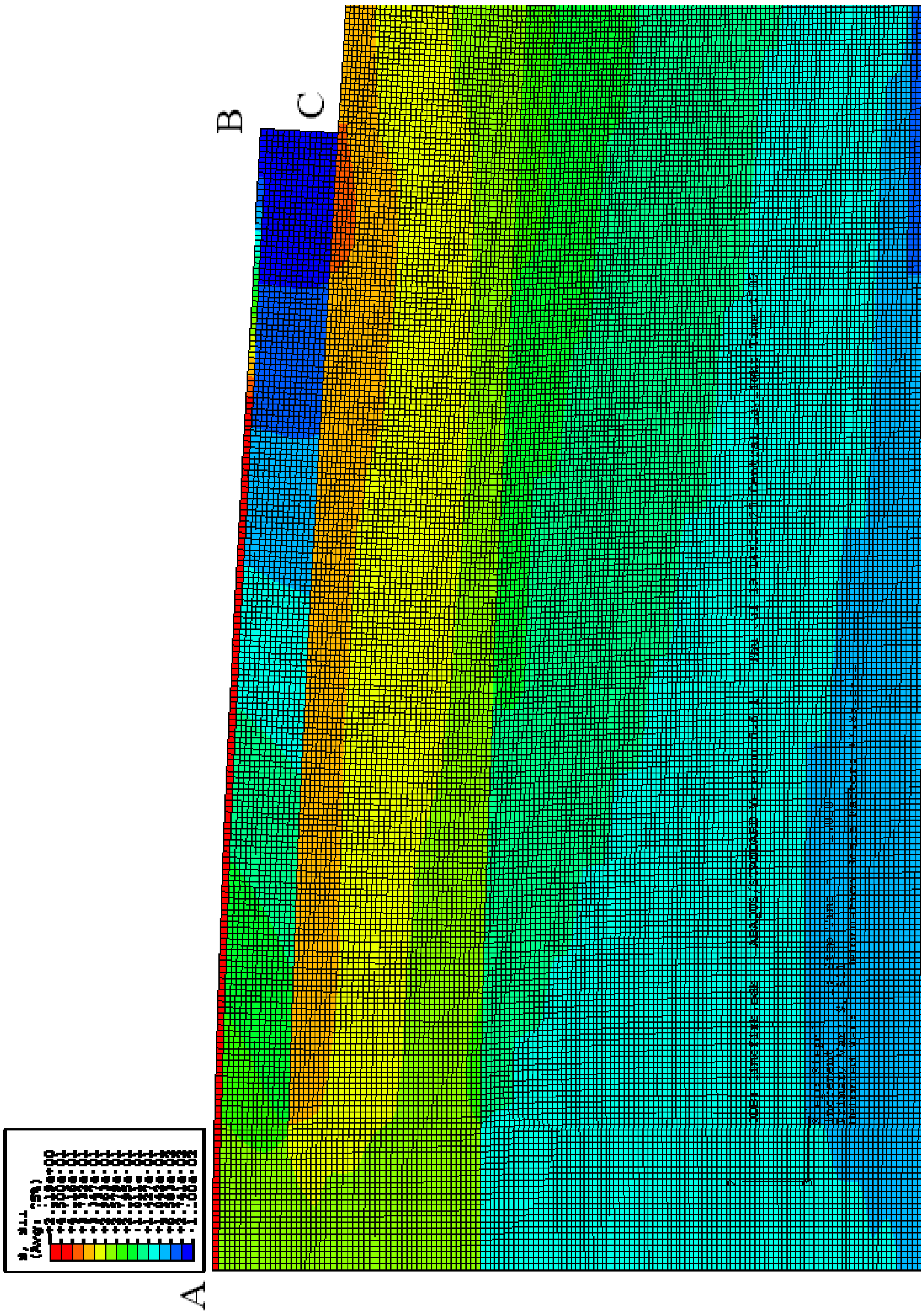


FIGURE 6 Transverse Stress Distribution in Pavement with Lime-Treated Layer (a).

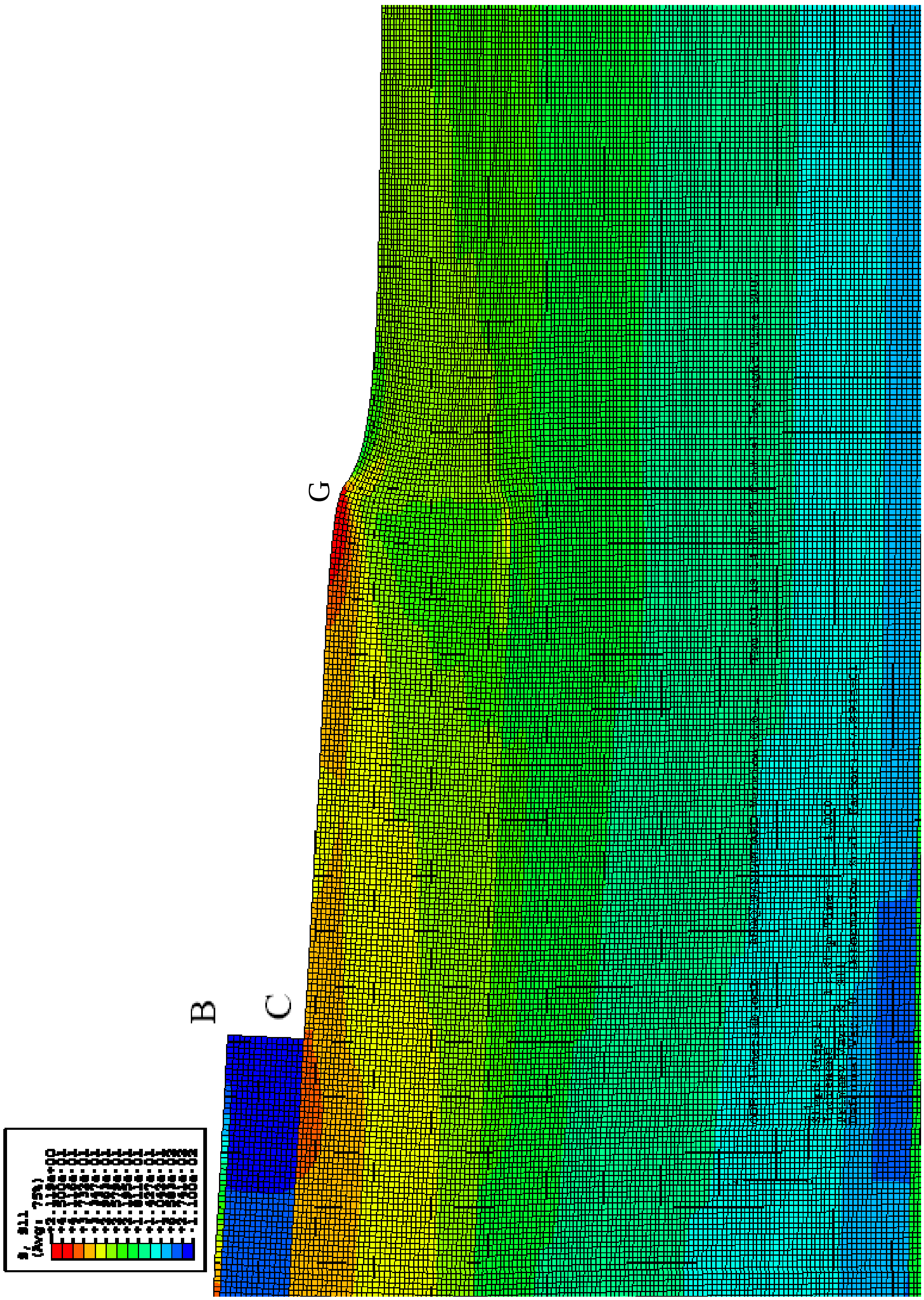


FIGURE 7 Transverse Stress Distribution in Pavement with Lime-Treated Layer (b).

However, tensile stresses are also found in the untreated subgrade soil and untreated field soil. The largest transverse tensile stress in the untreated subgrade soil has a magnitude of 0.22 MPa. It develops close to the vertical line at edge BC in Figure 5, and close to the interface of the lime-treated layer and the untreated soil. Transverse tensile stresses also develop at the surface of field soil (edge GD in Figure 5) with a range of 0.2 to 0.3 MPa. Shrinkage cracks have a high probability to initiate in these areas with transverse tensile stresses.

In a pavement without lime treatment, shrinkage cracks initiate right under the base layer, where the largest tensile shrinkage stresses are located, and propagate to the pavement surface (*3*, *19*). The calculation results in this study show that the inclusion of a lime-treated layer changes the location of initial shrinkage cracks. When treating the upper layer of subgrade and field soil with lime, the shrinkage cracks develop in the lower layer of the subgrade, and in the field soil which is not next to the pavement shoulder but further away from the pavement shoulder. The lime-treated layer becomes a "protection layer" which keeps the shrinkage cracks a certain distance away from the pavement.

IV. CRACK DEVELOPMENT IN UNTREATED SUBGRADE SOIL

A. Determination of Initial Crack

Since the untreated subgrade soil is subjected to large transverse tensile stresses (up to 0.22 MPa), an initial shrinkage crack with a length of 0.1 m is introduced to the untreated subgrade soil under the pavement. The horizontal distance between the initial crack and the pavement shoulder (edge BC in Figure 5) is assumed to be 0.2 m. To determine the critical vertical position of the initial crack, a number of trial cracks are placed at different vertical locations, while the horizontal coordinate and length of the trial cracks remain unchanged. The upper crack tip of the first trial crack is 0.01 m vertically away from the interface of the lime-treated layer and the untreated layer. The second trial crack is moved 0.01 m downward; the third trial crack is moved 0.02 m lower than the first trial crack; and so forth until the critical vertical position is determined.

All studied cracks are modeled as seams with specified crack tips and crack fronts in ABAQUS. A seam is defined as an edge in the ABAQUS. The seam is originally closed but can open during the calculation. ABAQUS places overlapping duplicate nodes along the seam when the mesh is generated. During the calculation, ABAQUS evaluates the stress intensity factors at every crack tip. Stress intensity factor is a parameter which quantifies the stress concentration at a crack tip. The magnitude of stress intensity factors depends on the applied loading, crack length and specimen geometry. A larger stress intensity factor indicates a higher level of stress concentration at the crack tip. According to the fracture mechanics theory, when the stress intensity factor is larger than the fracture toughness of a material, the crack is unstable and will propagate to release energy until equilibrium is reached; when the stress intensity factor is smaller than the fracture toughness, the crack remains stable. Fracture toughness is a constant material property and can be measured by experiment. The fracture toughness of the untreated soil is conservatively assumed to be 0.040 $MPa \cdot \sqrt{m}$, which is close to the lower value presented in the literature (*20*, *21*).

A total number of 7 trial cracks are studied in terms of the stress intensity factors at the crack tips. The results are summarized in Table 1, in which d is the vertical distance between the upper tip of the trial crack and the interface of the lime-treated layer and the untreated subgrade soil. The No. 6 trial crack is determined as the initial crack because the stress intensity factor at the upper crack tip is found to be the largest. Compared to the initial crack in the pavement model without lime-treated layer studied in the previous research (*3, 19*), the initial crack studied in this section shows smaller stress intensity factors at both crack tips, but they are still larger than the assumed fracture toughness of the subgrade soil (0.040 $MPa \cdot \sqrt{m}$).

TABLE 1 Stress Intensity Factors of Trial Cracks in Pavement with Lime-Treated Layer

Trial Crack No.	d (m)	K_I ($MPa \cdot \sqrt{m}$)	
		Upper Crack Tip	**Lower Crack Tip**
1	0.010	0.076	0.083
2	0.020	0.081	0.085
3	0.030	0.083	0.086
4	0.040	0.084	0.086
5	0.050	0.086	0.086
6	0.060	0.089	0.083
7	0.070	0.086	0.082

B. Development of Single Shrinkage Crack

Based on the above analysis, the initial crack is assumed to grow in both directions with an increment of 0.07 m so that the upper crack tip is within the lime-treated layer (0.01 m above the interface of lime-stabilized layer and the untreated layer, see Figure 8). This model is labeled "Model 1". Subsequently, the stress intensity factor of both crack tips is calculated in ABAQUS. The Mode I stress intensity factor (K_I) at the upper crack tip is 0.177 $MPa \cdot \sqrt{m}$, K_I at the lower crack tip is 0.116 $MPa \cdot \sqrt{m}$. If the fracture toughness of the lime-treated layer is larger than K_I at the upper crack tip, theoretically, the crack will remain stable.

However, the fracture toughness value of lime-treated soil has not been found in the literature. In a paper on the fracture properties of cement-stabilized soil (*22*), the researchers measured the plane strain fracture toughness of a number of cement-stabilized soil samples in terms of the critical stress intensity factor K_{IC} and the critical energy release rate J_{IC}. They found that the cement content was the primary controlling factor for toughness. With the standard compaction effort (ASTM D1557), the stabilized soil with a cement content of 15 percent by weight had an average K_{IC} of 0.230 $MPa \cdot \sqrt{m}$; K_{IC} decreased to 0.152 $MPa \cdot \sqrt{m}$ when the cement content was 10 percent; if cement content decreased to 5 percent, the average value of K_{IC} was 0.092 $MPa \cdot \sqrt{m}$. Since both lime stabilization and cement stabilization are typically used in Texas to improve the quality of soil in terms of strength and modulus, the lime-treated soil in this study is assumed to have similar properties to those of the cement-treated soil. Under this assumption, it is reasonable that the fracture toughness (K_{IC}) of the lime-treated soil has the same order as K_{IC} of the cement-treated soil.

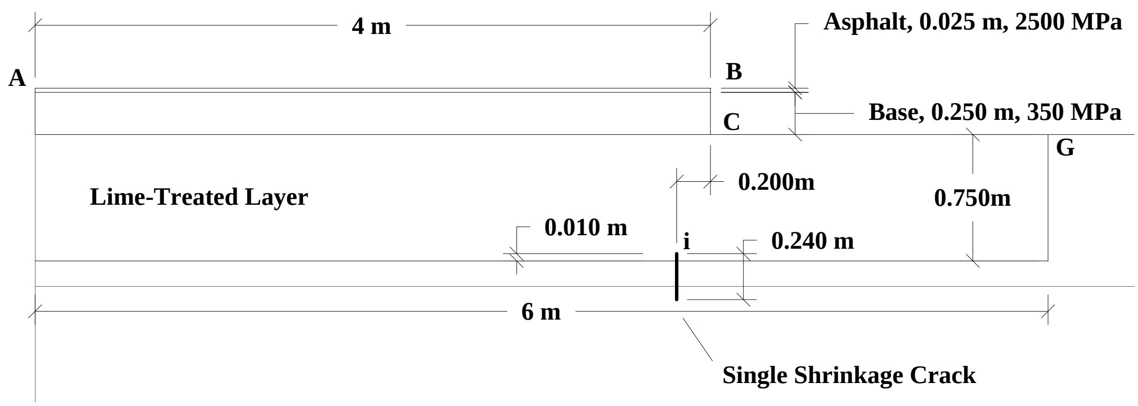


FIGURE 8 Single Shrinkage Crack in Subgrade Soil (Model 1).

If K_{IC} of the lime-stabilized soil has a value of approximate 0.150 $MPa \cdot \sqrt{m}$, the calculated stress intensity factor (0.177 $MPa \cdot \sqrt{m}$) is larger than K_{IC}. Theoretically, the crack is going to propagate in this condition. However, since the magnitude of the stress intensity factor is in the same order of K_{IC} value, it is possible that the crack will remain stable because the proposed model is extremely critical: first, only one vertical crack (Mode I crack) is considered in the untreated soil; second, the model does not include Mode II cracks. Based on the fracture mechanics theory (*23*), Mode I crack has an opening mode which corresponds to normal separation of crack under the effect of tensile stress applied normally to the crack plane; Mode II crack has a sliding mode which corresponds to the in-plane shearing of the crack in a direction normal to the crack front.

In engineering practice, there may be more than one Mode I crack developing in the untreated soil, and Mode II cracks may be present as well. For example, Figure 1 clearly illustrates multiple Mode I cracks and Mode II cracks in the untreated soil. In field observation, the initiation of macro cracks at the natural soil surface has spacing between 0.020 m to 0.024 m at the soil surface (*24*). The differential shrinkage in the soil leads to the Mode II fracture or an in-plane shear fracture mode where a crack propagates along the shear plane, as illustrated in Figure 9. The following section will address the situations with multiple shrinkage cracks developing in the subgrade.

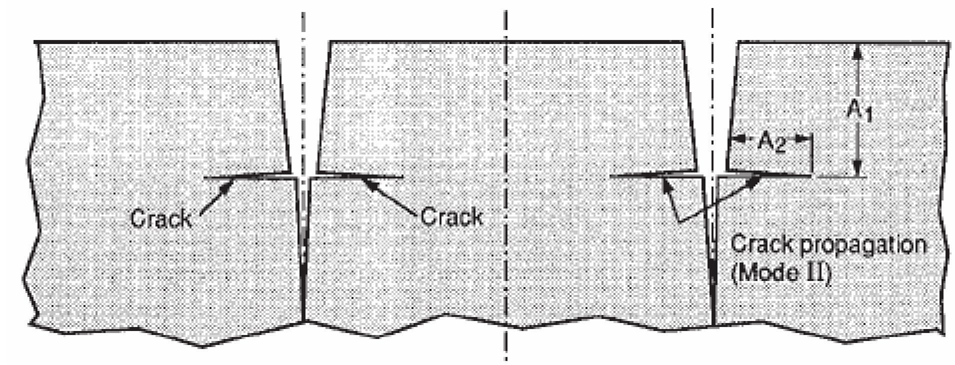
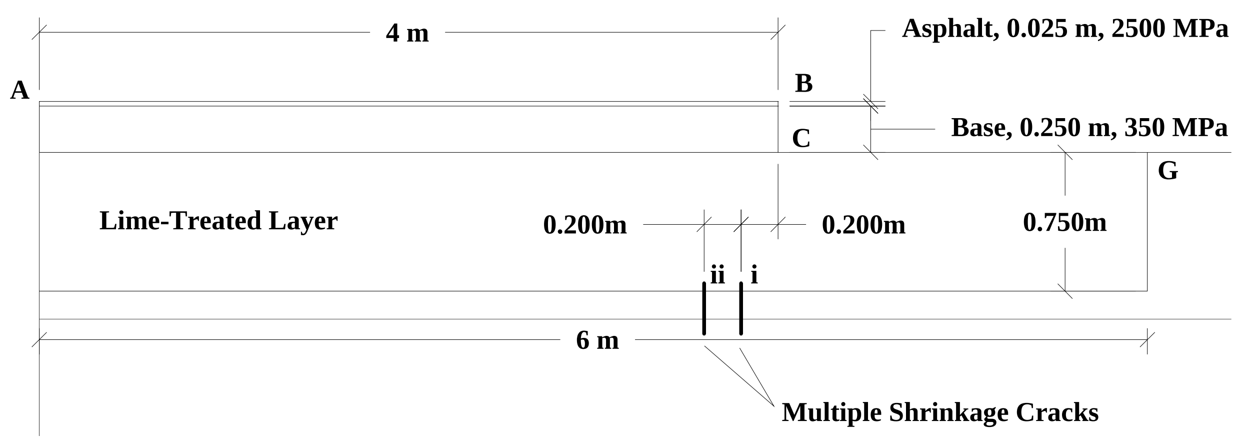


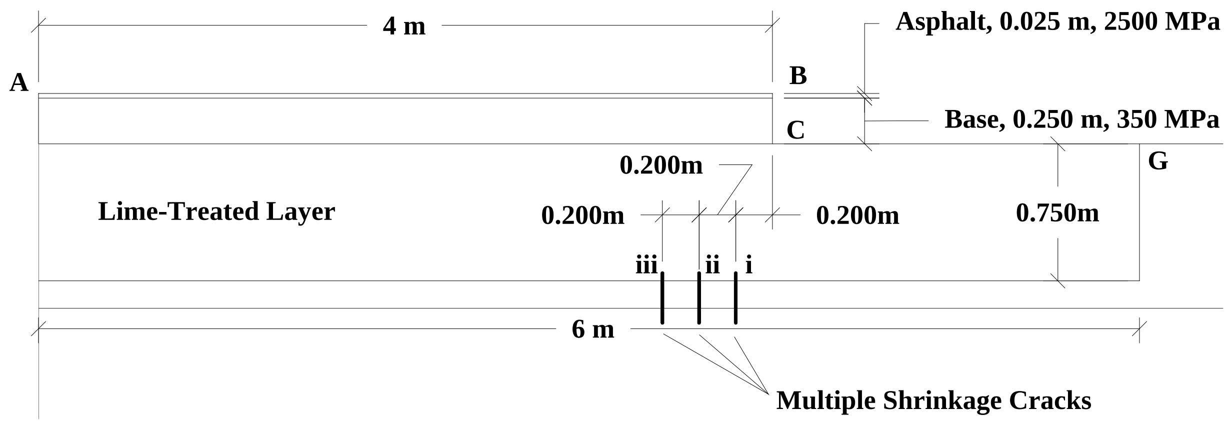
FIGURE 9 Mode II Crack in Shrinking Soil (*24*).

C. Development of Multiple Shrinkage Cracks

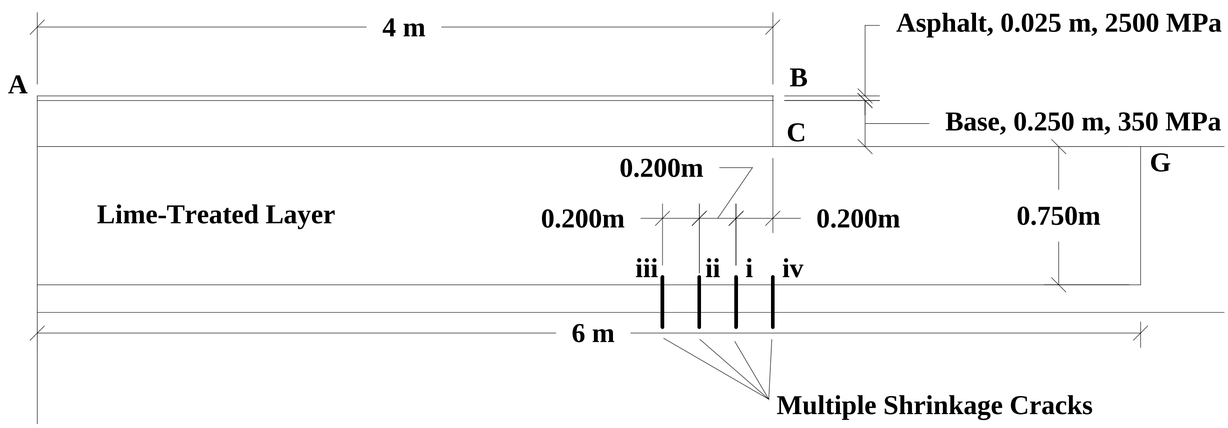
The increased number (more than one) of Mode I cracks and the existence of Mode II cracks release more strain energy induced by the differential suction change. When strain energy is released by multiple cracks simultaneously, the strain energy release rate at every crack tip should be less than that when a single crack released the strain energy. As a result, the stress concentration should be relieved at each crack tip of multiple cracks. To quantify the reduction in stress concentration, multiple Mode I cracks are introduced to the model of the pavement with a lime-treated layer. The models with different number of shrinkage cracks are studied individually. The models studied have 2 cracks, 3 cracks, 4 cracks and 5 cracks, respectively. To differentiate the studied models, the model with single shrinkage crack is labeled “Model 1”; the model with two cracks is “Model 2”; the model with three cracks is “Model 3”; and so on. Figure 10 presents the location and layout of the multiple shrinkage cracks in different models. In each model, the shrinkage cracks are parallel to the initial crack and have the same length and vertical location. The distance between two neighboring cracks is 0.2 m. The Model I stress intensity factors at crack tips are calculated in every model. The results are summarized in Table 2.



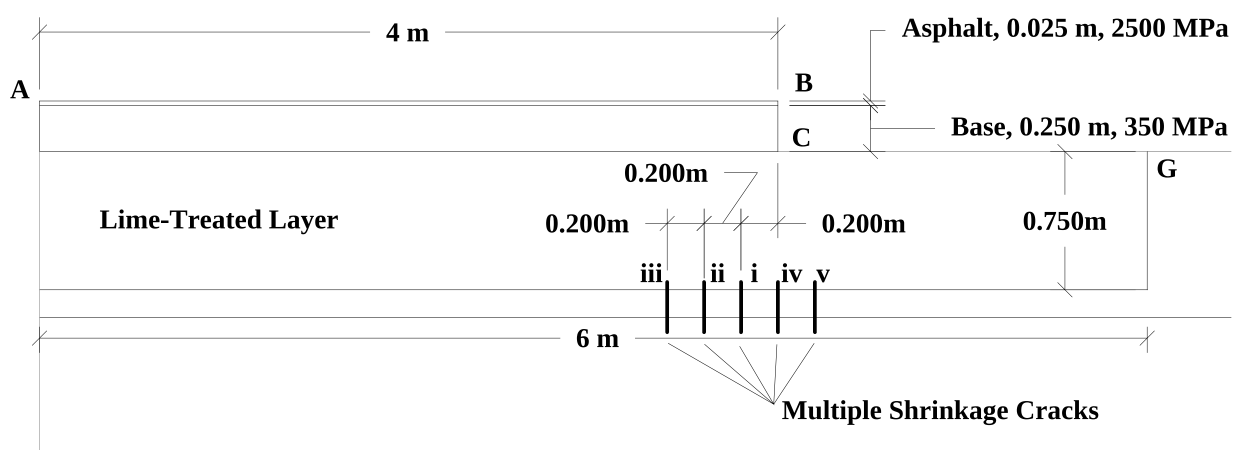
(a) Model 2: Number of Cracks = 2



(b) Model 3: Number of Cracks = 3



(c) Model 4: Number of Cracks = 4



(d) Model 6.5: Number of Cracks = 5

FIGURE 10 Multiple Shrinkage Cracks in Subgrade Soil.

TABLE 2 Stress Intensity Factors of Shrinkage Cracks

Label of Studied Model	Number of Cracks	Crack No.	K_I ($MPa \cdot \sqrt{m}$)	
			Upper Crack Tip	Lower Crack Tip
6.1	1	i	0.177	0.116
6.2	2	i	0.158	0.108
		ii	0.132	0.075
6.3	3	i	0.154	0.097
		ii	0.111	0.061
		iii	0.146	0.095
6.4	4	i	0.125	0.082
		ii	0.108	0.057
		iii	0.144	0.095
		iv	0.149	0.092
6.5	5	i	0.122	0.079
		ii	0.107	0.057
		iii	0.143	0.094
		iv	0.121	0.072
		v	0.150	0.091

The Mode I stress intensity factors of the cracks are compared among all studied cases with different number of shrinkage cracks, as shown in Figure 11. The single crack has a larger K_I at both crack tips than any one of the other cracks. When a second crack develops in the subgrade, K_I of Crack No. i decreases at both crack tips. As more cracks develop in the subgrade, K_I of Crack No. i continues to decrease because part of the strain energy is released by the other cracks. The same trend applies to the other cracks. For example, K_I of Crack No. ii decreases when the third crack develops in the subgrade. For each crack, the inclusion of an additional crack results in different levels of reduction in K_I, which means the decrease rate of K_I is not constant. In addition, the inclusion of a new crack has a different impact on every crack. When introducing Crack No. iv, K_I of Crack No. i decreases significantly at both crack tips, while Crack No. iii and Crack No. ii have only a slight reduction in K_I. The reason is that the decrease of K_I is dependent on the location of the new crack. The initiation of a new crack will considerably reduce K_I of its neighboring crack but will have little effect on the cracks a certain distance away from the new crack. In other words, if the new crack develops next to an existing crack, the existing crack will experience a significant reduction in K_I. Figure 11 gives good examples: the appearance of Crack No. v decreases the K_I of Crack No. iv significantly but has little impact on Crack No. iii; Crack No. ii has a clear reduction in K_I because of the occurrence of Crack No. iii, while Crack No. ii has very slight change in the magnitude of K_I with the initiation of Crack No. iv.

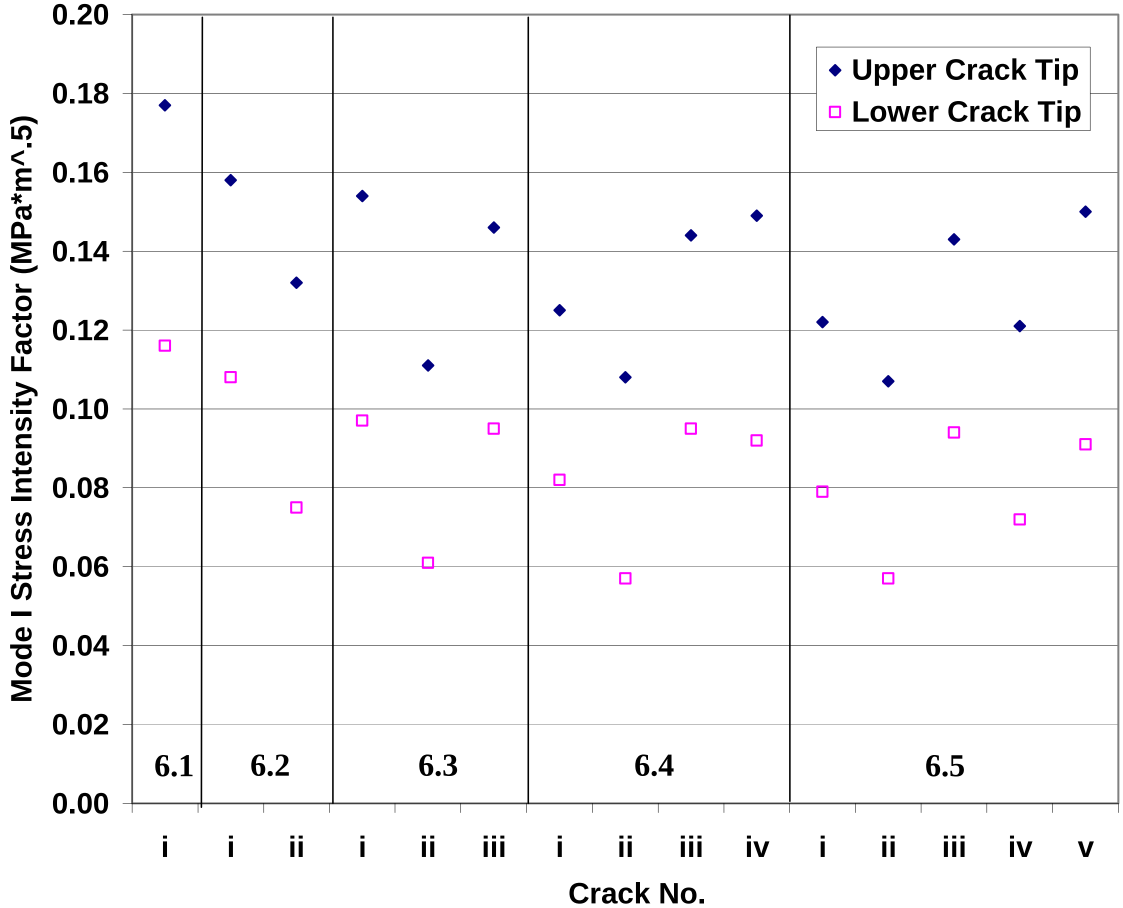


FIGURE 11 Comparison of Mode I Stress Intensity Factor in Single Model and Multiple Crack Models.

V. CONCLUSIONS

This paper investigates the mechanism of lime treatment controlling dry-land longitudinal crack at the pavement surface. The lime treatment can significantly change the properties of the natural soils through the processes of modification and stabilization. The lime-treated soil has lower plasticity index, smaller suction compression index and higher tensile strength. Finite element modeling results show that the tensile stress developing in the lime-treated layer is less than the tensile strength of the lime-stabilized soil. Therefore, shrinkage cracks are less likely to initiate in the lime-stabilized layer.

The possible location of the shrinkage crack initiation is in the untreated soil close to the bottom of the lime-treated layer, where tensile shrinkage stress exceeds the tensile strength of the untreated natural soil. The lime-treated layer is likely to make the shrinkage cracks stable because its fracture toughness has been considerably increased by the inclusion of lime. The theoretical calculation shows that, when the initial crack propagates into the lime-treated soil, the magnitude of the stress intensity factor at the upper crack tip is in the same order of the estimated fracture toughness of the lime-treated soil. The shrinkage crack is less likely to develop through the lime-treated soil with increased fracture toughness. If multiple shrinkage cracks are present in the subgrade, each crack tip has less stress concentration compared to the single shrinkage crack case. In other words, the occurrence of multiple shrinkage cracks reduces the stress intensity factor at every crack tip. This fact further reduces the probability of shrinkage crack propagation toward the pavement surface. Therefore, lime treatment is effective for the control of dry-land longitudinal cracks.

However, the reduced stress intensity factors are still in the same order of the fracture toughness of the lime-stabilized layer. This fact indicates that it is still possible for shrinkage cracks to propagate through the lime-treated layer. To further reduce the possibility of crack propagation, other effective methods such as geosynthetic reinforcement may be combined with lime treatment to prevent the shrinkage cracks from developing upward to the pavement surface. Since the geogrid can reduce the stress concentration at the crack tip (*2*), and the lime treatment is able to increase the fracture toughness of the soil, the design with both geogrid reinforcement and lime treatment may provide a safer and more conservative approach for the pavement structure over expansive subgrade. The modeling and analysis of pavement with a combination of the two methods are the subjects of ongoing research by the writers.

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