Research Article

Effect of the Dynamic Load on Stresses in a Deck Pavement with an Interlayer Contact Model

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Random surface roughness of bridge deck pavement just like random road surface roughness was simulated by the harmony superposition method in this paper. The dynamic load of vehicle was calculated by the random surface roughness of the deck pavement and the quarter-car model. A finite element model of a box girder bridge and its deck pavement was established, and the bonding condition between the adjacent layers was assumed to be contact bonding condition. The stress values of the asphalt concrete layer were calculated and analyzed when surface roughness condition, vehicle speed, and disengaging area changed. Results show that random surface roughness of deck pavement affected the stress trend of the asphalt concrete layer obviously. The appearance of disengaging area would increase the stress values of the asphalt concrete layer and the normal tensile stress value between the asphalt concrete layer and the waterproof layer. This would speed up the damage of the asphalt concrete layer and enlarge further the disengaging area.

1. Introduction

When a vehicle passes through asphalt concrete bridge deck pavement, vehicle vibration was excited due to surface roughness of the asphalt concrete bridge deck pavement. Additional dynamic loads [1, 2] were produced in the process of the vehicle movement because of surface roughness, which would give rise to stress increase of the asphalt concrete bridge deck pavement and exacerbate the damage of asphalt concrete bridge deck pavement. Furthermore, surface roughness of the pavement has a significant influence on driving comfort and driving safety [3]. Many researchers [4–11] had studied the response of road and bridge deck pavement under the action of dead loads, moving constant loads, and dynamic loads, and dynamic loads have been considered to be a better way to reflect surface roughness of road and bridge deck pavement. White noise filtering method, harmony superposition method, FFT method, and AR/ARMA method are four common types of models to produce surface roughness of road and bridge deck pavement, in which harmony superposition method and FFT method [12, 13] are considered to be simple, highly

accurate in computation, and based on rigorous mathematical foundation.

It is well known that asphalt concrete bridge deck pavement is multilayered composite system; thus, bonding condition between adjacent layers affects stress transmission and energy dissipation of bridge deck pavement structure. However, most previous researchers [14–18] assumed that the bonding condition between adjacent layers was full-bond and no sliding, and a few of them even assumed that the bonding condition between adjacent layers was no-bond. In fact, many researchers [19–22] agreed that the bonding condition between adjacent layers are not the two extreme conditions, and the real interlayer bonding condition lies somewhere between full-bond condition and no-bond condition. Therefore, there is a need to develop an interlayer bonding model of asphalt concrete bridge deck pavement to accurately predict the responses of asphalt concrete bridge deck pavement.

In this study, a box girder bridge and its asphalt concrete deck pavement were established by the ANSYS software. An interlayer contact model of asphalt concrete bridge deck pavement was constructed to obtain a more realistic and

FIGURE 1: A quarter-car model.

effective method to embody the interlayer bonding condition. Using the interlayer contact model, the extreme stress of the asphalt concrete layer was calculated by the ANSYS software under dynamic load of vehicle. The effect of surface roughness of asphalt concrete layer, vehicle speed, and bonding failure on stress value of the asphalt concrete layer were studied and analyzed.

2. Computing Method and Theory

2.1. Mathematical Model for a Quarter-Car. A quarter-car model as shown in Figure 1 is a classic vehicle vibration model in dynamical model, and it is effective for simulating dynamic load of vehicle. In the quarter-car model, the suspended mass is denoted by m_s and the mass of wheel is denoted by m_t . Their vertical displacements are denoted by x_s and x_t , respectively. The suspension stiffness and the tire stiffness are represented by k_s and k_t , respectively, and their damping is represented by c_s and c_t , respectively. The random surface roughness of the bridge deck pavement is a stochastic process denoted by x_g .

The equations of motion of the quarter-car model can be written as follows [23, 24]:

$$\begin{aligned} m_s \ddot{x}_s &= -k_s(x_s - x_t) - c_s(\dot{x}_s - \dot{x}_t), \\ m_t \ddot{x}_t &= k_s(x_s - x_t) + c_s(\dot{x}_s - \dot{x}_t) - k_t(x_t - x_g) - c_t(\dot{x}_t - \dot{x}_g). \end{aligned} \quad (1)$$

The dynamic load of vehicle acting on the bridge deck pavement is denoted by F_t , which is represented as follows:

$$F_t = F_d + F_m, \quad (2)$$

$$F_d = k_t(x_t - x_g) + c_t(\dot{x}_t - \dot{x}_g), \quad (3)$$

$$F_m = (m_s + m_t)g, \quad (4)$$

where g is the acceleration of gravity, F_d is the additional dynamic load, and the parameters used for the quarter-car model are listed in Table 1 [24].

2.2. Simulation of Random Road Surface Roughness. The random surface roughness x_g of asphalt concrete bridge deck pavement can be described by a zero-mean, real-valued, and stationary Gaussian process. In the researches [4, 23], x_g can be expressed as

TABLE 1: Parameters of the quarter-car model.

Parameters	m_s (kg)	m_t (kg)	k_s (N/m)	k_t (N/m)	c_s (N·s/m)	c_t (N·s/m)
Value	4450	550	1×10^6	1.5×10^4	1.75×10^6	2×10^3

TABLE 2: Estimation scale of surface roughness condition of bridge deck pavement.

Condition	Very good	Good	Average	Poor	Very poor
IRI value	0	2	4 or 6	8	10

IRI=0 means an absolutely smooth longitudinal surface; IRI=2 means a good surface condition; IRI=6 means a moderately rough surface; IRI=10 means a highly rough surface.

$$x_g(t) = \sum_{k=1}^N A_k \cos(\omega_k t + \phi_k), \quad (5)$$

where ϕ_k ($k = 1, 2, 3, \dots, N$) are independent random phase angles with uniformly distributed in the range $[0, 2\pi]$. N is the total number of terms used to build up the road surface roughness. ω_k is computed by

$$\begin{aligned} \omega_k &= 2\pi v n_k, \\ n_k &= n_1 + k \Delta n \quad (k = 1, 2, 3, \dots, N), \end{aligned} \quad (6)$$

$$\Delta n = \frac{(n_r - n_1)}{N},$$

where n_k is the spatial frequency within the interval $[n_1, n_r]$. n_1 and n_r are the lower and upper cut-off spatial frequencies (m^{-1}), respectively. v is a constant vehicle speed. A_k is computed by

$$A_k^2 = 4G_d(n_k) \Delta n \quad (k = 1, 2, 3, \dots, N), \quad (7)$$

where $G_d(n)$ is the power spectral density function, according to the research, which can be expressed as [23]

$$G_d(n) = \begin{cases} 1.63 K_0 \left(\frac{n}{n_0} \right)^{-2} \text{IRI}^2, & \text{for } n_1 \leq n \leq n_r \\ 0, & \text{elsewhere,} \end{cases} \quad (8)$$

where $K_0 = 10^{-6} \text{ (m}^3\text{)}$, $n_0 = 0.1 \text{ (m}^{-1}\text{)}$. IRI is the international roughness index (m/km). According to Paterson's research [25], the surface roughness condition of bridge deck pavement can be divided into six degrees, and its values are listed in Table 2.

2.3. Interlayer Contact Model. For simulating the real interlayer bonding condition, the bonding condition between adjacent layers was assumed to be contact bonding condition [26]. The concept of interlayer contact model can be illustrated in Figure 2. In the interlayer contact model, the bottom of asphalt concrete layer and the top of waterproof layer are covered with contact element and target element, respectively, and contact element and target element are

FIGURE 2: Modeling for contact bonding condition.

connected by translational spring element at the corresponding nodes. The bottom of waterproof layer and the top of cement concrete layer are covered with target element and contact element, respectively, and contact element and target element are also connected by translational spring element at the corresponding nodes. The bonding condition between the cement concrete layer and the box girder bridge is assumed to be full-bond condition.

The contact bonding condition is different from the full-bond condition and the no-bond condition. In the contact bonding condition, the stress transfer between contactor surface and target surface is nonlinear. The springs between the contactor surface and the target surface are used to measure the compression length, and the compression length will be used to calculate the stress at the corresponding nodes according to contact algorithm [27]. The contact algorithm has the characteristic of highly non-linearity and conducted and completed by iterative calculation. In elastic model, there are the same stress and strain at the corresponding nodes. In the interlayer contact model, the stress and strain at the corresponding nodes are calculated by the contact algorithm.

In the interlayer contact model, the shear stress transfer between the adjacent layers follows the Coulomb friction model [27]:

$$\tau_{lim} = \mu P + b, \quad (9)$$

$$|\tau| \leq \tau_{lim}, \quad (10)$$

where τ_{lim} is ultimate shear stress, μ is the sliding friction coefficient, and it equals 0.5 in this paper, P is the contact compressive stress in normal direction, b is the cohesion between the adjacent layer, and τ is the equivalent shear stress. In (9), if μ equals 0 or P equals 0, the cohesion b still exists; however, if b equals 0, two adjacent layers show cohesive failure. In (10), when $|\tau|$ between two adjacent layers is less than or equal τ_{lim} , the two layers keep sticking, or the two layers start to slide.

As vehicle load chronically and repeatedly acted on the asphalt concrete deck pavement, disengaging area (bonding failure) might occur between the adjacent layers. If disengaging area existed between the adjacent layers, the cohesion b there was assumed to be 0 in the interlayer contact model. Five types of disengaging area are shown in Figure 3, where the dashed box represents the load position on the asphalt concrete surface, and the shaded area means the disengaging area between the asphalt concrete

FIGURE 3: Type of disengaging area under loading position (unit: m²).

layer and the waterproof layer. The disengaging area in type “D0” was none, which means that there is no disengaging area between the asphalt concrete layer and the waterproof layer.

2.4. Computing Model and Simulation of Dynamic Load of Vehicle. In this paper, a box girder bridge with a length of 20 meters and its asphalt concrete bridge deck pavement were established by the ANSYS software, and their cross section size is shown in Figure 4. The asphalt concrete surface layer, the cement concrete layer, and the box girder bridge were simulated by solid45 element, and the waterproof layer was simulated by shell41 element. The surface layer was subjected to double-wheel load of 50 kN, and the tire ground contact area is 0.5 m × 0.2 m. The driving direction was the same with the direction of z-axis, and the dynamic load of vehicle moved from one end of the bridge to the other end of the bridge. The load moved forward 0.2 m in each iteration step. The parameters used for the finite element computing model are listed in Table 3, and the bonding condition between adjacent layers was assumed to be the contact bonding condition as shown in Figure 2.

In this study, the parameters n_1 and n_r are equal to 0.13 and 2.83, respectively, and the parameter N is equal to 100. Firstly, the values of the surface roughness of bridge deck pavement x_g were simulated by the MATLAB software according to (5). Then, (1) was solved, the values of additional load F_d were obtained according to (3), and the values of dynamic load F_t were obtained according to (2). Finally, for IRI = 2, 6, and 10, simulation of dynamic load of vehicle under different vehicle speed is shown in Figures 5–7, respectively. According to these figures, when vehicle speed increased, the extreme value of dynamic load of vehicle also increased under a certain IRI. When IRI increased, the extreme value of dynamic load of vehicle also increased under a certain vehicle speed.

FIGURE 4: Cross section of box girder bridge (unit: m).

TABLE 3: Parameters for the finite element computing model.

	Elastic modulus (GPa)	Thickness (m)	Poisson's ratio
Asphalt concrete layer	1.20	0.080	0.25
Waterproof layer	0.15	0.002	0.30
Cement concrete layer	30.00	0.080	0.15
Box girder bridge	36.00	—	0.15

FIGURE 5: Simulation of dynamic load of vehicle when IRI = 2.

Built the interlayer contact model by the finite element software ANSYS and input the values of F_t at every iteration as the vehicle load move forward. When all the iterations were conducted completely, the maximum stresses of the asphalt concrete layer were obtained by the ANSYS software. In this paper, the maximum transverse tensile stress $\sigma_{x\max}$,

the maximum longitudinal tensile stress $\sigma_{z\max}$, the maximum transverse shear stress $\tau_{xy\max}$, and the maximum longitudinal shear stress $\tau_{yz\max}$ of the asphalt concrete layer would be calculated by the ANSYS software for analyzing the stress of asphalt concrete layer and slide between adjacent layers, and the maximum normal tensile stress $\sigma_{y\max}$

FIGURE 6: Simulation of dynamic load of vehicle when IRI = 6.

FIGURE 7: Simulation of dynamic load of vehicle when IRI = 10.

FIGURE 8: Stress values of the asphalt concrete layer when IRI = 0.

between the asphalt concrete layer and the waterproof layer would also be calculated by the ANSYS software.

3. Stress Calculation and Analysis for Asphalt Concrete Deck Pavement

3.1. Influence of IRI and Vehicle Speed on Stress of the Asphalt Concrete Layer. Stress value of the asphalt concrete layer which was calculated by the contact bonding model is shown in Figure 8 under different vehicle speeds when IRI = 0. When the vehicle speed increased from 15 m/s to 35 m/s, the maximum transverse tensile stress $\sigma_{x\max}$ of the asphalt concrete layer decreased by 1.97%, the maximum longitudinal tensile stress $\sigma_{z\max}$ of the asphalt concrete layer decreased by 2.49%, the maximum transverse shear stress $\tau_{xy\max}$ of the asphalt concrete layer decreased by 1.32%, and the maximum longitudinal shear stress $\tau_{yz\max}$ of the asphalt concrete layer all decreased by 1.92%. This means that the stress value of the asphalt concrete layer would gradually decrease without consideration of the surface roughness of bridge deck pavement when rising driving speed gradually.

Stress value of the asphalt concrete layer which was calculated by the contact bonding model is shown in Figure 9 under different vehicle speeds when IRI = 2. When the vehicle speed increased from 15 m/s to 35 m/s, the maximum transverse tensile stress $\sigma_{x\max}$ of the asphalt concrete layer increased by 11.48% and the maximum longitudinal tensile stress $\sigma_{z\max}$, the maximum transverse shear stress $\tau_{xy\max}$, and the maximum longitudinal shear stress $\tau_{yz\max}$ of asphalt concrete layer all increased first and then decreased. When the vehicle speed was 20 m/s, the maximum longitudinal tensile stress $\sigma_{z\max}$, the maximum transverse shear stress $\tau_{xy\max}$, and the maximum longitudinal shear stress $\tau_{yz\max}$ of the asphalt concrete layer reached their maximum value, respectively. When the vehicle speed increased from 20 m/s to 30 m/s, the maximum longitudinal tensile stress $\sigma_{z\max}$, the maximum transverse shear stress $\tau_{xy\max}$, and the maximum longitudinal shear stress $\tau_{yz\max}$ of the asphalt concrete layer decreased obviously.

FIGURE 9: Stress values of the asphalt concrete layer when IRI = 2.

FIGURE 10: Stress values of the asphalt concrete layer when IRI = 6.

Compared with Figures 8 and 9, the maximum tensile stress value of the asphalt concrete layer when IRI = 2 was bigger than the maximum tensile stress value of the asphalt concrete layer when IRI = 0 under the same vehicle speed. Stress trend of asphalt concrete layer when IRI = 2 was different from stress trend of asphalt concrete layer when IRI = 0 with consideration of the road surface roughness.

Stress value of asphalt concrete layer which was calculated by the contact bonding model is shown in Figure 10 under different vehicle speeds when IRI = 6. When the vehicle speed increased from 15 m/s to 35 m/s, the maximum transverse tensile stress $\sigma_{x\max}$ of asphalt concrete layer increased by 46.48%, the maximum longitudinal tensile stress $\sigma_{z\max}$ of asphalt concrete layer increased by 25.37%, and the maximum transverse shear stress $\tau_{xy\max}$ and the maximum longitudinal shear stress $\tau_{yz\max}$ of asphalt concrete layer all increased first and then decreased. When the vehicle speed was 20 m/s, the maximum transverse shear stress $\tau_{xy\max}$ and the maximum longitudinal shear stress $\tau_{yz\max}$ of asphalt concrete layer reached their maximum value, respectively.

FIGURE 11: Stress values of the asphalt concrete layer when IRI = 10.

When the vehicle speed increased from 20 m/s to 25 m/s, the maximum transverse tensile stress $\sigma_{x\max}$ and the maximum longitudinal tensile stress $\sigma_{z\max}$ of asphalt concrete layer increased obviously and the maximum transverse shear stress $\tau_{xy\max}$ and the maximum longitudinal shear stress $\tau_{yz\max}$ of asphalt concrete layer decreased obviously. When vehicle speed increased from 25 m/s to 35 m/s, the maximum transverse tensile stress $\sigma_{x\max}$ and the maximum longitudinal tensile stress $\sigma_{z\max}$ of asphalt concrete layer increased slowly and the maximum transverse shear stress $\tau_{xy\max}$ and the maximum longitudinal shear stress $\tau_{yz\max}$ of asphalt concrete layer decreased slowly. Compared with Figures 9 and 10, the maximum tensile/shear stress value of the asphalt concrete layer when IRI = 6 was bigger than the maximum tensile/shear stress value of the asphalt concrete layer when IRI = 2 under the same vehicle speed. The maximum shear stress trend of asphalt concrete layer when IRI = 6 was same with the maximum shear stress trend of asphalt concrete layer when IRI = 2.

Stress value of asphalt concrete layer which was calculated by the contact bonding model is shown in Figure 11 under different vehicle speeds when IRI = 10. When the vehicle speed increased from 15 m/s to 35 m/s, the maximum transverse tensile stress $\sigma_{x\max}$ of asphalt concrete layer increased by 65.80%, the maximum longitudinal tensile stress $\sigma_{z\max}$ of asphalt concrete layer increased by 40.77%, and the maximum transverse shear stress $\tau_{xy\max}$ and the maximum longitudinal shear stress $\tau_{yz\max}$ of asphalt concrete layer all increased firstly, then decreased, and finally increased. When the vehicle speed was 20 m/s, the maximum transverse shear stress $\tau_{xy\max}$ and the maximum longitudinal shear stress $\tau_{yz\max}$ of asphalt concrete layer reached their maximum value, respectively. When vehicle speed increased from 20 m/s to 25 m/s, the maximum transverse tensile stress $\sigma_{x\max}$ and the maximum longitudinal tensile stress $\sigma_{z\max}$ of asphalt concrete layer increased obviously. When vehicle speed increased from 25 m/s to 35 m/s, the maximum transverse tensile stress $\sigma_{x\max}$ and the maximum longitudinal tensile stress $\sigma_{z\max}$ of asphalt concrete layer increased gradually and the maximum transverse shear stress $\tau_{xy\max}$ of

asphalt concrete layer increased slowly. Compared with Figures 10 and 11, the maximum tensile/shear stress value of the asphalt concrete layer when IRI = 10 was bigger than the maximum tensile/shear stress value of the asphalt concrete layer when IRI = 6 under the same vehicle speed. The maximum tensile stress trend of asphalt concrete layer when IRI = 10 was same with the maximum tensile stress trend of asphalt concrete layer when IRI = 6. In summary, when the IRI value increased, the stress values of the asphalt concrete layer all increased. When the IRI value was definite, the maximum tensile stress values of the asphalt concrete layer were bigger than the maximum tensile stress values of the asphalt concrete layer at the same vehicle speed.

3.2. Stress of the Asphalt Concrete Layer When Disengaging Area Existing. Assumed that IRI = 2 and the vehicle speed was 25 m/s, the dynamic load of vehicle acted on asphalt concrete surface is shown in Figure 5. When five types of disengaging area as shown in Figure 3 appeared, respectively, between the asphalt concrete layer and the waterproof layer, the stress values of the asphalt concrete layer which were calculated by the contact bonding model are listed in Table 4. Comparing the stress values of the asphalt concrete layer at the type “D0” with the stress values of the asphalt concrete layer at the other type, the maximum transverse tensile stress $\sigma_{x\max}$ and the maximum longitudinal tensile stress $\sigma_{z\max}$ values of the asphalt concrete layer all increased, and the maximum normal tensile stress $\sigma_{y\max}$ values between the asphalt concrete layer and the waterproof layer increased obviously when the type of disengaging area changed from the type “D0” to the type “D1.” The maximum transverse shear stress $\tau_{xy\max}$ of the asphalt concrete layer was almost not changed under different types of disengaging area. Except for the maximum transverse shear stress $\tau_{xy\max}$ of the asphalt concrete layer, the other maximum stress values of the asphalt concrete layer all increased with the disengaging area increased. When the disengaging area was same, such as the type “D1” and the type “D3,” or the type “D2” and the type “D4,” the more the disengaging area under the load was, the bigger the maximum tensile stress values of the asphalt concrete layer was (Table 4). Comparing the stress values of the asphalt concrete layer at the type “D1” with the stress values of the asphalt concrete layer at the type “D4,” there were the same disengaging area under the load, but the maximum tensile stress values and the maximum longitudinal shear stress $\tau_{yz\max}$ of asphalt concrete layer at the type “D4” were bigger than the corresponding stress values of the asphalt concrete layer at the type “D1” in Table 4.

Assuming that IRI = 6 and the vehicle speed was 25 m/s, the dynamic load of vehicle acted on asphalt concrete surface is shown in Figure 6. When five types of disengaging area as shown in Figure 3 appeared, respectively, between the asphalt concrete layer and the waterproof layer, the stress values of the asphalt concrete layer which were calculated by the contact bonding model are listed in Table 5. Comparing the stress values of the asphalt concrete layer at the type “D0”

TABLE 4: Stress value of the asphalt concrete layer when IRI = 2.

	$\sigma_{x\max}$ (MPa)	$\sigma_{y\max}$ (MPa)	$\sigma_{z\max}$ (MPa)	$\tau_{xy\max}$ (MPa)	$\tau_{yz\max}$ (MPa)
Type D0	0.162697	0.051349	0.173667	0.125011	0.080908
Type D1	0.200714	0.256144	0.230054	0.125010	0.080923
Type D2	0.329370	0.295752	0.343371	0.125008	0.116269
Type D3	0.191053	0.255890	0.181542	0.125010	0.080923
Type D4	0.276347	0.290198	0.241429	0.125008	0.103085

TABLE 5: Stress value of the asphalt concrete layer when IRI = 6.

	$\sigma_{x\max}$ (MPa)	$\sigma_{y\max}$ (MPa)	$\sigma_{z\max}$ (MPa)	$\tau_{xy\max}$ (MPa)	$\tau_{yz\max}$ (MPa)
Type D0	0.274626	0.087097	0.297205	0.161153	0.092727
Type D1	0.339848	0.430552	0.394346	0.171558	0.126995
Type D2	0.556346	0.497758	0.585864	0.206075	0.194432
Type D3	0.323336	0.430185	0.312521	0.190524	0.097266
Type D4	0.456856	0.479859	0.413859	0.186459	0.170920

TABLE 6: Stress value of the asphalt concrete layer when IRI = 10.

	$\sigma_{x\max}$ (MPa)	$\sigma_{y\max}$ (MPa)	$\sigma_{z\max}$ (MPa)	$\tau_{xy\max}$ (MPa)	$\tau_{yz\max}$ (MPa)
Type D0	0.386547	0.122836	0.422160	0.227311	0.130166
Type D1	0.478958	0.604918	0.558667	0.240900	0.177892
Type D2	0.783280	0.699711	0.828351	0.290451	0.272589
Type D3	0.455583	0.600399	0.443535	0.266538	0.138615
Type D4	0.637275	0.669413	0.586323	0.260713	0.238741

with the stress values of the asphalt concrete layer at the other type, the maximum stress values of the asphalt concrete layer all increased, and the maximum normal tensile stress $\sigma_{y\max}$ values between the asphalt concrete layer and the waterproof layer increased obviously when the type of disengaging area changed from the type “D0” to the type “D1.” Comparing the type “D0,” the type “D1,” and the type “D2” in Table 5, when the disengaging area increased, the stress values of the asphalt concrete layer increased. When the disengaging area was same, such as the type “D1” and the type “D3,” or the type “D2” and the type “D4,” the more the disengaging area under the load was, the bigger the maximum tensile stress values of the asphalt concrete layer was (Table 4). Comparing the stress values of the asphalt concrete layer at the type “D1” with the stress values of the asphalt concrete layer at the type “D4,” there were the same disengaging area under the load, but the maximum tensile/shear stress values of the asphalt concrete layer at the type “D4” were bigger than the corresponding stress values of the asphalt concrete layer at the type “D1” in Table 5. Comparing the stress values of the asphalt concrete layer in Tables 4 and 5, the stress values of the asphalt concrete layer in Table 5 were bigger than the stress values of the asphalt concrete layer in the Table 4 at the corresponding type of disengaging area.

Assuming that IRI = 10 and the vehicle speed was 25 m/s, the dynamic load of vehicle acted on asphalt concrete surface is shown in Figure 7. When five types of disengaging area as shown in Figure 3 appeared, respectively, between the asphalt concrete layer and the waterproof layer, the stress

values of the asphalt concrete layer which were calculated by the contact bonding model are listed in Table 6. The stress values of the asphalt concrete layer all increased with the type of the disengaging area increased from the type “D0” to the type “D2,” and the maximum normal tensile stress $\sigma_{y\max}$ values between the asphalt concrete layer and the waterproof layer increased obviously when the type of disengaging area changed from the type “D0” to the type “D1.” Comparing the stress values of the asphalt concrete layer at the type “D1” with the stress values of the asphalt concrete layer at the type “D4,” there was the same disengaging area under the load, but the maximum tensile/shear stress values of the asphalt concrete layer at the type “D4” were bigger than the corresponding stress values of the asphalt concrete layer at the type “D1” in Table 6. Comparing the stress values of the asphalt concrete layer in Tables 5 and 6, the stress values of the asphalt concrete layer in Table 6 were bigger than the stress values of the asphalt concrete layer in Table 5 at the corresponding type of disengaging area.

In summary, when the disengaging area appeared between the asphalt concrete layer and the waterproof layer, the tensile stress values of the asphalt concrete layer all increased. The more the disengaging area under the vehicle load was, the bigger the stress values of the asphalt concrete layer were. When the disengaging area was the same, the relative position between the disengaging area and the vehicle load affected the stress values of the asphalt concrete layer. The normal tensile stress value between the asphalt concrete layer and the waterproof layer also increased when the disengaging area increased.

4. Conclusions

The stress values of the asphalt concrete layer were calculated and analyzed when surface roughness of bridge deck pavement, vehicle speed, and disengaging area changed under the dynamic load of vehicle when the bonding condition of the adjacent layers was assumed to be the contact bonding condition. The following conclusions can be drawn:

- (1) An interlayer contact model of asphalt concrete bridge deck pavement was constructed to obtain a more realistic and effective method to embody the interlayer bonding condition. The interlayer contact model is used for not only concrete bridge deck pavement, but also steel bridge deck pavement. Whether the bonding failure between adjacent layers appears or not, the interlayer contact model can still be used to calculate the stresses of asphalt concrete layer.
- (2) The surface roughness of asphalt concrete layer affects the stress values of the asphalt concrete layer. Measures should be taken to prevent the surface roughness of asphalt concrete layer from "Good" to "Very poor" in order to reduce the tensile/shear stress values of the asphalt concrete layer. When the surface roughness of asphalt concrete layer changed from "Good" to "Very poor," vehicle speed must be restricted to abate the additional dynamic load and protect the asphalt concrete layer.
- (3) The disengaging area appeared between the asphalt concrete layer and the waterproof layer resulted in the increase of the stress values of asphalt concrete layer, which would accelerate the damage of the asphalt concrete layer and enlarge further the disengaging area. Therefore, bonding strength between adjacent layers must be improved to avoid the appearance of bonding failure.

Data Availability

The data used to support the findings of this study are available from the corresponding author upon request.

Conflicts of Interest

The authors declare that they have no conflicts of interest.

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