4. INVESTIGATION OF INTERFACE DEBONDING DUE TO FATIGUE USING ACCELERATED LOADING TEST

The fracture of the interface bond happens gradually due to fatigue. The primary traffic-related mechanism of such fatigue is depicted in Figure 1. When a traveling wheel comes across a joint, the asphalt on the loaded side of the joint is compressed resulting in the downward deflection of the asphalt on the loaded side of the joint, due to the continuous nature of the asphalt layer. However, the concrete overlay on the unloaded side tends to remain its position, causing a tendency of differential deflection between the concrete overlay and the underlying asphalt on the unloaded side of the joint. The repetitive application of such wheel loads will result in fatigue damage of the interface bond and eventually crack the interface.

![Figure 1 Mode-I debonding of the interface due to repetitive traffic loads.](image)

The fatigue of the interface under this type of load can be described using Paris’ law. However, empirical coefficients in Paris’ law have to be calibrated before they can be used in design. Fatigue tests of the interface debonding were carried out on large whitetopping slabs using the accelerated loading facility (ALF) at the University of Pittsburgh. ALF testing is a good compromise between field testing and small-scale laboratory testing. It is always expensive and time consuming to conduct a field test. Although the results from a field test reflect realistic conditions, e.g. real climate, real traffic, etc.; processing the data is often very complicated because of the many factors involved and collecting the data usually takes years. On the other hand, a small-scale laboratory test is cheaper and can be well controlled. Many test groups can be finished in a much shorter period of time, and when compared with a control group, the effects of suspicious factors such as moisture and temperature can be studied. The shortcoming of this type of test is the unavoidable loss of reality due to the size effect. The failure mechanism
for a large scale specimen is usually different from that of a small specimen. In an ALF test, large scale specimens are tested in the laboratory so that the testing environment is controllable. Actuators are used to simulate the traffic loads with much higher frequency than live traffic so it is less time consuming than field tests. The analysis of ALF testing is also simpler than field testing. For example, the boundary conditions of the slabs are well established so that they are known for the analysis of the ALF testing.

4.1 Accelerated loading test

4.1.1 Numerical simulations to determine the test setup

Finite element modeling was first carried out to identify the boundary conditions needed for the accelerated loaded slab so that it could represent the boundary conditions in the field. In Figure 2, a traffic lane as well as the full-depth asphalt shoulder is included in the finite element model. There are 12 overlay slabs that are 4 ft by 4 ft large on top of 6 in asphalt. The interface is modeled using the cohesive elements developed in Chapter 3.

Figure 2 Finite element model of whitetopping slabs in the field.

A tandem axle with dual tires for each wheel was loaded. For simplicity, static axle loading is used in the model. In order to simulate the damage caused by fatigue loading, the tire pressure was assumed to be 225 psi, which is twice the normal pressure that is typically seen for truck
tires, i.e. 110 psi. Under such static loading, the cohesive elements at the interface were damaged resulting in the interface bond to develop to the degree as shown in Figure 3. This debonding was caused by static loading, or in other words one pass of a wheel load. Therefore, the static load applied can be understood as the strength of the interface. When the more realistic tire pressure of 110 psi is used, the same damage can be expected due to certain repetitions of fatigue loading at a stress/strength ratio of 110/225≈50%.

![Figure 3 Interface debonding predicted using the finite element model in Figure 2.](image)

It is impossible to prepare and test the full-size specimen (see 12-slab model in Figure 2) at the PITT ALF. A specimen with reduced size had to be used. The feasible setup for the ALF was established by trying different boundaries of the finite element model shown in Figure 4 and comparing its debonding with the results of the full-size specimen.

In Figure 4, there are two overlay slabs, with displacement restraints in the Z direction applied to the free edge of the unloaded slab. The unloaded slab is tied to the restraint. As a result, the overlay slab on the unloaded side would remain in its position along the Z direction when its underlying asphalt deflects down due to the loading, resulting in peeling motion at the interface. In addition, restraints were also applied along the X and Y directions to avoid any shearing...
displacement of the slab along these directions resulting in no significant Mode II damage to the interface.

Displacement restraints were also applied to the free end of the loaded slab. However, they were applied to the end of rebars that were tied to the loaded slab. This arrangement was used to achieve a good balance between minimizing the X- and Y- displacement and maximizing the Z-deflection of the loaded slab. The use of rebar on the loaded side will restrain horizontal X- and Y-displacement similar to the unloaded side minimizing Mode II damage. However, the reduced stiffness of the rebar restraint on the loaded slab as compared to the tied concrete of the unloaded slab will result in substantially greater vertical Z-deflection of the loaded slab than the unloaded slab. This acts to create the desired differential movement between the loaded and unloaded slabs.

![Figure 4 Finite element model of the ALF test.](image)

The setup designed using the finite element model was realized in the PITT ALF as shown in Figure 5.
4.1.2 Layers

In Figure 5, the asphalt base is 3-5 inches deep depending on if they were milled or unmilled. These asphalt slabs were obtained from an in-service pavement, a segment of Highway 50 near Bridgeville, PA. The asphalt layer was an overlay on top of 12 in deep concrete slabs. The asphalt overlay had been in service for years and was decently aged and trafficked as can be seen from Figure 6. Both the asphalt overlay and the underlying concrete were to be removed due to an uneven settlement of the pavement possibly caused by mine subsidence. Due to the uneven settlement, this section of the pavement became a perfect candidate for extracting asphalt slabs for a couple of reasons. First, the underlying concrete provides support for the asphalt. It would have been extremely difficult to extract full-depth asphalt specimens of large size without damaging them. Second, a preliminary coring indicated that the two layers might have debonded or at least have had the tendency to debond due to the uneven settlement. The first step in extracting the asphalt was to cut the pavement to the bottom of the concrete layer with the desired sizes, as shown in Figure 6. Due to the clearance of the freight elevator to the laboratory, the largest allowable dimension of the asphalt slabs can only be approximately 54 in by 72 in.
The depth of the asphalt overlay is approximately 5 to 5.5 in. A portion of the pavement was milled off 1.25 inches using a milling machine that was used to prepare asphalt pavements for whitetopping, as shown in Figure 7, so that the texture of the asphalt after milling highly represents milled asphalt in whitetopping applications, as shown in Figure 8.
The asphalt overlay as well as the underlying concrete after being cut into the desired size was picked up using the contractor’s equipment and delivered to a yard, see Figure 9. In the yard, the asphalt slabs that were not debonded from the underlying concrete slabs were further processed by pushing the asphalt slabs carefully off the concrete with a fork lift. As shown in Figure 10, it was necessary to place a wood bar between the forks and the asphalt in order to distribute the shear load and prevent the fork from damaging the asphalt. A successful example of debonding the asphalt from the underlying concrete is shown in Figure 11. The asphalt slabs were then loaded to specially made pallets and transported to the pavement research laboratory at the University of Pittsburgh.
Figure 9 Lifting and transport of the asphalt slab together with the underlying concrete slab.

Figure 10 Debond the asphalt slab from the underlying concrete slab.
Concrete of 3-4 in thick was cast on the asphalt, without having a joint to differentiate the loaded and unloaded slabs, as shown in Figure 12. This is because the integrity of the concrete slab would be needed to protect the asphalt when lifting the whitetopping slab in place. Eye rings were cast in for the lifting of the slabs.

Figure 11 Asphalt slab debonded from the underlying concrete slab.

Figure 12 Cast of concrete on the asphalt slab to make whitetopping slabs.
Patching the bottom of the asphalt was required before the whitetopping slab could be tested. The roughness for the bottom of the asphalt slab can be seen from Figure 13. Uneven support would cause a stress singularity and thus introduce cracks in the asphalt interfering the interface debonding. In order to flip the asphalt slab over, the integrity of the concrete cast on top is needed to protect the asphalt. In addition, one specially made heavy-duty pallet was also placed on the concrete and clamped to the pallet under the asphalt. In such a way, the whitetopping slab is sandwiched to gain additional protection. The whitetopping slab was then flipped using a bridge crane as shown in Figure 14. The bottom of the asphalt was patched and compacted with a mixture of asphalt cold patch and asphalt epoxy. The appearance of the asphalt slab after patching is presented in Figure 15.

Figure 13 Uneven bottom of the asphalt slab.
The whitetopping slab was then flipped back and lifted onto an artificial foundation, as shown in Figure 16. An artificial foundation was created by two layers of neoprene pads, known as Fabcel 25 (http://www.fabreeka.com/Products &productId=24). The thickness of each Fabcel 25 pad is 5/16 inch. Two layers of Fabcel 25 exhibits a combined modulus of subgrade reaction equal to 200 psi/in. Under the Fabcels, a giant concrete slab with dimensions of 12-ft length, 6-ft width
and 2.75-ft thickness was cast on a heavy duty concrete floor, which can be considered as a rigid layer.

Figure 16 Placement of the whitetopping slab onto the foundation.

After placing the concrete slab onto the foundation, a joint was cut through the concrete layer in the middle of the concrete slab. Both the loaded and unloaded slabs then became 36 inches in length, 53 inches in width and 4-4.5 inches in depth.

The restraints to the unloaded slab are illustrated in Figure 17. A full depth concrete slab that is 48 in by 54 in by 9 in was cast to the side of the whitetopping slab. The concrete layer of the whitetopping slab was tied to the full depth concrete slab through two rebars as well as the bond between the concrete. The full depth concrete slab was clamped down to the steel testing frame as well as the giant concrete foundation. Neither upward nor downward displacement of the full depth concrete slab was then allowed.
4.1.3 Materials

For the aged HMA used in this study, dynamic modulus was tested and results are presented in Table 1.

Table 1. Dynamic modulus for the HMA.

<table>
<thead>
<tr>
<th>Temperature, °C</th>
<th>5</th>
<th>21</th>
<th>40</th>
</tr>
</thead>
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<td>Frequency, Hz</td>
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</tr>
<tr>
<td>10</td>
<td>1.535</td>
<td>9.5</td>
<td>0.83</td>
</tr>
<tr>
<td>1</td>
<td>1.206</td>
<td>12.4</td>
<td>0.505</td>
</tr>
<tr>
<td>0.1</td>
<td>0.905</td>
<td>16.1</td>
<td>0.289</td>
</tr>
<tr>
<td>0.01</td>
<td>N/A</td>
<td>N/A</td>
<td>N/A</td>
</tr>
</tbody>
</table>

Both fresh and harden concrete properties were tested and the results are presented in Table 2.

Table 2. Concrete material properties.

<table>
<thead>
<tr>
<th>Property</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>28-day Compressive strength, psi</td>
<td>5150</td>
</tr>
</tbody>
</table>
4.1.4 Loading profile

A single actuator was used to apply dynamic loads to the corner of the loaded slab. The loading plate is circular with a diameter of 12 inches. The load profile was designed in such a way that the actual loading period is equal to 0.035 seconds followed by a rest period of 0.165 seconds. Therefore, for each load cycle, there is in total 0.20 seconds or in other words the overall load cycle frequency was 5 Hz. During the actual loading period, the load rises from 500 lbf to 9000 lbf following a sinusoidal curve. In the rest period, 500 lbf of load was always maintained so that the actuator and slab remains in contact. The loading profile is illustrated in Figure 18, where it also shows the deflection at the corner of the unloaded slab measured by a linear variable displacement transducer (LVDT). There is a phase lag between the deflection and the load due to the viscosity of the asphalt. It is also interesting to note the damping of the deflection during the rest period although the load remains constant, which can be attributed to the visco-elastic nature of the asphalt.

![Figure 18 Loading profile used in the test and the resultant LVDT deflections.](image-url)
4.2 Tests to determine the area of debonding

Three methods were employed to quantify the area of interface debonding. A destructive method was used at the end of the test to examine the area of debonding. However, only during the fatigue test, only nondestructive evaluation (NDE) could be used to determine the development of debonding without disturbing the fatigue process. Two NDE methods are introduced in this section, with one developed based on impact echo technique and the other based on deflection principles.

4.2.1 Nondestructive evaluation- Impact echo

In traditional impact-echo testing, an impact is applied to the surface of the concrete which excites stress waves travelling into the concrete. The stress waves will reflect back if they meet an interface that could be either the bottom of the concrete or a defect in the concrete. When a piezo-electric sensor is placed nearby the impact, the depth of the defect can be predicted by characterize the time of flight or dominant frequency of the reflective stress waves. It is difficult to use the traditional impact-echo method to detect the propagation of interface debonding, because the interface reflects stress waves regardless of the existence of interfacial cracks and thus the time of flight as well as the dominant frequency would not change before and after the interface cracking. Therefore, it is necessary to develop a method that is sensitive to interface debonding so that the area of debonding can be determined to a relatively accurate level. Such a method was developed and applied to ALF testing in three steps, namely 1. Developing and validating an impact-echo based method for the detection of a 1-D crack, 2. Extending the method to determining 2-D interface debonding on small specimens and 3. Using the method to identify the area of debonding in ALF test setup.

Step one: quantification of 1-D crack propagation using an impact-echo based method

A method was developed based on impact echo principles. Its principle can be briefly summarized as follows. Stress waves are still generated by impacts and received by piezoelectric sensors. However, a baseline measurement of the specimen before any damage is registered. The difference between the baseline and a damaged state is measured periodically. Between the measurements, the impact as well as the sensors are kept the same. Since the system has nothing changed but the damage, it is possible to relate the difference in the stress waves to the degree of debonding.
Such a relationship was first established for a 1-D crack. A picture of the test setup is presented in Figure 19 and the sketch of the setup is also available in Figure 20.

Figure 19 Setup of the test for detecting 1-D interface crack using the impact-echo based method.

Figure 20 Registration of the sound wave responses of the uncracked system to impact loads.
First, a baseline measurement of the uncracked specimen was registered. Two piezoelectric transducers were instrumented on top of the specimen to receive the stress waves reflected from the concrete/asphalt interface. Impacts were generated by dropping a steel rod from a fixed height. Such drops were conducted on multiple locations that are in a line coinciding with the center axis between the two transducers, as shown in Figure 21. The waveforms of the reflected stress waves were recorded in time domain for both sensors and for all the impact locations. The assembly of all these waveforms is considered to represent the status of the uncracked specimen.

Figure 21 Sensor and impact locations for the 1-D crack specimen.

Three examples of the time-domain waveforms are presented in Figure 22, Figure 23 and Figure 24. They are induced by impacts that are -2.75 in, 0 in and 2.75 in away from the sensors, respectively (negative indicates a measurement to the left of the sensors and positive indicates a measurement to the right of the sensors in Figure 19).

One should notice that the comparison of waveforms at different locations is not capable of indicating the interface debonding. The difference from that type of comparison might be due to other reasons, such as the variation of concrete surface texture causing the small deviation of impacts, the inhomogeneity of concrete material or more importantly the change in the impact-transducer distance. When comparing Figure 22, Figure 23 and Figure 24, it is not difficult to find that the waveforms are different even when the impact locations are equally distant from the transducers and it is known that there should be no debonding anywhere at that moment.
Figure 22 Waveform for impact at -2.75 in and no crack.

Figure 23 Waveform for impact at 0 in and no crack.
The received waveforms were then transformed into frequency domain using the Fast Fourier Transform (FFT). A normalization of the frequency spectrum is performed so that the area under the spectrum is equal to 1. Although the steel rod for impacting the concrete was lifted by an electromagnet and thereby should introduce consistent impact energy, it was observed that there was still variation in the impact energy even between drops at the same impact location. It is hopeful that the variation could be eliminated by normalizing the waveform due to each impact and then averaging five impacts for the same location. Examples of the normalized and averaged frequency spectra for the waveforms in Figure 22, Figure 23 and Figure 24 can be found in Figure 25, Figure 26 and Figure 27, respectively. In Figure 25, Figure 26 and Figure 27, the area underneath the curves is 0.5 instead of 1. This is because only half of the spectrum is presented. During FFT, complex amplitude values are assigned to each frequency and thus the frequency spectrum is symmetric. Therefore, only the half of the spectrum that is physically meaningful is presented here.
Figure 25 Normalized frequency spectrum for impact at 2.75 in and no crack.

Figure 26 Normalized frequency spectrum for impact at 0 in and no crack.
After the baseline scan, cuts with different depths were made at the interface to simulate the 1-D development of the degree of debonding, as shown in Figure 28. Scans were then carried out on the specimens with the cracked interface. The impact energy, impact location as well as the sensor locations was kept the same as the baseline scan. It was then able to determine the difference of the frequency spectrum (DoFS) between a cracked interface and the baseline condition by comparing the corresponding frequency spectra.

Figure 27 Normalized frequency spectrum for impact at 2.75 in and no crack.

Figure 28 Cuts of different depth at the interface of the whitetopping specimen.
The DoFSs can be obtained for each impact location and then the total difference of frequency spectrum (TDoFS) could be calculated by summing the DoFS of all the impact locations. As a result, there is one TDoFS for each crack depth. Since the crack depths and their corresponding TDoFs are both known, they can be correlated as shown in Figure 29. It is interesting to observe a linear relationship between the TDoFS and the normalized crack depths. In total, two specimens have been subjected to the above mentioned testing and analysis. One has the cracking at the interface between concrete and asphalt and the other has the cracks at the middle of a concrete specimen. For both specimens, the TDoFS shows itself a sensitive indicator of the length change of a one-dimensional crack.

Figure 29 TDoFS vs. Normalized crack depth in 1-D experiment.

*Step two: quantification of 2-D crack propagation using impact-echo based method*

The experimental configuration for the 2-D test is shown in Figure 30. The triangular area represents the initial cut at the interface. The concrete is 4 inches thick and the datum is set at the surface of the concrete. Therefore, the interface has a Z coordinate of 4 inches. The shape of the subsequent cracks is presented in Figure 31, based on which the distance in the y direction between the crack front and the sensors can be easily calculated for each crack depth as presented in Table 3.
A good linear relationship between the Y coordinate and the TDoFS can again be seen from Figure 32.

Figure 30 Test configuration for 2-D crack test.

Figure 31 Crack depths for the 2-D test.
Table 3 Normalized distance in Y direction between the crack front and the sensors.

<table>
<thead>
<tr>
<th>Sensor</th>
<th>Cut 1</th>
<th>Cut 2</th>
<th>Cut 3</th>
</tr>
</thead>
<tbody>
<tr>
<td>Sensor 1</td>
<td>0</td>
<td>3</td>
<td>10</td>
</tr>
<tr>
<td>Sensor 2</td>
<td>10</td>
<td>15</td>
<td>22</td>
</tr>
<tr>
<td>Sensor 3</td>
<td>29</td>
<td>39</td>
<td>33</td>
</tr>
</tbody>
</table>

Figure 32 TDoFS vs. Normalized crack depth in 2-D experiment.

*Step three: quantification of interface debonding of whitetopping using impact-echo based method*

The test setup is illustrated in Figure 33. The reflected waveforms were collected by four piezoelectric sensors. The sensors were coupled onto the surface of the unloaded concrete slab with the layout depicted in Figure 34.
Figure 33 Setup of the impact-echo based test on whitetopping slabs.

Figure 34 Sensor layouts used in the impact-echo based test on whitetopping slabs.
The impacts were applied by lifting and dropping steel rods with an electromagnet. A mask with 1-in interval holes as shown in Figure 35 was used to guide the impacts so that the impact locations are consistent between scans.

![Impact locations for the impact-echo based test on whitetopping slabs.](image)

The test results are presented in Figure 36. The mean of TDoFS is calculated by dividing the TDoFS with the number of impacts. There is an increase of the TDoFS for every sensor over the two million loads applied. The growth of interface debonding can be interpolated based on this test result.

It can also be noticed that there are sudden increases of the TDoFS and they arrived at different times for the sensors. The peak for Sensor #2 arrives first, followed by Sensor #1, Sensor #3 and Sensor #4. This order indicates the direction of propagation for the interface debonding.
Figure 36 TDoFS vs. Loading cycles for the impact-echo based tests on whitetopping slabs.

4.2.2 Nondestructive evaluation-Deflection

Seven LVDTs were placed on top of the concrete at locations shown in Figure 4. The deflections of the slabs were monitored on the fly during the test. However, the LVDTs needed to be disassembled for the impact-echo based tests so that the datum for the LVDTs would vary often. In order to make use of the deflection data, a static and a dynamic scan of deflections were carried out to accompany the impact-echo scans.

During the static scan, the actuator load was increased to 4750 lbf from zero within 1 minute and then maintained for 5 minutes before decreasing back to zero. The deflections for all seven LVDTs were recorded. As an example, the profiles for LVDTs #4 and #5 are presented in
Figure 37 and Figure 38. In Figure 37, the deflections for LVDT #4 decrease as the applied loading cycles increase from 0 (the baseline line, i.e. BL) to 2 million (2000k). In Figure 38, there is an opposite trend, i.e. the deflections decreasing with increasing applied loading cycles. Both trends indicate the occurrence of interface debonding, because the deflections on the loaded side would increase and the deflections on the unloaded side would decrease as the interface debonding grows.
During the dynamic scan, a sinusoidal dynamic load was applied to the corner of the loaded slab. Sinusoidal loads were used with a minimum value of the load being 500 lbf and a varying peak
Different combinations of peak load and frequency were employed, namely 9kips-5Hz, 9kips-3Hz, 9kips-7Hz, 7kips-5Hz and 5kips-5Hz. Figure 39 shows the load and deflection profiles recorded during a dynamic scan and also defines the amplitude and offset for the deflection profile.

![Figure 39 Load and deflection profiles during a dynamic scan.](image)

The amplitudes of the deflections at LVDT #5 are plotted against the frequency of the dynamic loads and presented in Figure 40. It is obvious that the deflection amplitudes are not very sensitive to the load frequency. It can also be seen that there was a quick linear change of the amplitude over the first million loading cycles, but the growth ceased for the second million. Unlike frequency, it is expected that the deflection amplitude is dependent on the load amplitude as shown in Figure 41. Figure 42 and Figure 43 show the relationship between the deflection amplitude and the load frequency/amplitude but is shown on the unloaded side of the joint for Figure 4. It is quite apparent that the increase of the deflection amplitude became slower as more and more loading cycles were applied. Nearly half of the increase of the deflection amplitude occurred within the first half million loads. The relationship between the deflection offset and the
load frequency/amplitude for LVDT #4 is presented in Figure 44 and Figure 45, respectively. In general, the trend based on offsets agrees well with the trend based on amplitudes.

Figure 40 Amplitude of deflections for LVDT #5 vs. load frequency at various loading cycles.

Figure 41 Amplitude of deflections for LVDT #5 vs. load amplitude at various loading cycles.
Figure 42 Amplitude of deflections for LVDT #4 vs. load frequency at various loading cycles.

Figure 43 Amplitude of deflections for LVDT #4 vs. load amplitude at various loading cycles.
4.2.3 Destructive evaluation
In this section, a destructive method for examining the shape of the interface debonding is presented, based on which the results from nondestructive methods can be validated.

The whitetopping slab that had been fatigued with two million loads was first sealed along the free edges and the sides of the joint. Water dyed with red color was then poured into the joint, while the slab was dynamically loaded. It is assumed that all interface cracks were connected and a pathway from the joint to the debonding front must exist, through which the dyed water could seep and consequently color the interface. It is also believed that the dynamic loading should help with the seepage of the dyed water.

After coloring the cracked interface, the concrete overlay was cut off from the corner in a diagonal pattern (i.e. 45 degree with both the transverse joint and the edge). As shown in Figure 46, the cracked interface was successfully colored. The location and size of a new cut were then decided based on the last cut and girds were drawn on top of the concrete slab so that the size of every cut can be tracked.

![Figure 46 Cutting off the concrete overlay in a diagonal pattern, starting from the corners.](image)

The sketch of the debonded area eventually identified is presented in Figure 47. It can be seen that the interface debonding ceased to propagate towards the restrained edge after about 9 to 10
inches and then changed its direction to propagate across the slab to the unloaded corner. This agrees with the conclusions made from the impact-echo based test. The current crack path requires less energy than as if the interface debonding developed toward the restrained end. It is also interesting to note that the most debonded area coincides with the wheel path where the initial cracking of the overlay was often observed.

Figure 47 Debonded area at the end of the accelerated loading test.
4.3 Summary

Accelerated loading test was carried out at the PITT ALF to fatigue the interface bond between a bonded concrete overlay and the underlying asphalt. Interface debonding was successfully created for about 20%-25% of the interface after 2 million of loads. The development of the interface debonding was monitored using two nondestructive methods, namely an impact-echo based method and a deflection based method. The envelope of the interface debonding at the end of the accelerated loading test was examined in a destructive manner, the results of which can be used to validate the results from the nondestructive methods.

The results from the accelerated loading test, in terms of growth of the area of debonding as a function of the loading cycles will be used to calibrate the Paris’ law, which will be incorporated into the design of whitetopping to predict the degree of debonding.
5. ADJUSTMENT OF CRITICAL STRESS IN THE OVERLAY DUE TO PARTIAL DEBONDING

The existence of the interface debonding influences the critical stress in the whitetopping overlay. As the interface debonding grows with traffic, the effect of the influence grows too. The effect of the interface debonding on the critical stress in the overlay is quantified in this section via a finite element model shown in Figure 48. The degree of bond was varied by changing the size of the debonded area. However, the shape of the debonded area was always assumed as a triangle. Tie connection was used between the concrete and asphalt for the bonded area. The critical stress in the overlay was always extracted from the wheel path at the bottom of the overlay slab for all the debonding scenarios.

![Finite element model to study the effect of partial bonding on the critical stress in the overlay.](image)

The relationship between the degree of debonding and the critical stress in the overlay was established and presented in Figure 49. The critical stress in the overlay increases by 50% and 100% after the interface debonds develops from zero to about 20% and 50%, respectively. The critical stress would not increase anymore once more than half of the slab is debonded.
Figure 49 Relationship between the degree of debonding and the critical stress in the overlay.
6. CONCLUSION

This study describes the framework for considering interface debonding into the design of whitetopping.

The development of interface debonding due to fatigue was studied for whitetopping slabs in an accelerated loading facility. Aged asphalt slabs were extruded from in-service asphalt pavements with both milled and unmilled surfaces. Whitetopper slabs were made by casting concrete on the asphalt slabs and then fatigued using actuators to simulate live traffic but at an accelerated pace, resulting in the successful creation of interface debonding. The growth of the interface debonding area was monitored using nondestructive evaluation methods that were developed in this study. The results from the nondestructive evaluation were validated by destructive examination of the debonding at the end of the fatigue test.

In order to use Paris’ law to describe the interface cracking due to fatigue, material properties of the interface were also established by conducting the wedge splitting test. The critical energy release rate for the interface fracture of whitetopping specimens was calculated based on the wedge splitting test results, at various temperature, moisture and asphalt surface conditions.

Cohesive elements in the finite element environment were also established based on the wedge splitting test to predict the transient energy release rate at the accelerated loading tests. This is another prerequisite for using the Paris’s law.

So far, only one whitetopping slab that has milled asphalt has been tested and the analysis of the data remains in progress. Consequently, Paris’ law for considering the interface debonding in the design of whitetopping will become available following the completion of more whitetopping slab testing.