

A Case Study of Slab Cracking and Delamination in BCO Construction at the Bush International Airport (IAH) Part I: Impact on Service Life

Steve Byington, P.E.¹ Rashid Abu Al-Rub, Ph.D.²,
Stefan Hurlebaus, Ph.D., Ing.³ and Dan G. Zollinger, Ph.D., P.E.⁴

Abstract

Bonded Concrete Overlay (BCO) construction at the Bush International Airport (IAH) has been subjected to incidences of cracking and delamination that is speculated to be mainly climatically driven due the extremely dry and windy weather conditions prevalent throughout most of the construction period. An investigation was undertaken to determine the strength of the bond between the overlay and existing pavement as well as the structural integrity of the constructed pavement section to determine effects on project pavement service life. This paper reports the details of the investigation and the findings from the analysis of them relative to the impact on the service life.

Introduction

In February of 2011, construction of a bonded concrete overlay administered by the Houston Airport System (HAS) on Taxiway (T/W) “WA” at the Bush International Airport commenced. The center 7.6 m (25 ft) of the taxiway (i.e. keel section) consisted of 432 mm (17 in) of full-depth paving while the outer 7.6 m (25 ft) lanes consisted of an 203 mm (8 in) BCO over the existing concrete pavement. Essentially, the paving was placed between February 2011 and May 2011 and not long afterwards, airport engineering personnel had taken note of extensive cracking that had taken place in the overlay concrete. All the observed distress occurred in the overlay that was placed on either side of the taxiway keel section that consisted of approximately 3050 m (10,000 LF) of paving. The BCO was 203 mm (8 in) thick placed over an existing 762 mm (30 in) section of concrete pavement matching the existing joints with full-depth sawcuts. The existing pavement was essentially a 7.6 m (25 ft)x7.6 m (25 ft) wide slab (which consisted for the most part of an 457 mm (18 in) PCC layer partially bonded to an underlying 305 mm (12 in) PCC layer below) where the joints in the existing pavement were doweled. Table 1 itemizes the mixture design use for the BCO; w/cm was 0.31 which essentially eliminates any water of convenience and may have induced some autogenous shrinkage.

¹Steve Byington, P.E., Project Manager, Klotz Associates, Houston, TX 77079
E-mail : Steve.byington@klotz.com

²Rashid Abu Al-Rub, Ph.D, Assistant Professor, Texas A&M University,
College Station, TX 77843-3136, E-mail : rabualrub@civil.tamu.edu

³Hurlebaus, Stefan, Dr. Ing., Associate Professor, Texas A&M University,
College Station, TX 77843-3136 , E-mail : shurlebaus@civil.tamu.edu

⁴Dan G. Zollinger, Ph.D., P.E., Professor, (Corresponding author) Member, Texas A&M University,
College Station, TX 77843-3136, E-mail: d-zollinger@tamu.edu

Table 1 BCO Mixture Design.

1½” Limestone C. Aggregate	889.5 kg (1961 lbs)
Sand	626.4 kg (1381 lbs)
Ash Grove Cement	192 kg (423 lbs)
Class F Fly Ash	64 kg (141 lbs)
Water	79.4 kg (175 lbs)
SikaPlas500	13.3 liter (451 fl oz)

Many factors can synergistically combine to affect early age stress development leading to cracking and delamination in BCOs. Jang (2005) put many of these factors into perspective by incorporating the key aspects of early age jointed concrete stress and strength development over time in his analysis. A basic look at slab behavior and the mechanics associated with that, although not elaborated here, is especially helpful in developing a perspective as the effect that concrete mixture and climatic variables have on the potential for these problems to initiate. The types of cracking observed in the T/W WA paving fall into both the transverse and longitudinally oriented categories mainly initiated at the top or the surface of the slab; due to the age of the concrete at crack initiation and the nature of the mixture design, none of which are suspected of being plastic shrinkage cracking. However, in some instances there was some cracking randomly oriented that likely initiated at the surface of the pavement. Along with the slab cracking in some areas of the overlay concrete, there were other areas where delamination between the overlay concrete and the existing pavement had occurred. The delamination was largely identified by sounding of the overlay concrete using a 4.5 N (10 lbs) sledge hammer since delamination causes the overlay concrete to separate from the existing pavement. Delamination, if it was present (as noted in Figure 1), typically occurred along the edges and the corners of an overlaid slab (Lange and Shin, 2001).

**Figure 1** Hammer for Sounding Delaminated Areas (see red dots).

The stresses associated with the observed cracking are thought to be mainly caused by curling/warping behavior of the slab which is of course a non-load related phenomena. A portion of the volumetric moisture and temperature induced changes that take place within the concrete contribute to curling/warping action of the slab (Figure 2a); while other components of it may also contribute to horizontal contraction between the slab and the supporting medium. During hardening, the shrinkage of the surface concrete (principally due to evaporation) is often greater than the shrinkage near the bottom of the overlay. This action will cause the top of the slab to contract with respect to the bottom thus warping the slab in a concave configuration tending to lift the slab off the lower substrate in the vicinity of the slab corners and along the slab edges. It is possible for the BCO to be restrained in multiple ways, but it seems feasible that tensile stresses would be induced at the top of the slab in some instances while in others compressive stresses induced at the bottom of the slab primarily due to the dead weight of the concrete which tends to counteract the shrinkage induced volume changes. Under the right temperature regime (mainly in the daytime), high temperatures at the surface can cause the opposite behavior to occur forcing a stress gradient to develop with compressive stresses at the top of the slab and tensile stresses at the bottom again causing lift-off away from the corners and edges of the slab. Curling/warping stresses in PCC slabs could at times exceed a stress level of approximately 1.37 MPa (200 psi) under certain conditions potentially exceeding the strength of a hardening concrete layer. Furthermore, it is likely that the amount of curling and warping could be affected by the materials used, construction methodology, and weather conditions at the time of construction. Many of these factors are relevant to the nature of the concrete mixture and how the development of strength and early aged drying shrinkage occurs over time; especially the:

1. Amount of water used to finish the concrete surface,
2. Method and quality of curing.

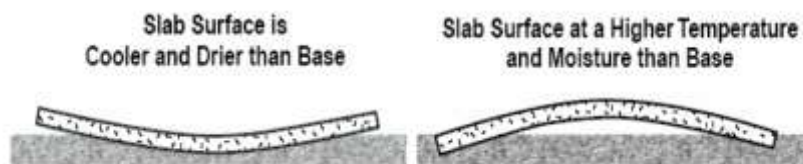


Figure 2a. Slab Curling and Warping Action.

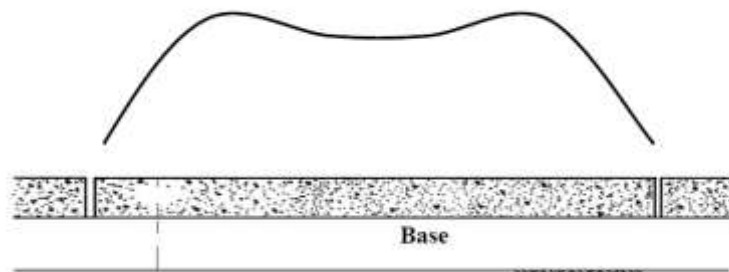


Figure 2b. Slab Curling and Warping Pattern of Stress.

Clearly, the impact of this type of stress development on early cracking of PCC slabs must be considered as a principal cause of cracking. This paper describes data collected and findings developed as part of an investigation into the effects of the

cracking and delamination associated with the T/W WA BCO paving on the longevity of the pavement structure. In this regard, the following topics are addressed:

1. Cracking and delamination distress surveys
2. Structural assessment
3. Analysis and discussion of collected data

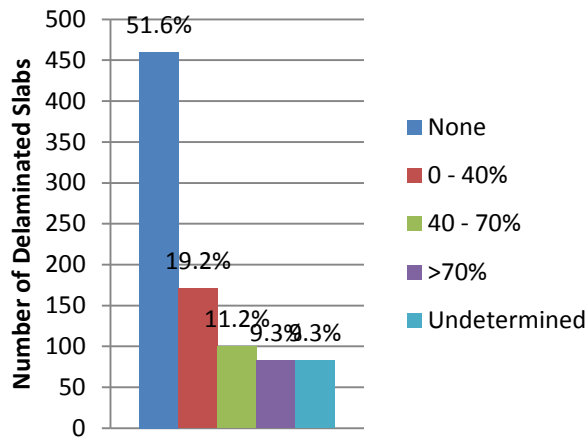


Figure 3. Summary of Delaminated Slabs.

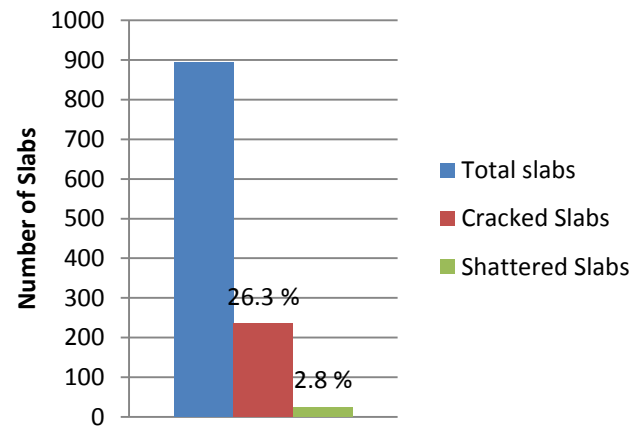


Figure 4. Summary of Cracked Slabs.

Cracking and Delamination Distress Surveys

The summary of distress based on the surveys taken of the overlaid portions of T/W WA is shown in Figures 3 and 4. The data in Figure 3 is presented with respect to the number of slabs that fall within the noted percentages of the slab area that has been subject to delamination. Most of the cracking was manifest as a single transverse crack in the mid section of the panel but many slabs were both cracked and delaminated. Multiple cracked slabs in some cases manifest as two orthogonal cracks again through the mid section of the panel since the slabs are nearly equal-dimensional. Cracking of this nature is consistent with the assumption that the principal cause of the cracking is due to curling and warping behavior. Furthermore, cracking driven by curling and warping behavior is often transversely oriented and due to the effect of drying shrinkage during the early hydration and curing stages; furthermore, this type of cracking is often initiated at the surface of the pavement and extends through the entire overlay thickness.

The incidence of shattered slabs occurred over the range of delaminated panels as shown in Figure 5. It is noteworthy that research (Lange and Shin, 2001) on the mechanism associated with delamination at the corners has suggested that the delamination stress diminishes after cracking has occurred which seems to be supported by the data shown in Figure 5. As previously noted, some of the cracked slabs may have also suffered some cracking due to plastic shrinkage cracking and climatic conditions as could be depicted by the data shown in Figure 6 prior to delamination (but for the most this is thought not to be the primary source of cracking); furthermore, the causes tied to slab warping can be from a number of sources but the most prevalent is internal to the mass of the concrete layer due to a gradient of shrinkage induced strain with depth. Although some freedom of movement exists at the surface – greater amounts of restraint are exhibited with depth into the overlay due to the restraint offered by bonded surface; in this regard, the degree of bond contributes to both stress development and strength of the overall pavement section. It is possible that in at least one circumstance that higher

evaporation rates may result in crack formation while the concrete is in a relatively weakened state. The potential of cracking of this nature is perhaps one reason why hot weather concrete measures are used in practice.

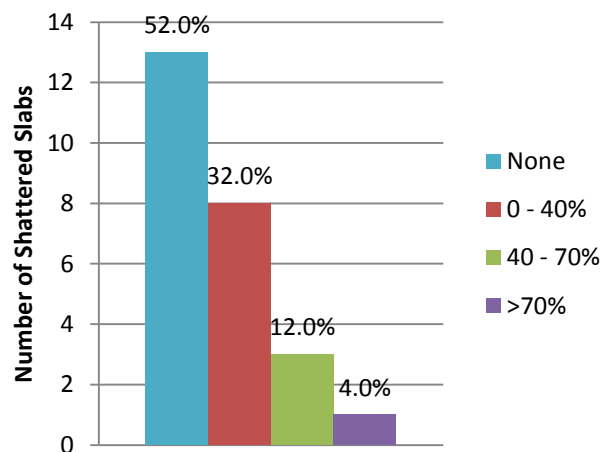


Figure 5 Distribution of Shattered Slabs with respect to the Extent of Slab Delamination.

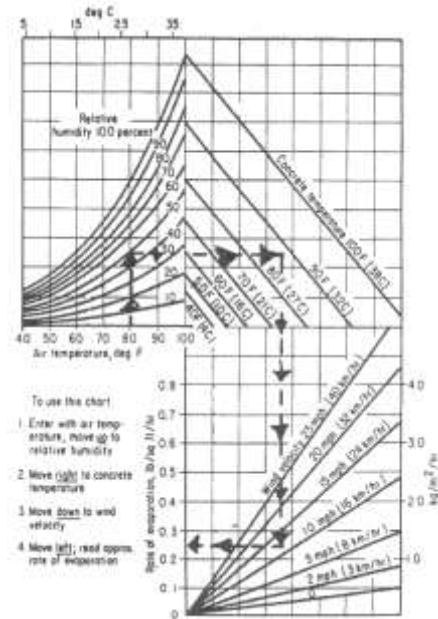


Figure 6 ACI Evaporation Nomograph (ACI, 1977).

Although the Figure 6 data is often used to ascertain the potential for plastic shrinkage cracking it is also has some utility with respect to the assessment drying shrinkage cracking during early concrete hardening. The observed slab cracking was also summarized with respect to the potential for evaporation which represents the effect of weather with respect to the ambient temperature, wind, and relative humidity in a single parameter. The potential evaporation (PE) is derived from the well known ACI 305 nomograph shown in Figure 6. In order to apply the above expression for PE to field conditions, the following expression is used:

$$PE = 4.88 * [(1.8 * T_c + 32)^{2.5} - (RH_a * (1.8 * T_a + 32)^{2.5})] [1 + 0.4 V / 1.609] * 10^{-6}$$

Where:

- PE = estimated evaporation rate, $kg/m^2/hr$
 T_c = concrete temperature, $^{\circ}C$
 T_a = air temperature, $^{\circ}C$
 R = (ambient relative humidity Percent)/100, and
 V = wind velocity, km/h .

Figure 7 depicts the trend in total cracking as a function of the difference between the average and maximum PE (due to the preponderance of gusting wind conditions which were not reflected well in the average PE) that occurred between the hours of 8am and 6pm on each day paving operations were carried out. The cracking levels that are presented in the figure are in terms of percentage of total slabs paved for a given day of paving; cracking generally increased as the PE different increased. Consideration of the windy conditions at the project site during the period of construction yielded the highest degree of correlation to the incidence of cracking and tends to suggest the effect that gusty conditions may have on cracking distress. It is

important to note that the construction records indicate that much of the cracking distress had occurred prior to the latter half of April 2011 when the daily air temperatures began to substantially exceed 29.4°C (85°F).

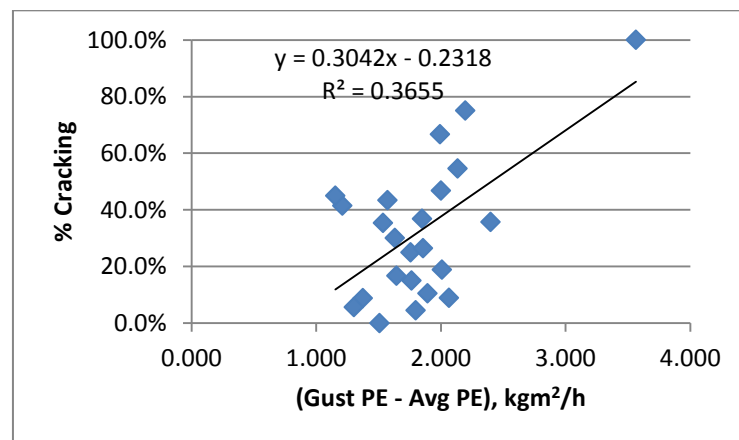


Figure 7. Incidence of slab cracking versus PE difference.

As previously noted, sounding of the slab corners and edges using a 10 lb hammer was carried out to identify the slabs that had delaminated. In order to validate the sounding, ultrasonic tomography (UST) was used. UST is a method of non destructive testing (NDT) that is proving to be effective in detecting shallow and deep delaminations (White et al. 2011). In this technology, an array of piezoelectric transducers is used to generate and detect shear or SH-waves in a concrete layer. The result is an image of the internal structure of the slab by using pulse-echo principles. The UST device consists of a 4x12 array of dry-point contact (DPC) transducers. Figure 8a shows the row of sensors that are involved in the UST technology where a single row of transducers serve as a source where other rows of transducers serve as receivers. Each transducer is independently spring-loaded which allows testing on uneven surfaces. The system compensates for surface SH-waves providing a clearer image in the near surface area.



Figure 8a Ultrasonic Tomography (www.pcte.com.au) View of Sensors.



Figure 8b Using Ultrasonic Tomography in the Field.

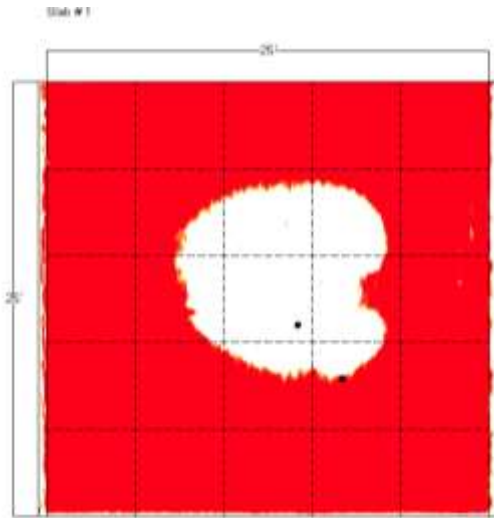


Figure 8c Scan for Slab Overlay
Located at Sta 83+00 T/W WA
(east side) >70% Delaminated
(red color denotes delaminated area).



Figure 9 Cap for Tensile Strength Testing.

UST was used to scan a slab located at T/W WA Sta 83+00 where results of the scan are shown in Figure 8c. Black dots shown in the figure indicate where cores were obtained to check the degree of bonding via shear or tensile testing as a means to verify the UST results. The testing locations in the BCO were determined according to the UST results in order to obtain bond strengths of varying degrees along the interface between the overlay and the existing concrete slab. Tensile strength was measured using a pull-off device (where the cap was glued to the concrete surface at the center of a core location – Figure 9). This technique was used in the field to determine the tensile strength where the cap was able to be sufficiently bonded to the new concrete; otherwise it was necessary to conduct shear testing by placing an anchor bolt in the cored portion of the overlay and using a calibrated torque wrench. Tests of this nature were conducted at both fully and partially bonded locations. Again, the degree of the bond was pre-assessed based on the results of the UST testing or the results of field coring. The results of this testing is shown in Table 2. Fully bonded tensile and shear strengths along the interface were nearly the same and can perhaps be considered to be complimentary of each other and consequently fall into the same range of expected strength values.

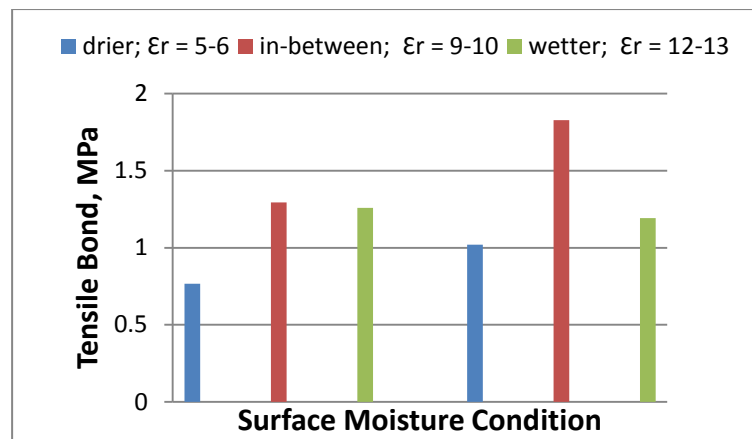


Figure 10 Bond Strength of Laboratory Prepared Samples.

Additional cores were obtained from the existing concrete to conduct further bond testing in the laboratory to examine the effects of surface wetting on the bond strength of the interface (Figure 10). The surfaces of the existing cores had been skid-abraded in the same manner the existing pavement surface had been prior to placing the overlay concrete. The existing surface was also wetted prior to placement of the BCO depending upon the evaporative conditions as the time of paving. Consequently, the amount of wetting varied from day to day during construction depending on the weather. The skid-abraded surfaces in the laboratory were treated at three different moisture contents to represent different levels of moisture that may occur under field conditions that were delineated by the measured dielectric constant (ϵ_r) of the surface concrete as noted in Figure 10 (the ϵ_r is particularly sensitive to the moisture content of the concrete). The laboratory test results were comparable to the field strengths listed in Table 2 and suggest that the in-between moisture conditions are optimal moisture levels for achieving a bond to the existing concrete surface. It appears that some wetness is desirable (as would be expected) and that a dry surface would not be the best condition prior to placement of the concrete mainly due to the suction of moisture needed for hydration into the existing concrete surface.

Table 2 Bond Strength Testing – Field.

Type	Degree of Bond	Strength	Location
Pulloff	Full	1.26 MPa	83+00
	Partial	0.13 Mpa	82+75
Torsion	Full	1.51 Mpa	82+25
	Full	1.23 Mpa	82+25

Structural Assessment

As part of the investigation of the effect of delamination of the 8 inch BCO on the structural capacity of the pavement section, NDT data was also collected using the heavy falling weight deflectometer (HWD). The HWD, (shown in Figure 11) measures deflections due to a dynamically applied load via a load plate at 7 specific locations referenced to the center of the load plate. The deflection data is recorded for three load drops; 11,400 kg (25,000 lbs), 20,865 kg (46,000 lbs), and 25,401 kg (56,000 lbs). The NDT testing was conducted at selected locations along the T/W WA alignment. The analysis from this collected data (Zollinger et.al., 2005) consisted of the determination of:

- Load transfer efficiency (%)
- Effective slab thickness (in)
- The coefficient of friction between the slab and the stabilized subbase
-



Figure 11 HWD and Load Plate.

Load transfer efficiency (LTE), although not an issue in this investigation, does have a role in the overall thickness design of the pavement from a structural perspective. The structural section of T/W WA considered to be of most interest in this assessment consisted of the 203 mm (8 in) BCO on the existing 457 mm (18 in) PCC layer which, due to the original construction, was unbonded from an underlying 305mm (12 in) PCC layer; wherein all of the above layers were supported by a 152 mm (6 in) flexible subbase. The effective slab thickness determinations made for this study (which was limited to only represent the structural thickness of the 203 mm (8 in) and 457 mm (18 in) layers combined), in addition to the LTE, varies depending on the following:

- The moduli of the slab and subbase layers
- The thickness of the slab and subbase layers
- The coefficient of friction between the slab and subbase layers (not reported)
- The degree of erosion between the 457 mm (18 in) and the 305mm (12 in) layers
- Strength of the subgrade

The average values of effective thickness (over 6 consecutive slabs), along with LTE, are summarized in Table 3 for selected portions of T/W WA (indicated in the first column) that represent different degrees of support and levels of delamination). Three locations (joint-center-joint) along the slab centerline and along the outer edge of the slab were tested but the maximum theoretical thickness assuming perfect bond between the BCO and the 457mm (18 in) substrate layer would be 660 mm (26 in). The 3 pavement sections were tested that represented combinations of subgrade strength and overlay delamination as:

- 1) Well supported (and 0% delamination)
- 2) Poorly supported (and ~ 40)% delamination), and
- 3) Mix supported (and 40-70% delamination)

Table 3 Summary of HWD Data Analysis.

Sta Loc/ % Delam			Avg h_e		Avg LTE	Avg h_e
	Time	Position	Slab Ctr	Joint	Joint	Diff
107+50	Morning	Ctr Line	574.0 mm (22.6 in)	99.1 mm (3.9 in)	0.30	353.0 mm (13.9 in)
~40%	Afternoon		576.6 mm (22.7 in)	312.4 mm (12.3 in)	0.65	271.8 mm (10.7 in)
	Morning	Edge	218.4 mm (8.6 in)	111.8 mm (4.4 in)	0.42	
	Afternoon		304.8 mm (12.0 in)	226.1 mm (8.9 in)	0.68	
90+25	Morning	Ctr Line	657.9 mm (25.9 in)	535.9 mm (21.1 in)	0.82	147.3 mm (5.8 in)
0%	Afternoon		645.2 mm (25.4 in)	574.0 mm (22.6 in)	0.82	104.1 mm (4.1 in)
	Morning	Edge	513.1 mm (20.2 in)	350.5 mm (13.8 in)	0.80	
	Afternoon		541.0 mm (21.3 in)	462.3 mm (18.2 in)	0.86	
83+25	Morning	Ctr Line	612.1 mm (24.1 in)	292.1 mm (11.5 in)	0.57	238.8 mm (9.4 in)
40-70%	Afternoon		637.5 mm (25.1 in)	492.8 mm (19.1 in)	0.77	241.3 mm (9.5 in)
	Morning	Edge	373.4 mm (14.7 in)	157.5 mm (6.2 in)	0.40	
	Afternoon		396.2 mm (15.6 in)	292.1 mm (11.5 in)	0.73	

areas of the taxiway. These combinations resulted in a variation in effective thickness from test location to test location; some greater than 660 mm (26 in) but most of them less than 660 mm (26 in). For the most part, the effective thickness at the mid-panel locations was considered to be very good but there were areas of very low support and stiffness due to either the location of the load on the slab and/or due to the presence of delamination. The last column in Table 3 shows the difference of effective stiffness between the pavement edge and the center of the slab where delamination was minimal. This difference tends to reflect the loss of structural integrity due to the effect of delamination. Nonetheless, this variation is reminiscent of the subgrade strength variation noted during the design phase in analysis of FWD data collected for design analysis of the existing pavement section. Due to the poor support conditions that existed at station 107+50, the assessment of damage due to slab delamination would perhaps be more appropriately based on the other two locations listed in the table. On that basis, it appears the loss of stiffness along the slab edges could be equivalent to a loss as much as 178 to 203 mm (7 to 8 in) of pavement thickness.

Analysis Using FEM and Effect on Service Life

Finite element method (FEM) analysis of different delaminated slab configurations, as were observed, was carried out to assess the effect of different delaminated slab configurations on the potential for cracking and impacting long term performance of T/W WA. Figure 12 shows the pavement structure, preexisting delamination, and

simulated loading and boundary conditions. The loaded area and magnitude from the main gears of a Boeing 777-200LR aircraft are obtained from www.boeing.com. For simplicity and saving in computational cost, two-dimensional plane strain FEM simulations are conducted using the well-known FEM software Abaqus (2008). The FEM analysis was carried out to examine the state of stress for different levels of delamination and for different amounts of friction between the delaminated layers.

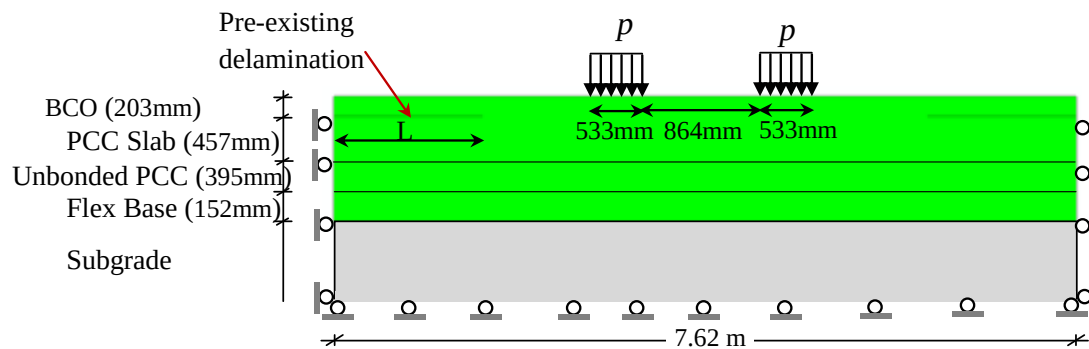


Figure 12 Finite element geometry and boundary conditions of the simulated pavement structure with a partially debonded PCC overlay.

However, here the results from only two levels of delamination (40% and 70%) with an effective friction coefficient of 30 (representative of the interfacial resistance offered by the delaminated slab based on FWD testing) along the debonded area are presented (see Figure 13). The results are compared to the case of no delamination (i.e. 0% delamination and perfect bond). The bending stress distribution, which is equivalent to the maximum principle stress and is the most critical stress for causing the potential for cracking, is shown for the different delamination levels and for two tire loading conditions (edge and center). Clearly, one can notice from Figure 13 that the bending stress is increased as the level of delamination is increased. This increase is more noticeable for the case of edge loading (20% increase in bending stress for 40% delamination and 39% increase in bending stress for 70% delamination). However, this analysis shows that sufficient structural capacity remained (even for relatively highly delaminated slabs) within the overlaid structure such that critical load stresses were relatively insignificant as compared to the flexural strength of PCC. Including the effect of thermal stress in the overall stress analysis, due to further curling of the delaminated overlay results in additional stress as a worst case condition in the range of 100 to 120 psi contributing to the accumulated fatigue damage of the overlay. These stresses result from assuming a 15°F temperature difference between the top and the bottom of the overlay but the increase in the stress ratio is still below 0.5 which may result in minimal effects.

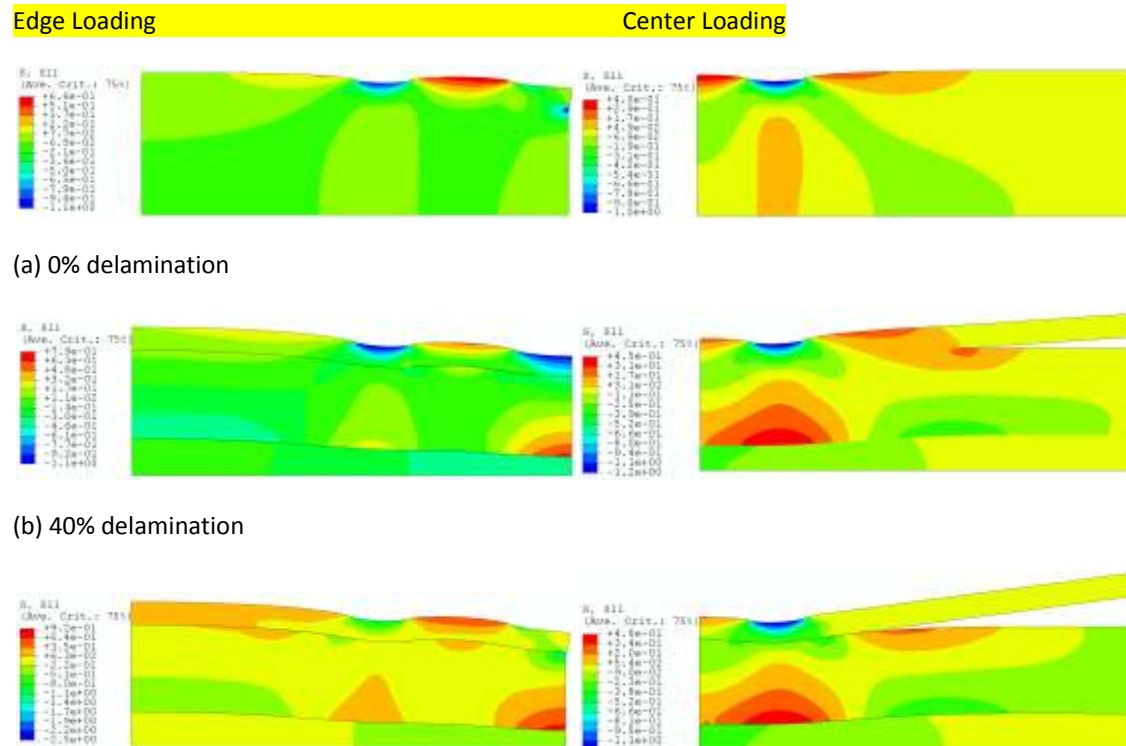


Figure 13.

Of great concern and perhaps more critical than the effect of climatic stresses are induced shear stresses under load at the tip or along the edge of the delaminated slab which were found to be relatively insignificant – at least under normal operational conditions. When the 777 gear loading is positioned near the center of the overlaid slab, upward deflections could occur along the delaminated edges as shown in Figure 12. This upward deflection may over time (and under saturated conditions) facilitate the infiltration of debris in the delaminated portions of the overlay possibly creating a critical stress condition under load where the debris could impinge upon the concrete in the overlay creating an exceptionally high tensile stress conditions that likely to cause cracking of the concrete. For example, Figure 14 shows the evolution of a crack due to presence of debris. This result is obtained using the coupled plastic-damage model in Abu Al-Rub and Kim (2010). Another possible form of failure would be spalling of the joint due to a high bending moment being carried due to the lack of bond at the interface. For this reason, either grouting the interface or pinning the corners and edges on selected slabs is a recommended alternative to minimize the potential of this problem occurring. The main idea with pinning is to reduce the spall related stress that would allow the corners to break free of the BCO layer.

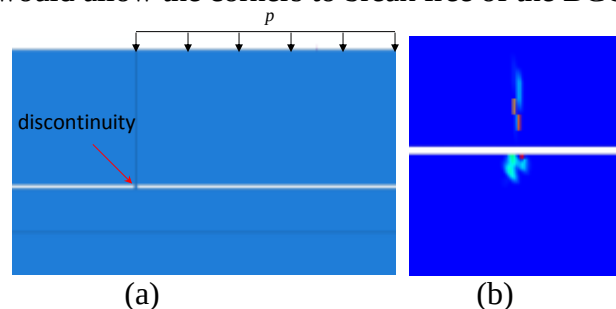


Figure 14 (a) A discontinuity under the PCC overlay, (b) crack evolution.

Summary

Based on the data shown in Figures 3 through 7, it appears that under certain combinations of wind and relative humidity that bonded overlay construction may be subject to a relatively high risk of delamination. The determination of the extent of delamination based on sounding the concrete with a 10 lb hammer appeared to be sufficiently accurate to identify areas of the BCO that had debonded. It is difficult to speculate the depth of detection that sounding in this manner will be effective but it could be well over 12 inches particularly if earphones are incorporated to magnify the sound wave. FWD testing and analysis appeared to be useful to reveal the extent of loss in structural integrity incurred due to the delamination of the 203 mm (8 in) BCO. Due to the frictional restraint at the delaminated interface, load induced stresses for the design aircraft were not found to be high enough to create a concern relative to service life of the pavement however, under environmental effects, the combination of climatic and load stresses could be a factor impacting the fatigue life of the overlay. This in combination with the potential for displacement under load at the slab edges may create a circumstance as to where debris and other foreign objects may cause stress intensity levels to initiate cracking. For these reasons, measures to repair the delaminated areas are recommended to preserve the integrity of the bonded overlay.

References

- Abaqus (2008). Version 6.8, Habbitt, Karlsson and Sorensen, Inc, Providence, RI.
- Abu Al-Rub, R.K., Kim, S.-M. "Computational Applications of a Coupled Plasticity-Damage Constitutive Model for Simulating Plain Concrete Fracture," *Engineering Fracture Mechanics*, Vol. 77, No. 10, pp. 1577-1603, 2010.
- Journal of the American Concrete Institute, *Proceedings*, Vol 74, No. 8, p. 321, 1977.
- Jang, Se Hoon, "Automated Crack Control Analysis for Concrete Pavement," Thesis, Texas A&M University, College Station, Tx, 2005.
- Lange, David A. and Hak-Chul Shin, "Early Age Stresses and Debonding in Bonded Concrete Overlays," *Transportation Research Record No. 1778*, Transportation Research Board, Washington, D.C., 2001, pp. 174-181.
- www.boeing.com/commercial/airports/acaps
- Zollinger, Corey, Dan G. Zollinger, Dallas Little, and Adel Godiwalla, Innovative Approach to Pavement Rehabilitation Analysis and Design of Runway 15L-33R at George Bush Intercontinental Airport in Houston, Tx," *Proceedings, Eighth International Conference on Concrete Pavements*, August 14-18th, 2005, Vol. 3, pp. 1101-1119, Colorado Springs, Colorado.