Bridge Approach Pavements, Integral Bridges, and Cycle-Control Joints

MARTIN P. BURKE, JR.

The 1977 Final Report of the National Experimental and Evaluation Program for Watertight Bridge Deck Joint Seals documents the experiences of 40 state transportation departments for a period of 7 years. The summary contains a paragraph indicating that a number of states have shown strong interest in bridges of some length built without joints, and designs with integral abutments. Many such bridges have been built in Tennessee, according to this report, with the only evidence of movement showing in the form of a crack in the approach pavement off the structure. Those cracks are of a minor nature and do not appear to present a problem. However, as is evidenced by the bridge approach designs presently being used by a number of states, and illustrated and evaluated in this paper, some transportation departments are fully aware that the designs of approach pavements for integral bridges need special consideration and provisions if they are to survive for more than 5 to 10 years without serious distress. They are aware that the cracks described do not begin to suggest the potentially serious distress that such approaches will experience if appropriate pavement designs are not developed and employed. Additionally, the bridge approach designs presently being used suggest that some engineers are not fully aware of the great growth potential of unrestrained rigid pavement and the great pressure potential of restrained pavement, or are unable to provide an effective means to contend with such behavior. Even the most effective of the present designs appear to need improvement if they are to survive for more than 5 to 10 years without modification or repair. Finally, the functional effectiveness of these designs is becoming even more critical with the current emphasis on integral design and the development of even longer integral bridges. Although some of the current designs will be illustrated and evaluated in this paper, completely effective designs are not suggested. Such designs will depend on materials that are presently not available to the transportation profession. However, it is hoped that this paper will help spread an awareness of the problems in this transition area so that there will be greater coordination between the bridge and pavement engineers and greater demands upon the elastomer industry to develop the kinds of materials needed to construct durable and effective cycle-control joints for bridge approaches. The designs for integral bridges and their approach pavements should then more effectively accommodate the approach-pavement characteristics and the characteristics of the integral bridges being built and contemplated.

The National Experimental and Evaluation Program for Watertight Bridge Deck Joint Seals (NEEP Project 11) documents the experiences of 40 state transportation departments for a period of 7 years. In the 1977 final report of that project *(1)*, the summary contained the following paragraph:

> Tennessee, Idaho, South Dakota, Ohio, and other States have taken a strong interest in bridges of considerable length without joints and designs with integral abutments. Tennessee has built many such bridges that are clean, neat looking structures with the only evidence of movement showing in the form of a crack in the approach pavement off the structure. Those cracks are of a minor nature and do not appear to present a problem.

However, as is evidenced by the bridge approach designs presently being used by a number of states, designs that are illustrated and evaluated in this paper, some transportation departments are fully aware that the designs of approach pavements for integral bridges need special consideration and special provisions if such designs are to survive for more than 5 to 10 years without serious distress. They are aware that "Those cracks...of a minor nature" do not begin to suggest the potentially serious distress that such approaches will experience if appropriate approach pavement designs are not developed and employed.

Additionally, the bridge approach designs presently being used suggest that some engineers are not fully aware of the great growth potential of unrestrained rigid pavement and the great pressure potential of restrained pavement, or are unable to provide an effective means to contend with such behavior. Even the most effective of the present designs appear to need improvement if they are to survive for more than 5 to 10 years without modification or repair. Finally, the functional effectiveness of these designs is becoming even more critical with the current emphasis on integral designs and the development of even longer integral bridges.

Before discussing and evaluating some of the approach pavement designs being used adjacent to integral bridges, it is first necessary to describe some of the background that has motivated the development of integral bridges. Pavement pressure, pavement growth, and the bridge damage associated with these phenomena, along with deicing chemical corrosion and ineffective bridge deck joint seals, are the primary motivating influences shaping or directing this development. However, pavement behavior appears to be the most significant influence.

PAVEMENT PRESSURE

Pavement blowups are a clear indication of the high pressures that can be generated in restrained pavement. However, some believe and others suspect that pavement blowups are indica-

Burgess & Niple, Limited, 5085 Reed Road, Columbus, Ohio 43220.

tions of localized high pressures and not indications of generalized longitudinally oriented compressive stresses existing throughout extensive lengths of pavement and distributed both laterally and vertically throughout the pavement cross section. However, a brief and simplified explanation should help to illustrate that extremely high compressive stresses, distributed throughout the pavement cross section, are probably the rule rather than the exception.

Figure 1 illustrates the cyclic movement that occurs at pavement contraction joints. This movement is caused by the response of the pavement to changes in the pavement's moisture content and temperature. Also illustrated is the effect that incompressible debris has on this cyclic movement.

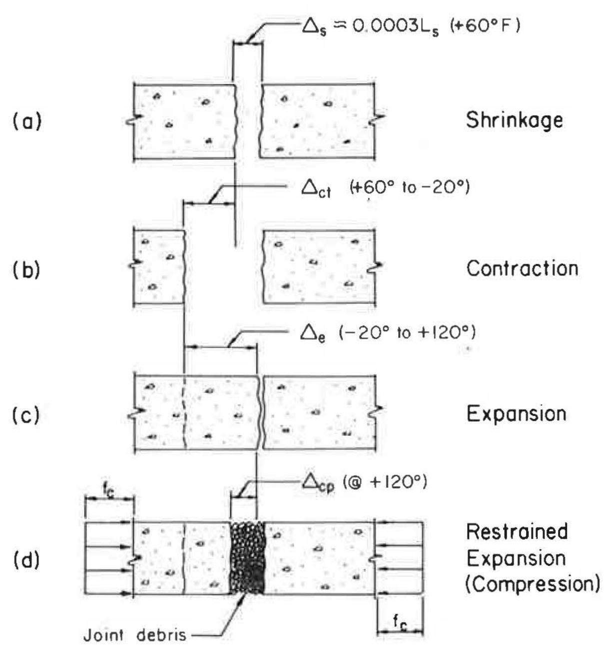


FIGURE 1 Cyclic movement at contraction joints.

In drying, following a wet curing period, concrete shrinks a maximum of about 0.0005 of its length. This is the well-established average value of the total free shrinkage, namely, from a saturated to a dry condition. Most of the shrinkage can be recovered by a thorough rewetting. Since concrete pavement in contact with a subgrade probably retains a substantial amount of moisture, a coefficient of 0.0003 may be used to represent the initial shrinkage of the average pavement. After being cast, concrete pavement responding to a loss of moisture, to cooling after the heat of hydration, and to a lowering of the ambient temperature tends to shorten. This shortening is resisted by the tensile strength of the concrete. Ultimately, the tensile strength is exceeded and the pavement cracks at the precut contraction joints. Ignoring the effects of concrete hydration, pavement reinforcement, subgrade friction, and so on, the initial cracking due to shrinkage, as illustrated in Figure 1 (a), may be assumed to be equal to about $0.0003L_s$ where L_s equals the length of a pavement section between contraction joints.

Responding to changes in ambient temperature, the initial shrinkage crack opens wider at temperatures lower than normal, Figure 1(b), and closes at temperatures higher than normal, Figure 1(c). With daily fluctuations in temperature, and with magnification of these fluctuations due to seasonal temperature extremes, this movement at the contraction joint continues to cycle, as illustrated in Figures 1(b) and 1(c). However, as these joints are only surface sealed, initial infiltration of debris begins at the open ends and open bottoms of the joints. This infiltration is facilitated by the movement of water, which penetrates the pavement and shoulder joints from above, and the groundwater, which seeps through the shoulders and migrates along the subbase from below. As the joint seals begin to fail because of a combination of age degradation, low temperature stiffening, traffic abrasion, neglect, and so forth, debris infiltration accelerates both from above and below.

Owing to the presence of debris, the cyclic movement at the contraction joint is restrained by compression of the debris and by the restrained expansion (compression) of the pavement Δ_{cp} [compare Figure 1(d) to 1(c)].

As stress is proportional to strain ($f_c = E_c\varepsilon$), the stress induced by this restrained expansion (compression) can be estimated by assuming a value for the strain associated with the condition illustrated in Figure 1(d). By assuming Δ_{cp} of Figure 1(d) to be about equal to the original pavement shrinkage, Δ_s, of Figure 1(a) equal to $0.0003L_s$, the unit strain $\varepsilon = 0.0003$.

With the weight of concrete, W_c, equal to 145 lb/ft^3, the 28-day cylinder strength of concrete, f_c, equal to 4,000 psi, and the unit strain, ε, equal to 0.0003, the concrete compression stress, f_c, equals about 1,000 psi.

$$f_c = E_c\varepsilon$$
$$E_c = W_c^{1.5}(33)\ (f_c')^{1/2}$$
$$E_c = 145^{1.5}(33)\ (4{,}000)^{1/2}$$
$$E_c = 3.64 \times 10^6$$
$$f_c = (3.64 \times 10^6)\ (0.0003)$$
$$f_c \approx 1{,}000 \text{ psi}$$

This is the stress associated with a pavement compression Δ_{cp} about equal to the original shrinkage crack width, Δ_s. Obviously, other assumptions will yield other stresses, but any reasonable assumptions will yield stresses of similar magnitude.

Pressures of these magnitudes have been measured by A. M. Richards of Akron University. In the paper "Causes, Measurements and Prevention of Pavement Forces Leading to Blow-ups" *(2)*, Richards describes the application of rock mechanics techniques to the measurement of pavement pressures. Essentially, the process consists of drilling a 1½-in. diam. hole in pavement suspected of being compressed and bonding strain gauges to the concave surfaces of the hole. At the same location, the pavement is then overcored with the hole located at the center of the core. After the core is removed, the changes in the strain gauges mounted within the core indicate the magnitude of the pressures that were compressing the core before its removal. Of 13 locations sampled in various Ohio counties, 3 cores indicated pressures in excess of 900 psi; 2 of these 3 cores were removed from pavement on bridge approaches. Other cores removed by Richards indicated a complete spectrum of stresses from about 70 psi up to and including a stress of 1,064 psi.

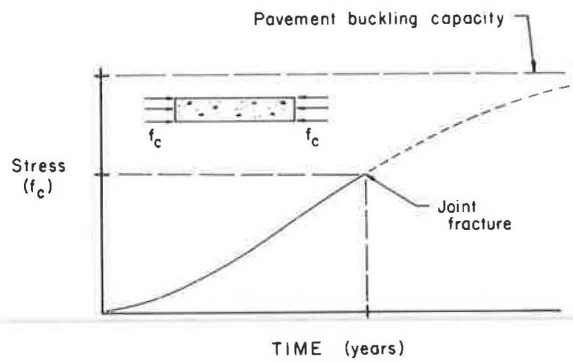


FIGURE 2 Pressure generation in jointed rigid pavement.

The generation of such pressures may be visualized as suggested in Figure 2. Illustrated is an idealized chart of the maximum compressive stress in a pavement, f_c, as it is generated over time. Initially, the pressure is insignificant as the joints are relatively clean and the joint seals are intact and functioning. However, as time passes and the joints begin to fill with debris, the pressure increases at a growing rate. As the joints continue to fill, the somewhat-compressible debris functions to minimize the infiltration of additional material, slowing the rate of joint infiltration and pressure generation. Somewhere along this hypothesized pressure generation curve, the pavement fractures adjacent to a joint, relieving some of the pressure; or the pavement buckles, relieving all of the pressure at the location of the buckle. Illustrated in Figure 2 is the pressure-generation curve for one particular pavement. Owing to the many factors which affect the performance of pavement joints, innumerable stress-time curves could be illustrated on this chart. This suggests that the fracturing could occur at an earlier or later time depending on the number of factors that combine to affect the behavior of such joints.

From observations of various projects throughout Ohio, it appears that the major factors contributing to joint infiltration, and consequently to pavement pressure generation, are the following:

1. Subgrade drainage,
2. Sealant quality and durability,
3. Temperature range and duration,
4. Pavement moisture content,
5. Deicing applications (including grit),
6. Traffic volume,
7. Joint spacing,
8. Pavement reinforcement,
9. Sealant maintenance, and
10. Other factors.

It is apparent by examining this list of factors that care should be exercised in the original design and construction to ensure the best functioning of pavement contraction joints by careful and thoughtful attention to Items 1, 2, 7, and 8. However, after a project has been constructed, the most efficient functioning of the contraction joints can only be influenced by maintenance attention to Item 9. Where modification and repair can be justified, Items 1 and 2 can effectively be improved. But as Figure 2 and the list of factors above would suggest, a pavement that has experienced a broad temperature range (requiring deicing applications), good subgrade drainage, high quality sealants, modest traffic volumes, and a good joint maintenance program should be expected to survive 25 to 30 years before the pressure generation reaches a point where pavement distress becomes evident. On the other hand, a similar pavement that has experienced the same temperature range and deicing applications, but with poor subgrade drainage, high traffic volumes, and minimal or no joint maintenance, should be expected to exhibit pavement distress and substantial bridge damage within 10 years.

PAVEMENT DAMAGE

As manifestations of pavement distress, such as pavement fracturing and pavement blowups, are so widespread, there is little need to illustrate them in this paper. Nevertheless, one photograph and a brief account of the problem seems appropriate. The term blowup is generally understood to mean an instantaneous fracture or buckling of pavement or both. It is sometimes triggered by the movement of vehicular traffic but it is caused by the high residual compression stresses within the pavement itself. The stresses are relieved or released by the blowup. The size of blowups has not been quantified. They can consist of minor localized joint fractures and slight buckling, major fractures with little or no buckling, and occasionally minor fracturing with significant buckling. An example of the latter is shown in Figure 3. This blowup occurred on Route 25 in Butler County, Ohio, in 1963. The apex of the buckle is close to 2 ft above the original pavement elevation.

As reported in a previous paper *(3)*, it is estimated that there were in excess of 500 pavement blowups in Ohio in 1970. During the late 1960s and early 1970s, Michigan reported 1,000 or more blowups/yr *(4)*. A bulletin of the Associated Press in Detroit contained a report stating that in 1971, Michigan experienced 1,387 blowups in the month of June alone. New York is reported to have experienced 1,590 blowups in 1 year with most of them occurring on the same day, July 3, 1966. It is apparent that at the time neighboring states were experiencing problems of similar magnitude. These are a few of the records that have been compiled by engineers concerned with the problem. Obviously, these records only serve to indicate the significance of

FIGURE 3 Pavement blowup.

the problem in states that have experienced the severest winter weather, have soils that have an adverse effect on subgrade drainage, have facilities subjected to a large amount of traffic, and at the same time have dry pavement policies and limited maintenance funds. Other states with better geographical and geological locations and with only moderate amounts of traffic to contend with, experience only modest amounts of pavement distress and consequently their incidents of pavement blowups remain unreported.

Of particular significance is the fact that pavement blowups are abrupt manifestations of severe pavement pressure. Unreported are those innumerable instances of pavement fracturing where progressive joint damage effectively moderates the generating pressures to sustainable levels.

Damage owing to generating pavement pressure can be manifested in other ways. One informative example is the distress exhibited by the standard approach slab used adjacent to integral types of bridges.

BRIDGE APPROACH SLAB DISTRESS

Figure 4 shows a cracked approach slab of a bridge on Route 271 in Cuyahoga County, Ohio. This bridge is a three-span continuous-concrete slab supported on capped pile piers and abutments, a rather commonplace type of bridge used in Ohio since 1946 for small stream crossings and minor grade separations. As shown in Figure 5(a), the deck slab of this and many of its companion bridges is keyed to what are considered flexible abutments. Flexible abutments are supported on a single row of piles that can flex back and forth without substantial resistance as the bridge deck lengthens and shortens in response to daily and seasonal temperature changes. Consequently, this bridge, which is constructed without deck joints to facilitate its longitudinal cyclic movement, is an early example of the integral type of bridges that are now being given more and more consideration by bridge engineers throughout the United States.

Figure 5(a) also illustrates the relationship of the approach slab to the bridge deck slab. Note that the seats for the two slabs are constructed at the same elevation. Since the spans of the bridge require a bridge deck slab considerably deeper than the short span of the approach slab, the end of the approach

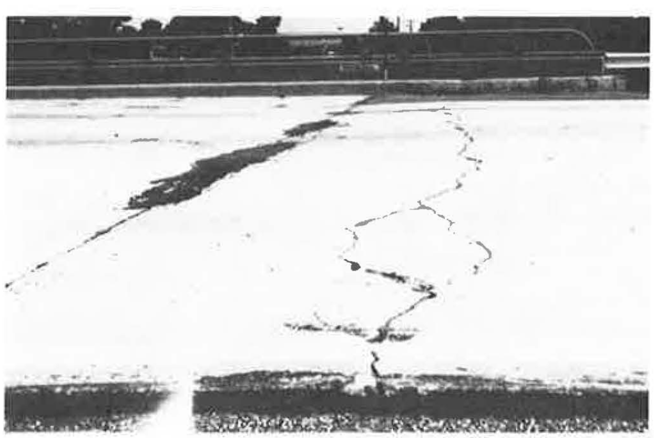


FIGURE 4 Approach slab distress.

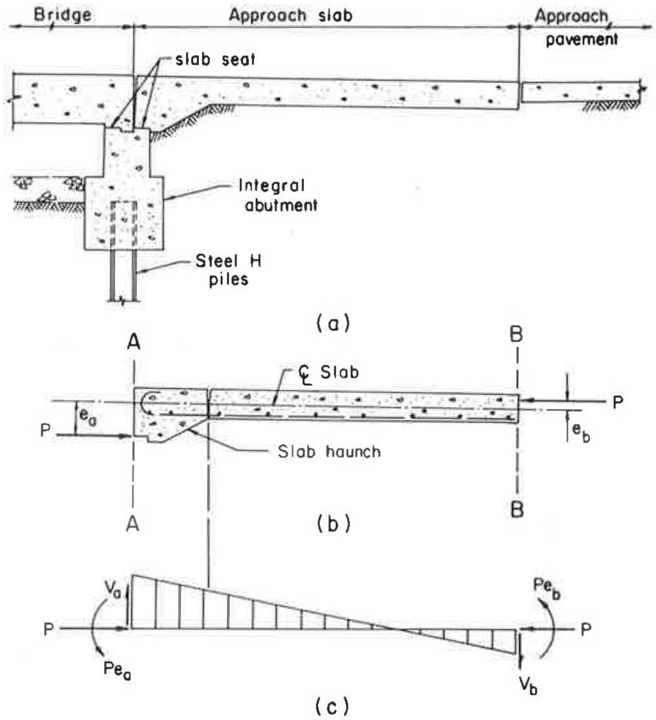


FIGURE 5 Bridge approach slab distress.

slab is provided with a haunch to accommodate the different slab thicknesses and the common deck slab and approach slab seats. Approach embankment consolidation is rather commonplace in Ohio, and this characteristic is suggested in the sketch where the slab at the pavement end is shown lower than the slab at the abutment end.

As longitudinal pressures are generated in approach pavements by the cyclic movement at debris-infiltrated pavement joints, the cyclic movement of integral bridges supplements or magnifies this pressure generation. This is because of the comparatively greater cyclic movement of the bridge, with its longer deck and greater exposure to temperature variations. Consequently, at these boundary joints, the greater cyclic movement is accompanied by proportionally greater expansion restraint and induced compression. The result of this pressure generation is indicated in Figure 5(b) where the magnified pressures are indicated as a longitudinal force, *P*, eccentrically applied through the top of the slab at the pavement end of the approach slab and at the bottom of the slab at the bridge end of the approach slab. The approach slab bending that accompanies these longitudinal forces, Figure 5(c), will generally culminate in cracking of the approach slab immediately adjacent to the approach slab haunch.

The moment diagram of Figure 5(c) indicates that the initial approach slab crack is located at the position of the largest bending moment and thinnest slab section. This cracking probably occurs at a moderate pressure, which is located somewhere in the lower portion of the pressure-generation curve shown in Figure 2. As pressure generation continues and the forces on the approach slab increase, the approach slab begins to buckle, rising at the crack and rotating about the ends of the slab. The end rotation tends to close the vertical joint between the approach slab and bridge slab, compressing the debris within the joint to such an extent that fracturing of the concrete adjacent to the joint begins. Evidence of this initial fracturing is suggested by the bituminous patches and sealant placed over and adjacent to the construction joint visible in Figure 4.

The transverse approach slab crack visible in Figure 4 and illustrated in Figure 5(b) is a characteristic of similar approach slabs located throughout the State of Ohio. In fact, these approach slabs can be used as indicators of pressure generation, as this cracking is a harbinger of other and more extensive pavement and bridge fracturing. As pavement pressure generation continues, the forces squeezing these approach slabs and the integral bridges between them culminate in either an extensive deterioration of the pavement joints, blowup of the approach pavements, or blowup of approach slabs. One such blowup is shown in Figure 6.

Shown in Figure 6 is one of the approach slabs of an integral type of continuous-concrete slab bridge located on Route 21 in Summit County, Ohio. In the right side of the photograph is the deck slab of the bridge. A joint between the deck slab and the abutment backwall is indicated by the line of sealant. Paralleling this joint and located at the apex of the approach slab, the remnants of the initial approach slab crack can be seen. Of significance in this photograph is the extensive bituminous patching on both the approach slab and the bridge deck slab, clear evidence of the fracturing that preceded the blowup. At the other end of this bridge, the approach slab has the characteristic transverse crack, and both the approach and deck slabs show evidence of extensive bituminous patching. The approach slab also contains a large longitudinal fracture, suggesting significant differential pressures being generated in the two parallel lanes of the approach pavement.

Obvious in these approach-slab photographs is the fact that the cyclic movement of approach-pavement sections and the cyclic movement of those short integral types of concrete slab bridges, movements which are restrained by debris infiltrated contraction joints, are generating pressures which culminate in significant distress. Such distress is manifested by: (a) progressive fracturing of approach pavements, (b) progressive fracturing of approach slabs, (c) progressive fracturing of deck slabs (d) blowups of approach pavements, (e) blowups of approach slabs, or potentially (f) a progressive failure of compressed bridges. Apparently, approach pavements generally are the weakest link in this particular chain of structures, so approach pavements fail before extensive approach-slab or bridge-slab distress. Integral types of bridges, with their substantially greater cross sections and solid jointless construction, generally suffer only minor surface fracturing. However, nonintegral types of bridges, bridges that have been provided with deck joints, have not been so fortunate.

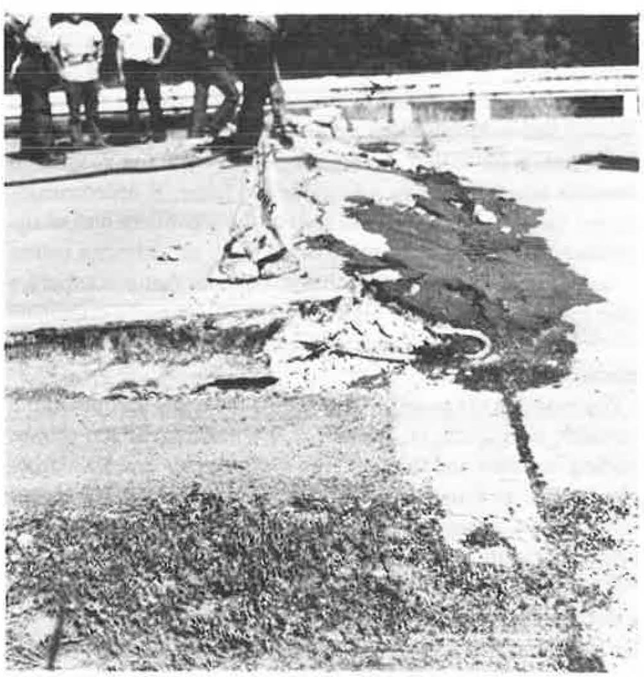


FIGURE 6 Approach slab blowup.

BRIDGE DISTRESS

In Ohio, the nonintegral type of bridge was generally provided with a 3-in.-wide open joint between the bridge deck and the abutment backwall and a 2-in.-wide sliding joint in the end dam (see Figure 7, Detail A). During construction, the superstructure and abutment parts of the end dam were bolted together while the concrete in the abutment backwall was placed assuring that, as cast, the joint in the end dam would be 2 in. wide at concrete placement ambient temperatures. In other words, no adjustment was made in the width of the end dam joint because of various ambient temperatures.

Shown in Figure 8 is a view of such an end dam and the top of the abutment backwall of a bridge on Route 77 in Summit County, Ohio — a nonintegral type of bridge constructed along the lines of the abutment illustrated in the sketch in Figure 7. In Figure 8, the bridge deck is shown on the right, the approach slab in the upper left hand corner, with the shoulder edge of the approach slab coinciding with the white striping. The fractured concrete to the left of the structural-steel end dam in the foreground is the top of the abutment backwall (see Figure 7). Of significance in Figure 8 is the opening in the end dam shown to be nearly 2 in. wide in the foreground, but entirely closed adjacent to the approach slab. This abutment, which is sup-

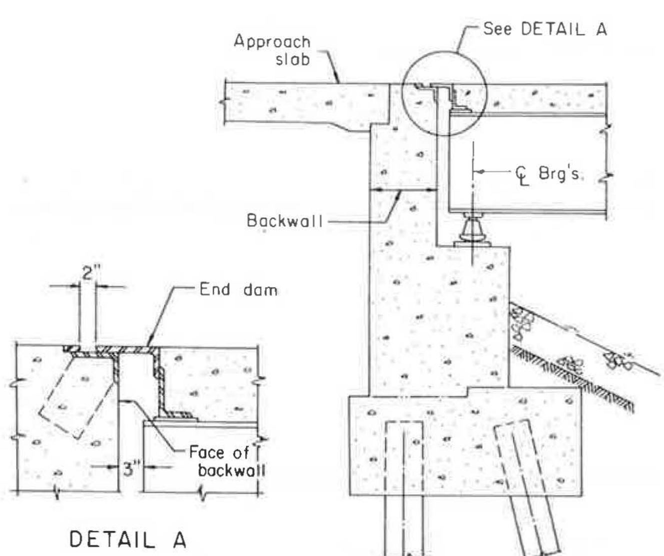


FIGURE 7 Nonintegral abutment.

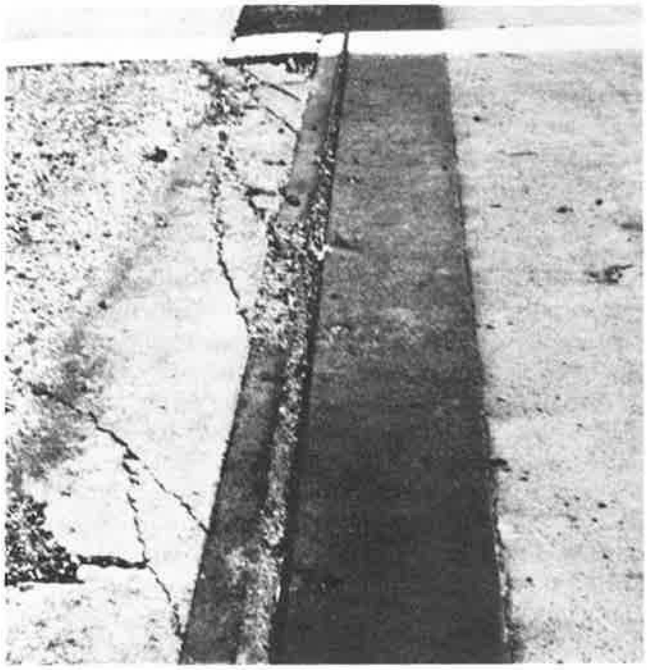


FIGURE 8 Abutment backwall fractures.

ported on short steel H piles driven to bedrock, is apparently sufficiently resistant to horizontal forces to prevent the abutment footer and bridge seat from being jammed against the bridge deck by the longitudinal thrust of the approach pavement. Consequently, the abutment backwall near the curbs has remained intact and essentially unmoved by the thrust of the approach slab. This is evidenced by the 2-in.-wide joint in the end dam visible in the foreground of Figure 8. However, in the roadway area, the approach slab has sheared completely through the backwall and closed the 2-in.-wide joint. Incidentally, this bridge was less than 10 years old when this photograph was taken, illustrating that in some projects pavement pressure generation begins early and generates quickly.

Evidence from other comparable bridges shows similar backwall fracturing. In addition, after the end dam has been closed (2-in. movement), the generating pavement pressure, supplemented by the pressure of an expanding bridge deck, is sufficient to thrust the backwall portion of the end dam under the superstructure end-dam angle (Detail A, Figure 7), completely lifting the superstructure off the bearings. Then, with the superstructure supported on the backwall, the reaction of the superstructure supplemented by the weight of the vehicular traffic continues the backwall fracturing until the backwall is completely fractured or until the pavement pressure is released by cutting pressure relief joints in the bridge pavements.

PAVEMENT GROWTH

At the Stanley Avenue Bridge in Dayton, Ohio, the concrete approach pavements were cut transversely so that 3-ft-wide bituminous filled pressure relief joints could be installed. The need for these relief joints became necessary when the deck joints at the bridge abutments were found closed and evidences of substantial longitudinal pressure were evident.

Periodic observations of these relief joints were made over a 5-yr period. The cutting of the pavements and the release of pressure was followed by a gradual and progressive closure of the joints. At one of the relief joints, for instance, the movement of the approach pavement into the joint amounted to 7½ in. As this movement occurred over a 5-yr period, an average movement rate of 1½ in./yr was experienced.

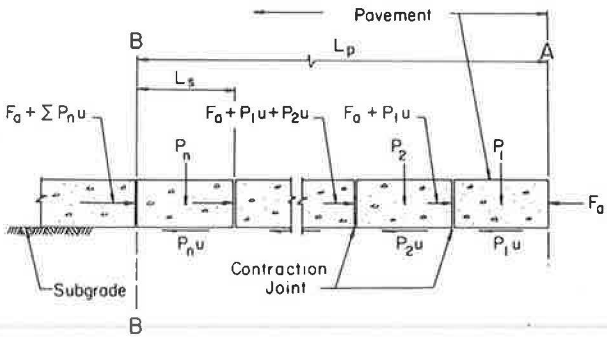
The approaches to this bridge consist of a pair of two 12-ft-wide pavements with a separately cast 4-ft-wide raised median. When the pavements were constructed, the presawed contraction joints were placed to coincide with the vertical joints in the median curbs. The longitudinal movement of the pavement was manifested not only by the movement of the pavement into the relief joints, but also by a differential movement of the pavement joints in relation to the median curb joints. The joints closest to the bridge showed the greatest differential movement, 7½ in., while the joints further removed from the bridge showed progressively less movement, with the joints located approximately 1,000 ft from the bridge showing no appreciable movement.

Consequently, based on the behavior of the pavement approaches to the Stanley Avenue Bridge and to similar pavements of many other bridges, it appears that up to 1,000 ft or more of pavement can contribute to the movement of the pavement at pressure relief joints. As these pavement movements are both progressive and accumulative, and as a substantial length of pavement contributes to this movement, it has come to be called "growth" to distinguish it from "expansion," the term usually used to refer to the minor component of cyclic movement.

The distinction between the terms growth and expansion is important. Many standard joint details that have been designed to facilitate the expansion component of cyclic movement, which have been name expansion joints, have been mistakenly selected to accommodate the growth of bridge-approach pavements. This is primarily because the individual making the selection associated the movement with the term expansion. The use of a name or label more indicative of the behavior being considered has been found to aid in a more appropriate selection of standard designs for particular applications, especially by individuals with limited background and experience.

Pavements experiencing growth are also subjected to substantial pressures. However, instead of the pressures being uniformly distributed throughout an extensive length of pavement, as is typical of restrained pavement, the pressures vary linearly along the length of the pavement experiencing growth. These pressures can be approximated by using the development suggested in Figure 9.

In Figure 9, line *A-A* represents the location of a pressure relief joint or a deck joint at a bridge abutment. Line *B-B* represents a location along the length of pavement where the pavement can be assumed to be fully restrained. The movement of the pavement sections within the length L_p is toward *A-A*, or it should be said that the maximum movement of the pavement because of the accumulated growth of the pavement occurs at line *A-A*. Resisting this growth is the force F_a at line *A-A* representing the resistance of the relief joint filler to compression or the resistance of a bridge abutment to longitudinal movement. Also resisting the growth are the forces due to



P_1, P_2, P_n = Weight of pavement sections

F_a = Resistance to movement by bridge abutment or relief joint

u = Coefficient of subgrade friction

T & W - Thickness and width of pavement

FIGURE 9 Forces associated with pavement growth.

subgrade friction, P_1u, P_2u, and so on. The summation of these forces must equal the total force generated at line *B-B* by restrained compression, Δ_{cp}, of the pavement at that location [see Figure 1(d)].

Since it has been shown how the restrained compression at a location such as *B-B* can equal or exceed a pressure of 1,000 psi, the total force in two lanes of pavement at such a location, $F_a + \Sigma P_n u$, can equal or exceed 1,300 tons.

$$F_a + \Sigma P_n u \geq (1{,}000 \text{ psi})(12 \text{ ft})(.75 \text{ ft})(2)(144 \text{ in}^2/\text{ft}^2)$$
$$F_a + \Sigma P_n u \geq 2{,}592{,}000 \text{ lb or } \approx 1{,}300 \text{ tons}$$

Assuming the resistance at line *A-A* to be negligible, $F_a = 0$, and the coefficient of subgrade friction, u, equal to 1.0, then

$$\Sigma P_n \approx 1{,}300 \text{ tons}$$

As ΣP_n equals the total weight of the pavement sections between lines *A-A* and *B-B*, the length of the pavement between these lines, L_p, can be computed as follows:

$$\Sigma P_n \approx 1{,}300 \text{ tons}$$
$$\Sigma P_n = (L)(12 \text{ ft})(.75 \text{ ft})(2)(145 \text{ lb/ft}^3)$$
$$(L_p)(12)(.75)(2)(145) \approx (1{,}300)(2{,}000)$$
$$L_p \approx (1{,}300)(2{,}000)/(12)(.75)(2)(145) \approx 1{,}000 \text{ ft}$$

This length is the same as the length of pavement contributing to the growth of the approach pavements of the Stanley Avenue Bridge. Of course, pressures greater or less than 1,000 psi would be capable of sustaining growth in pavements longer or shorter than 1,000 ft. Similarly, it can be shown that at the center of 1,000-ft length of pavement, the pressure would be about 500 psi. At locations closer to line *A-A*, the pressures would be proportionally less.

The generation of pavement pressure or the generation of pavement growth appear to be two sides of the same coin, or two major aspects of the same problem. The debris infiltration of contraction joints will result in pressure generation where the pavements are restrained (no relief joints or bridge joints) or growth generation where the pavements are not restrained (with relief joints or bridge joints, and so on). In many instances, growth will take place until all available space has been consumed. (All available space refers to space provided in pavement expansion joints, space available in pressure relief joints by compression of the filler, and space provided in bridge joints to facilitate the cyclic movement of bridge decks.) Then, as the pavements are restrained from further growth, pressure generation begins along the pressure generation curve illustrated in Figure 2. As both pressure and growth generation appear to be directly related to the debris infiltration of contraction joints, it goes without saying that the factors that have a significant effect on pressure generation have a similar effect on growth generation. Ideally, the solution to this problem is simple. All that is needed is a pavement joint design that would completely seal the joint against the intrusion of all foreign materials. Designs somewhat less than ideal would be suitable as a reasonable life cycle could be attained. However, it should be clear that present technology is not sufficiently developed to provide a cost-effective solution to the significant problem of debris infiltration at contraction joints.

Having described this background about pavement pressure and pavement growth, it is now appropriate to describe how these pavement problems helped to initiate the development of what have come to be called integral bridges.

INTEGRAL BRIDGES

During the past two to three decades, many bridge engineers have become more acutely aware of the relative performance of bridges built with cycle-control joints (sometimes known as expansion joints), and the relatively short bridges built without them. In most respects, short bridges without control joints have performed more effectively as they remain in service for longer periods of time with only moderate maintenance and only occasional repair.

Owing to the growth and pressure generation of jointed concrete pavement, many bridges built with cycle-control joints have been severely damaged. After the joints have been closed by pavement growth and the bridge squeezed by the generating pavement pressure, the restrained expansion of the bridge itself contributes substantially to the total pressure on the abutments. As described earlier, these pressures can easily exceed 1,000 psi or accumulatively the total force due to such pressures can exceed 650 tons/lane of approach pavement. When considering the design of abutments for short- and medium-span nonintegral types of bridges, forces of these magnitudes are irresistible. Many abutment backwalls have been fractured; other abutments have been split from top to bottom. In the longer bridges with intermediate deck joints, pier columns have been cracked and fractured as well.

(With respect to the massive bridge damage that many states have experienced owing to the pressures generated by bridge approach pavements, it is curious to note that these pressures are not recognized in the AASHTO Standard Specifications for Highway Bridges. This code mentions and quantifies earth pressure, water pressure, ice pressure, wind pressure, earthquakes, and so on. However, in this specification, there is not a single clue warning or cautioning the engineer about the probability of pavement pressures, one of the most destructive pressures known to bridge engineers.)

In areas of the country with low seasonal temperatures and an abundance of snow and freezing rain, the use of deicing chemicals to maintain dry pavements throughout the winter season has also had a significant effect on the durability and integrity of bridges built with cycle-control joints. The sliding plate joints of the shorter bridges and the open finger joints of the longer bridges have allowed the deck drainage, which has been contaminated with deicing chemicals, to penetrate below the surface of the decks and wash over the supporting beams, bearings, and bridge seats. The resulting corrosion and deterioration have been so serious that some structures have collapsed, others have been closed to traffic, many have required extensive repair, and most structures that have remained in service have required almost continuous maintenance to counteract the adverse effects of these chemicals. To help minimize or eliminate these corrective efforts, a whole new industry has been created.

Beginning about 1964, the first elastomeric compression seals were installed in a bridge in the United States to seal the bridge's cycle-control joints. From this first installation to the present time, numerous types of elastomeric joint seals have been developed and improved in an attempt to achieve a joint seal design that would be both effective and durable. Most of these designs have been disappointing; many of them leaked, and some required more maintenance than the original bridge built without them. The cost for seals has become alarming, for example, in one rehabilitation project for two moderate span bridges containing 9 joints, the joint remodeling and seal installation cost about $250,000. Yet these seals failed after the first winter of service and had to be removed because they were becoming a hazard to the movement of vehicular traffic. Other seals have remained intact but were not watertight. Some seals, notably the simple compression seals, strip seals, and the modular joints containing these seals as elements, have experienced a fair measure of success. But by and large, the many disappointments associated with the various types of seals have caused the bridge engineer to consider other options.

The costs of various types of bridges show marked differences. For two bridges essentially the same in most respects but with one provided with separate abutments, cycle-control joints and elastomeric joint seals, and the other provided with integral abutments [Figures 5, 10, and 11(a)], the one with cycle-control joints is usually significantly more expensive. In addition, bridges with integral abutments suffered only minor damage from pavement pressure, are essentially unaffected by deicing chemicals, and function for extended periods without appreciable maintenance or repair.

Consequently, more and more bridge engineers have begun to appreciate the merits of the integral bridge for short-to-moderate bridge lengths. Gradually, design changes have been made and longer bridges have been built and evaluated. Ohio's initial 1946 limitation on continuous concrete slab bridges [Figure 5(a)] was 184 ft. In 1962, a similar integral design was adapted for steel beam bridges. Currently, such designs are being used for continuous bridges with lengths up to 300 ft. Recently, a haunched three-span steel girder was built with integral type of construction at the abutments.

The attributes of integral bridges have not been achieved without cost. These bridges operate at very high stress levels, stress levels that cannot be easily quantified. These levels are

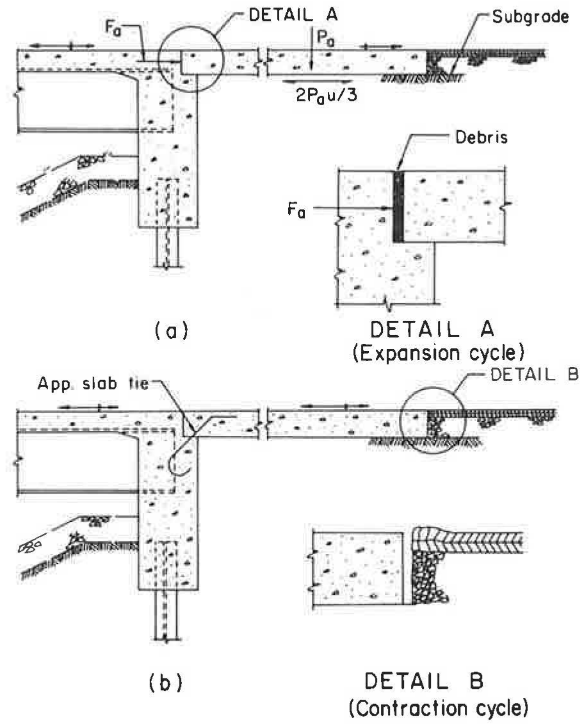


FIGURE 10 Approach slab joints.

significantly above those that are permitted by the AASHTO Standard Specifications for Highway Bridges. In this respect, bridge engineers have become rather pragmatic. They would rather build the cheaper integral bridges and tolerate these higher stress levels than build the more expensive jointed bridges with their vulnerability to destructive pavement pressures and deicing chemical corrosion.

Secondly, elimination of the cycle-control joints from the bridges has not eliminated the need for joints to facilitate their cyclic movement. Bridges will still cycle in response to changes in temperature. Consequently, as the joints are not incorporated in the bridges, they must be incorporated in the bridge approaches. Ignoring this need can lead to problems with the approach slabs.

APPROACH SLAB JOINTS

Consider the type of construction illustrated in Figure 10(a). This shows an approach slab transition between an integral bridge and flexible-approach pavement. On the left, the approach slab is supported on the slab seat, and on the right, the slab is supported on the subgrade. Due to the weight of a portion of the approach slab, say $\frac{2}{3}(P_a)$, and the friction between the approach slab and the subgrade, u, the frictional resistance, $2P_au/3$, makes the slab somewhat resistant to longitudinal movement. As the temperature drops below normal and bridge contracts to the left, this frictional resistance prevents the slab from moving with the bridge, thereby opening the vertical joint between them. Debris infiltration begins and continues each time the joint is opened during low-temperature contraction cycles. As the temperatures rise above normal, the bridge expands to the right. Since the pressure against the

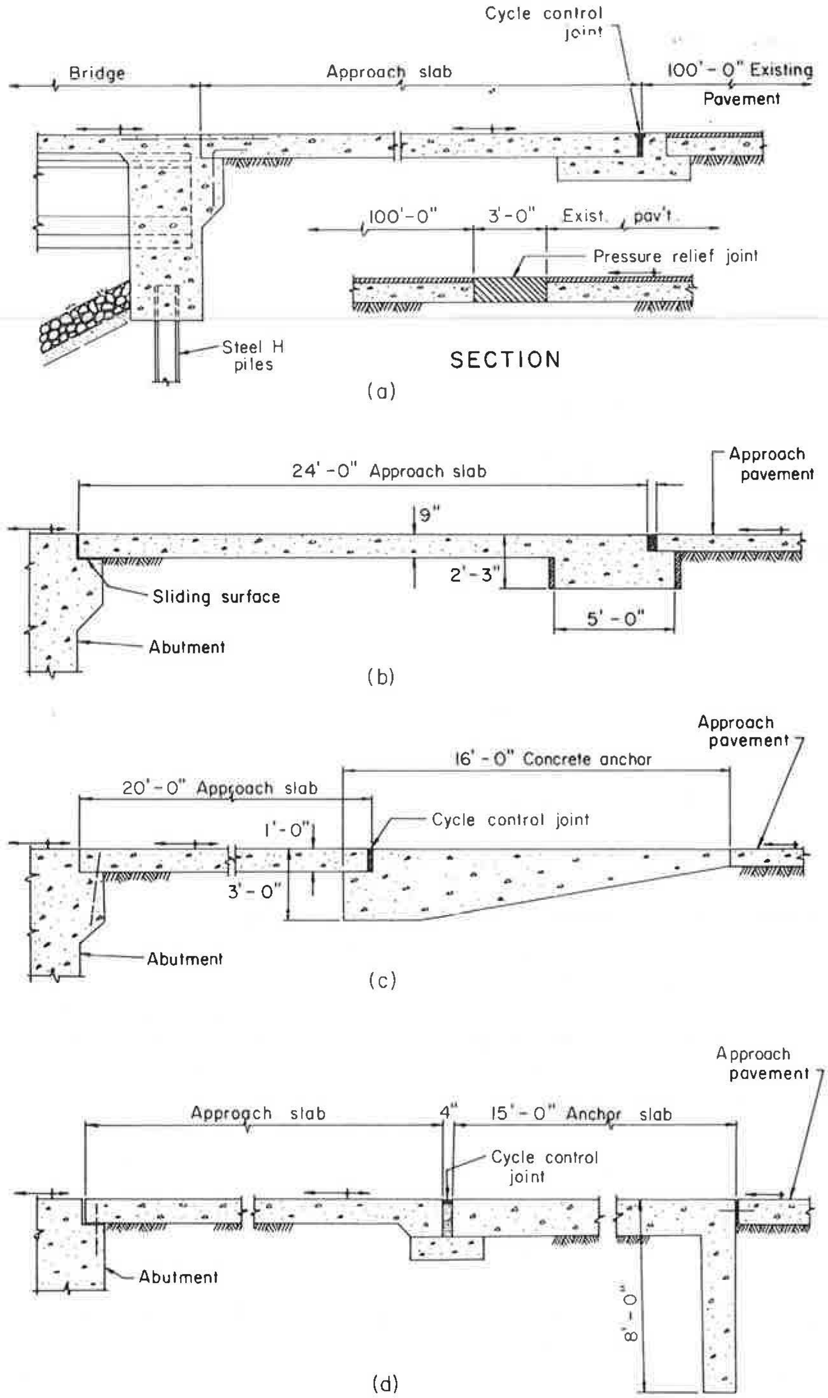


FIGURE 11 Bridge approach details.

debris-filled joint at the left end of the approach slab, F_a, is significantly greater than the frictional resistance of the subgrade, $2P_a u/3$, the approach slab is pushed to the right, compressing the flexible pavement in the process. As cyclic movement of the bridge and debris infiltration of the joint continues, the compressed debris in the joint causes the approach slab to be pushed further and further to the right, eventually shifting the slab off its seat entirely. Occasionally, during this shifting of the slab, the movement and weight of vehicular traffic causes fracturing of the edge of the slab or fracturing of the edge of the slab seat.

Where pressure relief joints have been installed in rigid approach pavements adjacent to such an approach slab, the behavior is essentially the same. The constant cycling of the integral bridge will push both the approach slabs and intervening pavement slabs toward the relief joints. To prevent such movement of the approach slabs, the bridge engineer has found it necessary to tie the approach slabs to the bridge abutments with steel reinforcing bars. However, since the intervening pavement slabs remain unattached, the cycling of the bridge and approach slabs pushes the intervening pavement slabs into the relief joint, thereby opening the pavement joints between the approach slabs and the pavement slabs.

Tying the approach slabs to integral bridges has consequences of its own. The approach slabs, functioning as part of the structure, add to the length of the structure responding to the cyclic temperature variations, thereby magnifying the cyclic response of the total structure to these variations.

With the approach slabs attached to an integral type of bridge, the joints between the approach slabs and the flexible approach pavements are then subjected to cyclic movement, and the response of these joints to that movement has other consequences.

Movement at this joint is illustrated in Figure 10(b), Detail B. During expansion cycles, the ends of the approach slabs compress the flexible pavement. Then, during the contraction cycle the joint is opened. This not only allows debris and accumulated roadway drainage to penetrate to the subgrade, but the slab movement also removes lateral support from the pavement adjacent to the joint, support that the pavement must have if in turn it is to remain intact and adequately support the movement of vehicular traffic. The water penetration can be especially serious when the bridge and approach slabs have been provided with curbs, as the accumulated drainage from the bridge is then channeled along the curbs and into the joints. The consequences of such uncontrolled water penetration can be highly destructive to the integrity of the subgrade and to the integrity of the pavement itself.

The destructive consequences of the cyclic movement of approach slabs adjacent to rigid pavement approaches has already been discussed in this paper.

As described in the preceding paragraphs, the bridge engineer has been able to solve many of the most pressing problems by first eliminating the bridge joints. Then some of his approach slab problems have been resolved by attaching the approach slabs to the integral bridges. In both instances, the need for a cycle-control joint has not been eliminated; this troublesome joint has just been moved out of the short- and medium-length bridges and onto the bridge approaches.

Considerable thought and attention has been given to the development of cycle-control joints for bridge approach pavements. But as the discussion in the foregoing and following pages seems to suggest, improvement is needed if the joints and approach slabs in this boundary area between the integral bridge and its approaches are to function effectively for extended periods of time.

BRIDGE APPROACH PAVEMENTS

Figures 11(a), (b), (c), and (d) illustrate design details that are now being used by four different states for the pavement approaches to integral bridges. (A number of other states are using similar details; some have not devised details especially for integral bridges; others are not building integral bridges except for the usual single-span slab, beam, or rigid frame bridge.) Based on the background described in this paper, the following observations appear pertinent.

The design shown in Figure 11(a) shows good recognition of the problems inherent in integral bridges and their approach pavements. Notice that the approach slab is anchored to the bridge; a 3-ft-wide pressure relief joint is used to protect the cycle-control joint and the bridge from the effects of pavement pressure, and the joint itself is furnished with a substantial sleeper slab. However, the filler used in the joint may not be entirely suitable. The joint appears to be made with a preformed filler and a surface-mounted compression seal.

Speculation about the performance of this joint suggests that as the bridge and attached approach slab contract, a void will be created in the joint below the surface seal and adjacent to the filler. Then water and debris infiltration will begin at the ends of the joint. During expansion cycles, the debris and filler will be compressed. After a number of cycles of contraction with infiltration and expansion with compression, the pressure will be sufficient to initiate shifting of the sleeper slab and the 100 ft of pavement into the relief joint. As shifting of the sleeper slab commences, the cycle-control joint will begin to widen, ultimately loosening the sealer, which would then be dislodged by the movement of vehicular traffic. The joint will then be exposed to upper-surface intrusion of water and debris. However, for this particular joint design, the degradation of the joint as described above may be a somewhat long-range process. So on a cost-benefit basis, the present design appears to have considerable merit.

It is also apparent that the degradation of the cycle-control joint for this particular bridge approach design will not have significantly adverse consequences. The presence of the pressure-relief joint will ensure against the generation of destructive pressure, either by the growing approach pavement or by the cycling of the integral bridge.

This particular design would be substantially improved if a material was available that would keep the joint filled throughout the complete movement cycle. With respect to the relief joint, the approach pavement will grow at an accelerated rate and additional joint sealing will be needed to keep the expanding contraction joint's surface sealed to minimize debris infiltration and pavement growth.

The design shown in Figure 11(b) has a number of faults. First, the approach slab is keyed to the subgrade and is not tied to the integral bridge. Consequently, the joint between the bridge and the approach slab will fill with debris during contraction cycles, and during expansion cycles the pressure of the deck against the debris will push the approach slab to the right. Concurrently, the 4-in.-wide polyurethane joint filler, if it is constructed in conjunction with jointed-approach pavement, will be compressed by a growing pavement in a period of about 4 years' time. Subsequently, the pressures generated in the restrained approach pavement, supplemented by the pressures created by the restrained expansion of the bridge itself, should be sufficient to cause fractures adjacent to pavement joints or pavement blowups.

The design in Figure 11(c) also suggests a number of problems. The approach slab is attached to the bridge and a pavement anchor is placed to prevent the growth of the approach pavement and protect the cycle-control joint and the bridge from the effects of pavement pressure. The control joint appears to consist of a 1-in.-wide preformed joint filler covered on the top surface with a poured joint sealant.

It is apparent that this pavement anchor could not develop enough passive resistance in the subgrade to resist a force equal to or greater than 650 tons/lane of pavement. It is also obvious that the preformed filler will not expand and keep the joint filled during contraction cycles. Consequently, the joint provided will quickly be compressed by the pavement pressures, and when these pressures become supplemented with the pressures generated by the restrained expansion of the bridge, joint fracturing or pavement blowups should be evident early in the life of this particular design.

The last illustration, Figure 11(d), also indicates clear recognition of the movements and forces that should be anticipated in this boundary area between approach pavements and integral bridges. The anchor slab furnished should be fully sufficient to resist all the pressure that can be generated by the approach pavement. The approach slab is anchored to the bridge. Again, the materials used in the cycle-control joint do not appear capable of filling the joint during contraction cycles as 4-in.-wide fillers are specified for a 4-in.-wide joint. With each contraction cycle, water and debris will infiltrate the joint, gradually changing the resilience of the filler. As the joint continues to fill with compressed debris, the pressures generated by the restrained expansion of the bridge should be sufficient to first initiate transverse cracking of the approach slab at the haunch, followed by slight buckling of the approach slab at the haunch during periods of peak ambient temperature. Again, the movements are so well controlled by this design that even the filler in this cycle-control joint should be reasonably effective for an extended period of time. It would appear that this design could be enhanced by a material that would keep this joint filled throughout the full expansion and contraction cycle of the bridge.

Not a single one of the cycle-control joints that have come to the author's attention were furnished with a filler that would keep the joint fully filled throughout the complete movement cycle. Only the design of Figure 11 appears to be able to protect the joint, and consequently the bridge, from the destructive pavement pressures. All of the designs seemed to be intended for very short bridges. It does not appear that any of the designs used short lengths of continuously reinforced concrete pavement adjacent to cycle-control joints to minimize pavement movements.

It is clear that the bridge-approach pavements and the cycle-control joints illustrated by these four designs (and by a number of other similar designs not illustrated) are limited in their ability to fulfill their function. It is also clear that agencies that are building integral bridges without cycle-control joints should expect high pressures in and fractures of rigid-approach pavements and distress and deterioration of flexible-approach pavements. Finally, it is also evident that fully effective bridge-approach designs will not be available to the transportation profession until members of the elastomer and sealant industries recognize and satisfy the need for more appropriate and more durable fillers and sealers for cycle-control joints.

SUMMARY

Described in this paper is some of the background that has motivated the recent interest in and burgeoning applications for integral types of bridges. The change appears to be a cost-effective solution for many of the bridge engineers' most pressing problems. Nevertheless, as integral bridges respond to temperature changes similar to their nonintegral counterparts, both types of bridges must be furnished with fully effective and durable cycle-control joints. Much improvement has been made in the last two decades in design and construction of cycle-control joints and joint seals for nonintegral types of bridges. Now a similar effort should be made to improve the present designs and develop new ones for the cycle-control joints for integral bridge approaches.

Responsibilities are also changing. The responsibility for cycle-control joint design has been shifting right along with the design. In the past, the bridge engineer was completely responsible for its design. Now that the joint has shifted to the bridge approaches, the responsibility for its design will probably shift right along with it. However, to develop truly effective and durable designs, the cooperation of many specialists will be necessary, including the construction, maintenance, test, and specification engineers. Most important, members of the elastomer and sealant industries will also have to cooperate if the specialized materials required for these applications are to be developed, tested, and marketed. The continued development of the integral type of bridge will depend upon such cooperation.

As has been suggested for some of the approach pavement designs illustrated in this paper, and, by inference, for many other similar designs being constructed, many of these designs will begin to experience significant distress in a rather short period of time. Consequently, the proliferation of integral types of bridges will result in considerable pavement and joint distress unless the pavement approaches to these bridges are furnished with more appropriate design details, which will facilitate the full cyclic movement of the integral bridges and more effectively accommodate the approach pavement characteristics. It is also clear that as the length of integral bridges increases, a great deal of cooperation between the bridge and pavement engineers will be necessary to ensure that the movement capacity of the cycle-control joints is increased commensurate with the increased length of the integral bridges.

ACKNOWLEDGMENT

The author wants to take this opportunity to thank the many state and province bridge engineers who responded to an NCHRP Synthesis survey about bridge expansion devices. The data received in that survey and the pavement approach design details that they have so willingly shared have formed the basis for this paper. However, the opinions and evaluations expressed herein are those of the author and do not necessarily represent those whose designs were illustrated.

REFERENCES

1. *National Experimental and Evaluation Program - Project 11 - Watertight Bridge Deck Joints.* Federal Highway Administration, Washington, D.C., 1977.
2. A. M. Richards, *Causes, Measurements and Prevention of Pavement Forces Leading to Blow-ups.* The University of Akron, Akron, Ohio, 1976.
3. M. P. Burke, Jr., The World's Most Expensive Pavement Expansion Joints. *Proc., Twenty-Sixth Annual Ohio Highway Engineering Conference*, The Ohio State University, Columbus, 1972, pp. 61–115.
4. C. J. Arnold, M. A. Chiunti, and K. S. Bancroft, *Jointed Concrete Pavements: Design, Performance and Repair.* Michigan Department of Transportation, Lansing, 1981.

Publication of this paper sponsored by Committees on Sealants and Fillers for Joints and Cracks and on Pavement Maintenance.