Performance Evaluation of Integral Abutment Bridges

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To establish a maximum safe length and design details for zero skew steel and concrete bridges with integral abutments, a survey of the highway departments of all 50 states was conducted. The findings of the survey include maximum lengths of integral abutment bridges being constructed, advantages and disadvantages of these bridges as perceived by the various states, and the extent of field observations done. The variation in the design assumptions and length limitations used as well as the problems associated with thermal movement experienced by the various states are discussed. A critique of the design and construction practices of the states that have pioneered the use of integral abutment bridges is also presented.

Before World War II most bridges with an overall length of 50 ft or more were constructed with some form of expansion joints. Periodic inspection of these bridges has revealed that the expansion joints tend to freeze and close and, therefore, do not operate as intended. Closer inspection of such bridges has also revealed that no serious distress was associated with the freezing or closing action of the joints.

Studies conducted by the Ohio Department of Highways have shown that the increase in internal stress in the approach slabs and not in the bridge slabs has been the main cause of bridge failures (*1*). Such a problem could be resolved easily by providing adequate expansion joints in the approach pavements without any expansion joints on the bridge at all. This has led to the advancement of the case for the construction of continuous bridges. Bridges in which the girders are fixed at the abutments, thereby requiring no expansion joints at the abutment, are called integral abutment bridges. Presented are findings of a survey conducted to evaluate the current design practices and performance of such bridges.

BACKGROUND

Most bridges in the world are constructed with some form of expansion joints. An example of these bridges is shown in Figure 1, and a detail of an abutment in which such a joint is provided is shown in Figure 2. This type of an expansion joint at the abutment causes the water from the backfill and roadway to penetrate into the bearing areas and onto the bridge seats. The joints could then potentially be forced to close, resulting in broken backwalls, sheared anchor bolts, and damaged roadway expansion devices. These problems and the maintenance costs associated with them have accelerated the development of integral abutments in the United States.

The routine use of integral abutments to tie the bridge superstructure to the foundation piling began in the United States about 30 years ago. The states of Kansas, Missouri, Ohio, and Tennessee are some of the early users. This method of construction has steadily grown more popular. In addition to being aesthetically pleasing, integral abutment bridges offer the advantage of lower initial costs and lower maintenance costs, because they eliminate expensive bearings, joint material, piles for horizontal earth loads, and leakage of water through the joints. An example of a bridge with integral abutments is shown in Figure 3. In this type of bridge, the thermal stresses are transferred to the substructure via a rigid connection in which the abutment contains sufficient bulk to be

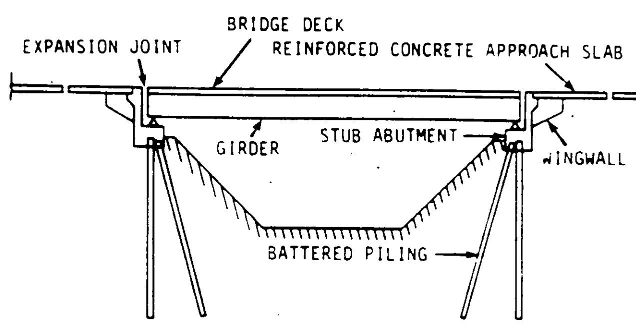

FIGURE 1 Cross section of bridge with expansion joints.

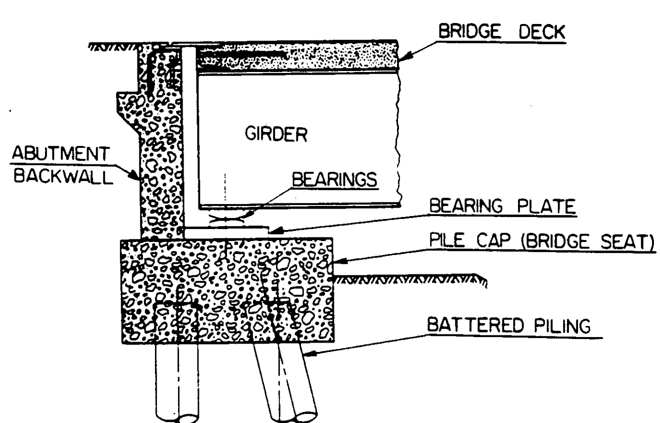

FIGURE 2 Abutment detail of bridge with expansion joints.

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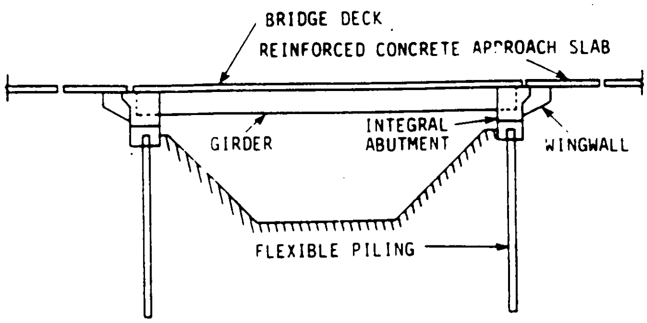

FIGURE 3 Cross section of bridge with integral abutments.

considered a rigid mass. Various construction details have been developed by the various states to accomplish the transfer; a few of these are shown in Figure 4. A positive connection to the girder ends is generally provided by vertical and transverse reinforcing steel. This connection provides a mechanism for full transfer of temperature variation and live load rotational displacements to the abutment piling. Also, because of the confinement of the bridge slab between the two abutments, the horizontal displacements can be transferred to the end of either the abutment or the approach slab, where the approach slab is tied to the abutment. This is because, in this type of design, the bridge acts as a rigid frame.

The semi-integral abutments, as shown in Figure 5, are designed to minimize the transfer of rotational displacements to the piling. They do, however, transfer horizontal displacements and also allow elimination of the deck expansion joints. Rotation is generally accomplished by using a flexible bearing surface at a selected horizontal interface in the abutment. Allowing rotation at the pile top generally reduces the pile load.

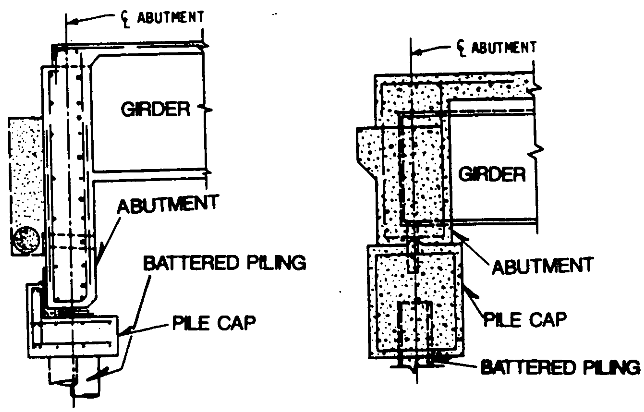

FIGURE 5 Details of semi-integral abutment used by various states.

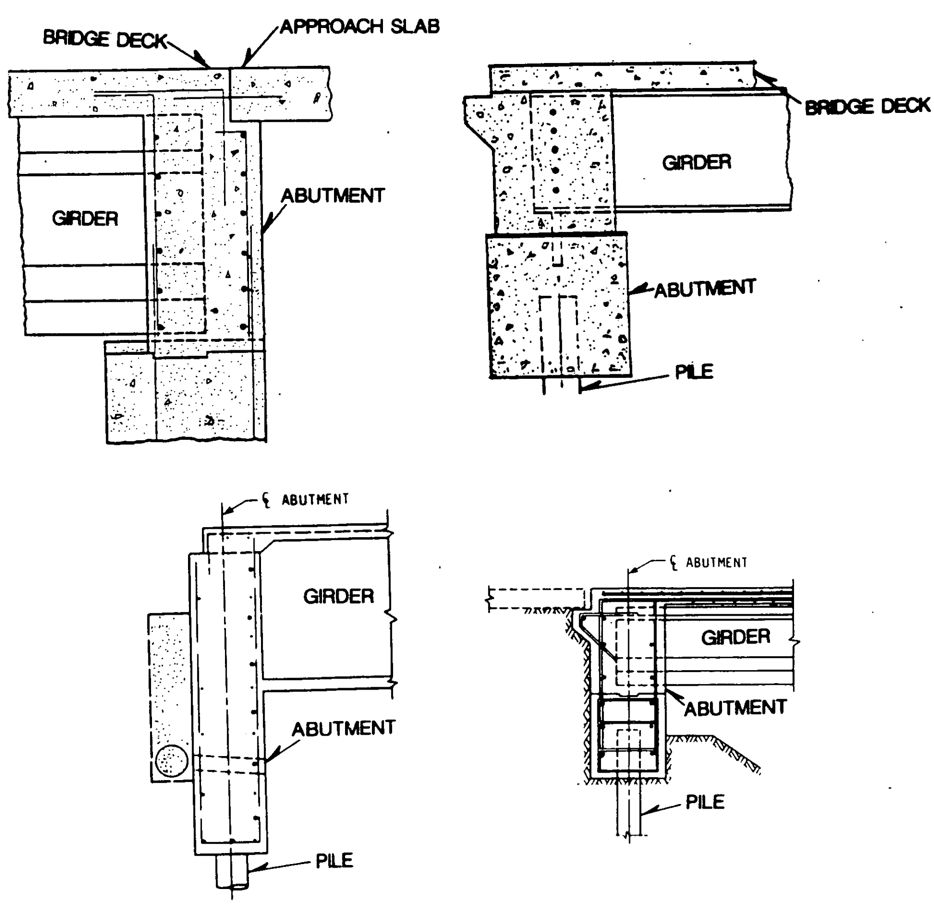

FIGURE 4 Details of integral abutment used by various states.

REVIEW OF STUDIES AND CONSTRUCTION DETAILS

Many surveys have been conducted concerning the use of integral abutments (*2–5*). These surveys indicate that there are significant variations in the design limitations and criteria currently followed. Many states have not felt comfortable using a system that does not contain some "free space" to accommodate the displacements caused by temperature changes.

Today, more than 80 percent of the state highway agencies have developed design criteria for bridges without expansion joint devices. Most of the states initially used integral abutments for bridges that were less than 100 ft long. Later, allowable lengths were gradually increased on the basis of good performance of the connection details used. However, the increased lengths have varied in the different states.

Full-scale field testing and sophisticated national design methods were not commonly used as a basis for increasing the allowable length. This led to wide variations in criteria for the use of integral abutments from state to state. FHWA recommends integral abutments for steel bridges less than 300 ft long (*6*). For pre- or posttensioned concrete bridges and for unrestrained bridges (i.e., bridges on which the abutments are free to rotate, such as a stub abutment connected to a row of piles or an abutment hinged to the footing), FHWA recommends a 600-ft maximum (*5*).

The limits of allowable horizontal movement that will cause objectionable pile stresses have not been established. In fact, the value constituting an objectionable pile stress has not yet been defined. This partly explains the wide variation in the design criteria used for integral abutment bridges by the various state highway agencies.

A survey of 50 states and a review of the literature have shown that little theoretical or experimental work has been conducted to establish limits or to develop design procedures for integral abutment bridges. Iowa State University is one of the few institutions conducting some field and model tests but has not yet concluded these studies.

SURVEY QUESTIONNAIRE AND RESULTS

To develop a design procedure and related construction details for integral abutment bridges, it is imperative to understand not only the state of the art as far as analysis and design are concerned but also the current practices followed by the various highway agencies. These practices have been developed by observing the actual performance of such bridges constructed over the years. Also, a thorough understanding of the problems that these agencies have encountered, in particular the "phobia" surrounding the design and detailing of integral abutment bridges, is important; one could either suggest solutions to these problems or at least be able to explain them. Only then could a practical design—one that could be acceptable and used by most state agencies—be possible.

To develop a design procedure and related construction details, it is imperative to know the disadvantages as well as the advantages of the current details used by the various state agencies. The construction details that have shown signs of failure need to be studied and scrutinized to find shortcomings and ways to improve them.

Other problems that need to be addressed are (a) how the states compensate the active and, most importantly, the passive soil pressure behind the abutments and (b) whether they leave a free space behind the abutments to allow for thermal expansion and, if so, what construction details they have used.

A survey questionnaire was prepared to obtain this information from the departments of transportation across the United States. It was also intended to collect additional information about the history of these bridges in each state to determine whether field testing had been conducted or is currently being conducted to monitor their performance. States were requested to confine their answers to 90-degree bridges and to send a copy of their design procedure and standard details for integral abutment bridges. After the responses were obtained and analyzed, some departments were contacted on the telephone for additional information.

Summary of Responses Obtained

Of 38 responses received, 29 indicated that their states either were using or had used integral-type abutments. A few states, such as Florida, Pennsylvania, and Virginia, were just beginning to use them; their first integral abutment bridges were constructed in the 1980s. On the basis of a review of the survey results, several states were contacted later for additional information to gain a better understanding of successful design details and assess the performance of relatively long integral abutment bridges.

The results of the correspondence and the telephone conversations with the bridge engineers are presented in Table 1. Each column in this table corresponds directly to the survey questionnaire that was sent to all 50 states. The following information corresponds to each column of Table 1:

- Column 1. STATE: name of the state supplying the information.
- Column 2. USE: whether integral abutment-type bridges are being used (Y = yes; N = no).
- Column 3. TYPE: type of road normally using integral abutment-type bridges (1 = state; 2 = county; 3 = all).
- Column 4. LENGTH: the span length each state uses for integral abutment-type bridges. The following code numbers are used to categorize various span lengths:

1 = <200 ft	5 = 500–600 ft
2 = 200–300 ft	6 = 600–700 ft
3 = 300–400 ft	7 = 700–800 ft
4 = 400–500 ft	8 = >800 ft

- Column 5. EXP. JTS: if expansion joints are used in the approach slabs, at what distance they are from the abutments.
- Column 6. PILE CAP: Whether pile caps are used (Y = yes; N = no).
- Column 7. 1st IAB: year the first integral abutment-type bridge was constructed in the state.
- Column 8. LONGEST: length of the longest integral abutment-type bridge constructed with the following materials: (1) steel, (2) cast in place, and (3) pre- or posttensioned.

TABLE 1 Summary of Responses

STATE	USE	TYPE	LENGTH								EXP JTS	PILE CAPS	1ST IAB	LONGEST			BKF MAT	DIS ADV
			1	2	3	4	5	6	7	8				1	2	3		
ALABAMA	N																	
ARIZONA	Y	3	x								NO	Y	76	296		162	G	10,14
ARKANSAS	N																	6
CALIFORNIA	Y	3	x	x							YES	Y	50	240	260	240	G	10,14
COLORADO	Y	3	x	x	x						200'	Y	75			450	G	4
FLORIDA	Y	1	x								0'	Y	85		200		N.A.	N.A.
GEORGIA	Y	3		x		x					N.A.	Y	77	340		477	N.A.	N.A.
HAWAII	N		x	x							N.A.	N.A.	67			100	N.A.	3
IDAHO	Y	3	x	x							50'	Y	72	266	262	521	G	5,2,4
ILLINOIS	Y	3	x								20'	Y	86	230	100	250	G	6
INDIANA	Y	3	x								YES	Y	72		150		G	NONE
IOWA	Y	3	x	x							YES	Y	64		130	300	G	2,3
LOUISIANA	N																	
MICHIGAN	N	2		x							YES	Y	70	250			G	N.A.
MINNESOTA	Y	3	x								YES	Y	60	200			G	2,3
MISSISSIPPI	N		x										40	30	40		N.A.	N.A.
MISSOURI	Y	1	x	x	x	x	x				50'	Y	60's	500	600		N.A.	2
MONTANA	Y	3	x	x							0'	Y	N.A.	150	108	230	G	4
NEBRASKA	Y	1		x							0'	Y	69	464		260	G	13
NEVADA	Y	3	x	x	x						0'	Y	74	294		530	G	NONE
NEW JERSEY	N																	
NEW HAMPSHIRE	N																	
NEW MEXICO	Y	3	x	x	x	x					NO	Y	77	250		450	G	16
N. CAROLINA	N																	
N. DAKOTA	Y	1	x	x		x					20'	N	60	350	100	450	G	1,2,3
OKLAHOMA	Y	1	x	x							15'	Y	79		279	210	G	10
OREGON	Y	3	x	x							NO	Y	30		385	420	G	3
PENNSYLVANIA	Y	1	x	x							YES	Y	86			250	G	8
S. CAROLINA	Y	1	x								20'	Y	74	220	150			N.A.
S. DAKOTA	Y	3	x	x	x						20'	Y	48	354	324		N.S.	NONE
TENNESSEE	Y	3	x	x	x	x	x	x	x	x	0'	Y	70	416	460	927	G	NONE
TEXAS	N																G	
UTAH	Y	3	x	x	x						NO	Y	50's	450	150	400		9
VIRGINIA	Y	1	x								NO	Y	82	180			G	2,15
WASHINGTON	Y	1	x								NO	Y	20		350		G	11,7
W. VIRGINIA	N																	
WISCONSIN	Y	3		x							NO	Y	70	100	150	300	W.G.	7
WYOMING	Y	3	x	x							NO	Y	60	356	199	172		N.A.
																	G	
																	G	

- Column 9. BKF MAT: type of material used behind the abutments (*G* = granular, *W.G.* = washed gravel; *N.S.* = no specification; and *N.A.* = no answer).
- Column 10. DIS-ADV: disadvantages of integral abutment-type bridges. The numbers are the disadvantages listed separately in Table 2 as perceived by the various states.

Trends Observed in the Responses

Most of the states that use integral abutments, as reflected in Table 2, have developed specific guidelines (policies) concerning allowable bridge lengths. The basis of these guidelines, however, is largely empirical. It was hoped that these guidelines would have been developed according to results of some rational analysis and design procedures or experiments conducted to find anticipated movement of the bridge or approach slab and the pile stresses. However, it was found from the responses that all states used empirical data to arrive at the maximum allowable bridge lengths. It is evident that much of the progress in the use of integral abutments resulted from successive extension of limitations based on acceptable performance of prototype installations. The following points are summarized from the responses to the survey questionnaire and subsequent correspondence and telephone conversations with several states:

1. Most of the states cited the reason for using integral abutments as cost savings. They pointed out that typical bridge designs that use integral abutments require less piling, have simpler construction details, and eliminate expensive expansion devices. Some states indicated that their primary reason for using integral abutments was that they eliminate the problems encountered with use of expansion joints. A few respondents pointed out that simplicity of construction and lower maintenance costs were their primary motives for using integral abutments.

2. The span length and limitations currently being used for bridges with integral abutments are given in Table 1. In summary, for states that use this type of design, the acceptable range of limitations is as follows: steel, 200 to 300 ft; concrete, 300 to 400 ft; and prestressed concrete, 300 to 450 ft. A few states, such as Idaho, Missouri, Nevada, Tennessee, and Utah, use longer limitations for each structure type. Typically they have been building integral abutment bridges longer than most other states and have had good success with them. The move toward the use of integral abutment bridges for longer spans is based on the excellent performance observed for shorter-span bridges and the maximum benefit from what many regard as a very-low-maintenance, dependable abutment design. The difference in concrete and steel length limitations reflects the greater sensitivity of steel in reacting to temperature changes. Although the coefficients of expansion (α) are nearly equal for both materials (for concrete $\alpha_c = 60 \times 10^{-7}$ in./in./°F and for steel $\alpha_s = 65 \times 10^{-7}$ in./in./°F), the relatively large mass of most concrete structures makes them less reactive to ambient temperature changes. This is reflected in the design temperature variation specified by AASHTO (*6*), which states:

> Provision shall be made for stresses or movements resulting from variations in temperature. The rise and fall in temperature shall be fixed for the locality in which the structure is to be constructed and shall be computed from an assumed temperature at the time of erection. Due consideration shall be given to the lag between air temperature and the interior temperature of massive concrete members or structures.

3. Almost all states indicated that a free-draining backfill material be used behind the abutments. One major problem seems to be to achieve a 95 percent compaction requirement of backfill. This requirement eliminates settlement of the approach slab.

4. Most states reported that construction and maintenance costs are lower if integral abutments are used. The following are some comments made about construction and maintenance problems using integral abutments:

a. Field placement of precast beams could be a problem, since cranes cannot get close to the abutments because the backfill is not placed until after the beams are placed.

b. The proper compaction of backfill is critical.

c. Careful consideration at the end of the bridge is necessary.

d. The effects of elastic-shortening after posttensioning should be carefully considered.

TABLE 2 Disadvantages of Integral Abutment Bridges as Perceived by Various State Departments of Transportation

1.	Increased earth load can cause abutment cracking.
2.	Skew over 20° cannot be accommodated.
3.	Can only be applied to short bridges.
4.	Cracks developed in the asphalt backface of the abutments, as a result of which a bump at the end of bridge or approach slab could appear.
5.	Integral abutment bridges are limited to pile supported abutments, and drill shafts cannot be used.
6.	Lack of rational method for predicting behavior. Also, thermal stresses are unknown.
7.	Temporary shoring will be required in precast bridges.
8.	Crane cannot go close to place precast beams, since backfill is put in after the beams have been placed. Therefore, cranes with large booms are required.
9.	Good details for tying the approach slab to the abutment are not available.
10.	Longer than normal approach slab is required.
11.	Limits future modifications, such as widening.
12.	Cracks in slab, end diaphragm or wingwalls are possible.
13.	Wingwalls cannot be tied to the abutment.
14.	Erosion of the approach embankment caused by water intrusion.
15.	Field problems exist when constructing a bridge on a steep slope.

e. Wingwalls may need to be designed for heavier loads to prevent cracking.

f. Adequate pressure relief joints should be provided in the approach slab to avoid overstressing of the abutments.

g. Positive tie connection between the approach slab and abutment may be necessary to avoid opening in cold weather.

Review of Design and Construction Details in Selected States

To become more familiar with the designs and construction practices of various highway agencies in the United States, a critique of integral abutment bridge developments in a few selected states (pioneers in designing such bridges) is summarized in this subsection.

Tennessee

Numerous integral abutment bridges have been constructed and their field performance has been evaluated in Tennessee. In an unpublished memorandum, Edward Wasserman, of the Tennessee Department of Transportation (DOT), wrote:

> In Tennessee DOT, a structural engineer can measure his ability by seeing how long a bridge he can design without inserting an expansion joint.

He also explained that in the past 20 years, nearly all their new highway bridges with spans ranging several hundred feet long have been designed with no expansion joints, even at the abutments. The largest bridges include a 927-ft prestressed concrete bridge, a 416-ft steel bridge, and a 460-ft cast-in-place concrete bridge. Details of a typical bridge are shown in Figure 6. The Tennessee DOT reports (*7*):

> We have found neither the deck elongation nor the superstructure stresses to be abnormal. All measured stress data were lower than predicted. Exactly why, we don't know, but we think we have some of the answers. One factor appears to be creep in concrete. If concrete is expanded or contracted slowly, as by temperature, it creeps. Stresses due to shrinkage/expansion don't reach the level predicted. To make theory better fit reality, in the case of concrete we have reduced its thermal modulus of elasticity to one-third that's used for dynamic loads.
>
> In addition, temperature cycling of concrete bridges appear to reach lower peaks than in steel. Apparently concrete's greater mass provides a heat sink. Thus, its temperature tends not to rise as high nor as low as theory predicts. We design Tennessee bridges in concrete for a temperature range of 20° to 90°F, and steel superstructure bridges for a range of 0° to 120°F. Based on these ranges and thermal coefficients of expansion for respective materials, we design for 0.505 inch of movement per 100 feet of span in concrete, and 0.936 inch of movement per 100 feet of span in steel.

To demonstrate this procedure, the 2,800-ft-long Kingsport bridge in Tennessee can be chosen as an example, in which the center of the bridge is assumed to be neutral or fixed. The total movement of the superstructure of this bridge, which is made of concrete, is obtained by multiplying 1,400 (equals span) by 0.505 (equals rate of expansion), which yields the total movement at each abutment equal to 7.07 in. This bridge has performed within this range since 1978 (*8*).

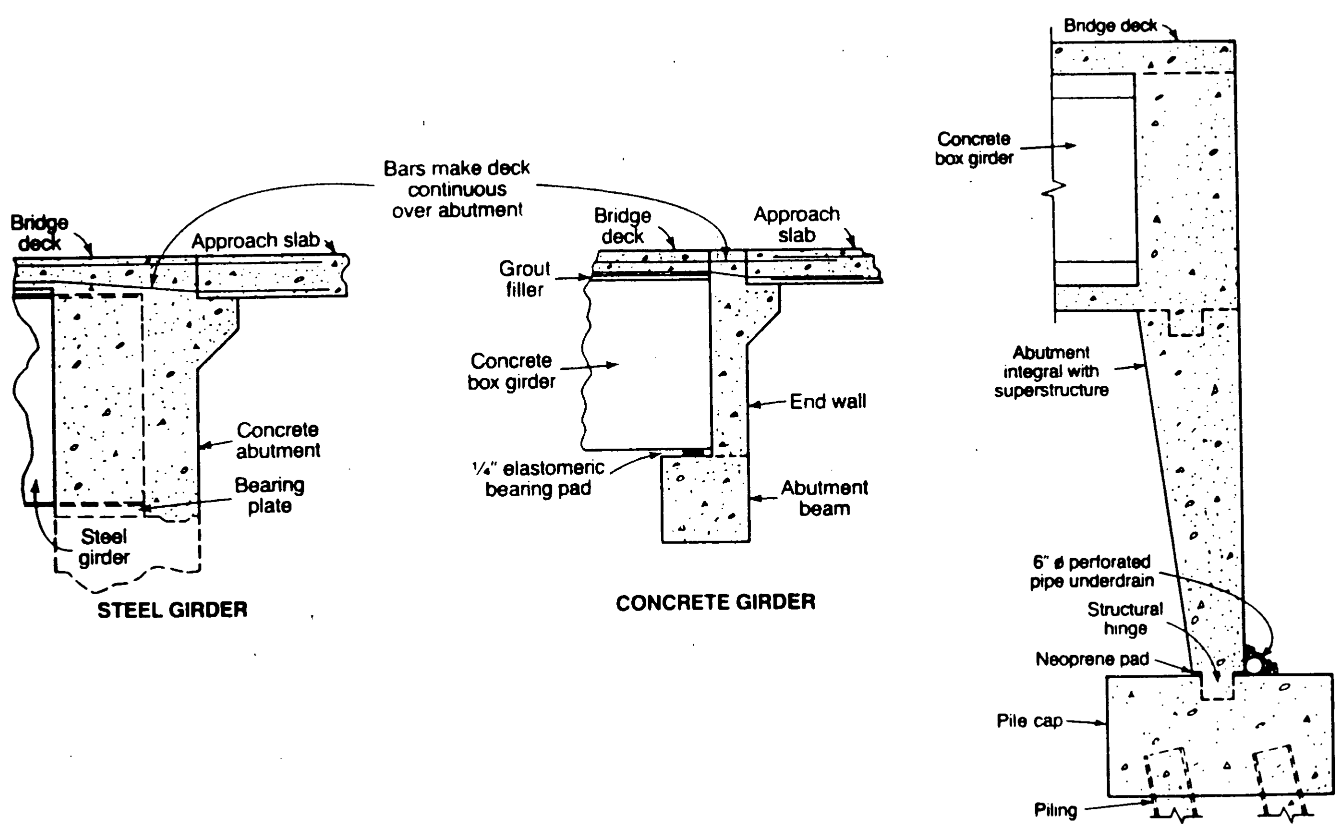

FIGURE 6 Typical details of integral abutment bridge construction used in Tennessee.

Tennessee DOT also explains (*7*) that when they are asked, "How do you set about to reduce or eliminate expansion joints?" they give the following explanations:

1. "We take advantage of pile translation and rotation capabilities."
2. "By modifying foundation conditions, if feasible."
3. "By taking advantage of reduced modulus of elasticity of concrete for long-term loads (1,000,000 psi versus 3,000,000 psi)."
4. "By allowing hinges to form naturally or constructing them."
5. "Employing expansion bearing, where necessary."

Where there is a concrete pavement approaching a concrete deck, Tennessee DOT installs a compression seal joint between them. But where the interface is asphalt to concrete, they believe that no special treatment is necessary. Regarding this, they point out: "This will eventually cause some local pavement failure and a 'bump,' which is a minor problem compared to joint maintenance." In summary, Tennessee's 20-year experiment with integral abutment bridges has proven that, for thermal movements up to 2 in., both immediate construction savings and long-term maintenance savings can be realized by total elimination of joints.

California

One of the reasons for the popularity of jointless bridges in California is their performance during the 1971 San Fernando earthquake. During this earthquake, the jointless abutment-approach fill concept suffered less overall damage than the jointed abutment-type bridges. Also, because of initial cost and inherent problems with expansion joints, especially at abutments, California DOT has pointed out (*9*):

> It has been general practice in California since 1971 to construct highway structures without expansion joints. Consequently, most structures less than 350 feet long built since that time are jointless. California also has over 100 jointless structures with lengths exceeding 350 feet. Even on most structures with expansion joints, the abutments are jointless.

California DOT has indicated (G. D. Mancarti, Memo to Designers):

> The major benefits of integral abutment are its low initial cost, its effectiveness on absorbing seismic loads and its ability to accommodate structurally, relatively large thermal movements. Reinforced concrete bridges up to 400 feet in length have shown no evidence of structural distress of the abutments from thermal movements.

Because water intrusion has been the main problem with this design, California DOT connects the approach slab directly to the abutment and extends it over the wingwalls. Also, an underlying drainage system is provided to give additional insurance. California's limitation on bridge movement (including temperature, creep and long-term prestressed shortening) is a maximum of 1 in. at the expansion joint between the approach slab and the adjoining pavement.

South Dakota

South Dakota has extensive experience in the use of integral abutments, particularly for steel bridges. It is also one of the first states to conduct a full-scale testing program to evaluate the performance of integral abutment bridges. A brief review of this program is presented in this section.

In this testing program they have measured the magnitude of the stresses induced by thermal movements in the girder and the upper portion of the steel-bearing piles of integral abutment-type bridges. A full-scale model representing the end portion of a typical highway bridge was constructed and tested to simulate the following four construction stages:

Stage 1: Girder welded to piles only.
Stage 2: With an integral abutment in place.
Stage 3: With abutment and slab.
Stage 4: Backfill completed.

For each stage, the test specimen was subjected to a series of predetermined longitudinal movements via hydraulic jacks to simulate expansion and contraction caused by temperature changes. The fact that the test specimen was designed and constructed as a full-scale model that represents the end portion of an actual bridge confirmed this approach as a field study instead of as a model study (*3*). Many variables (e.g., compressive strength of concrete, placement of steel) that normally would be under tight control in the laboratory were not controllable in the field. In addition, a single concentrated load was applied to the girder to simulate the thermal stresses, which in reality should have been applied by a distributed load. This certainly would have introduced some inaccuracy into the results. The conclusions based on these test results were as follows:

1. The induced movement and shear force in the girder, caused by temperature changes alone, are usually smaller than the overstress allowance made by AASHTO (*6*) for combined loadings.
2. The integral abutment acts as if it were a rigid body.
3. Thermal movements larger than 0.5 in. may cause yielding in the steel piling.

More studies are required to prove the accuracy of the last conclusion. This contradicts the practices followed, with complete success, by the states of Tennessee and North Dakota, which recommend 7 in. and 4 in. of expansion, respectively.

Another point (*3*) is that the stresses at the various parts of the test specimen, in Stage 4, were of greater magnitude during the expansion cycle than during the contraction cycle. This result was attributed to the passive soil resistance of the backfill to expansion and to the fact that the active soil pressure actually helps contraction, even making the differences between the two actions more pronounced.

North Dakota

North Dakota has been constructing integral abutment bridges for more than 30 years. It is also the only state that has tried to eliminate the effect of passive soil pressure behind the abutments. Following is a brief summary of a field study (*10*) conducted by the North Dakota State Highway Department in November 1981.

The bridge studied had integral abutments, piers, concrete box girders, and a concrete deck. The bridge, 450 ft long, was made up of six 75-ft spans. Expansion joint material was placed between the back side of the abutment and the soil backfill, as shown in Figure 7, and compressible material (Type R Zerolite with compressive strength range of 5 to 10 $lb/in.^2$) was placed on the webs of the abutment piles, as shown in Figure 8.

The equation the North Dakota DOT uses to calculate temperature change, ΔT, is

$$\Delta T = T_1 - T_2 + (T_3 - T_1)/3 \qquad (1)$$

where

T_1 = air temperature at dawn on the hottest day,
T_2 = air temperature at dawn on the coldest day, and
T_3 = maximum air temperature on the hottest day.

This temperature change will result in the change in length, ΔL, of the bridge given by

$$\Delta L = L \alpha \Delta T \qquad (2)$$

where

L = original length of bridge and
α = coefficient of thermal expansion of the bridge deck material.

An interesting point noted by the North Dakota DOT was that, after 1 year, the gap had closed by 0.5 in. on the north abutment and 0.75 in. on the south abutment. The abutment-to-backfill gap is a questionable measure of abutment movement inasmuch as the soil backfill may move into the opening, depending on the ability of the expansion joint material to expand and fill the gap.

At the time of maximum change in bridge length, the south abutment had moved out by 1.96 in. and the north abutment had moved out only by 0.74 in. Therefore, North Dakota DOT concluded that the total change in the bridge length will not result in equal movement at each abutment.

Pile stresses were studied for the case in which the maximum abutment movement (1.96 in.) occurred. North Dakota DOT had concluded that the yielding took place in less than the top 1 ft of the piles, and at the highest stress point (top of the pile) the yielding occurred only on the outer one-fourth depth of each of the flanges. In other words, the stress at the top of the pile is sufficient to initiate a yield stress in the steel but not sufficient to cause the formation of a plastic hinge (*11*).

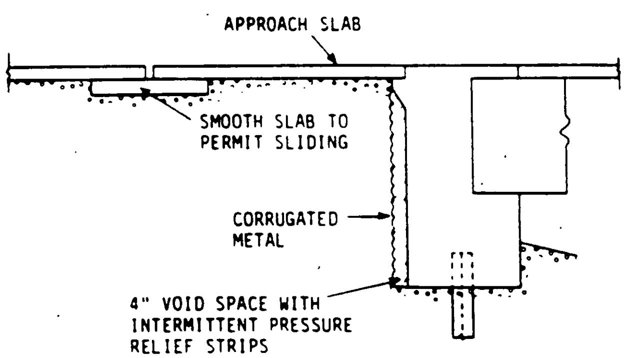

FIGURE 7 North Dakota integral abutment system with pressure relief strips.

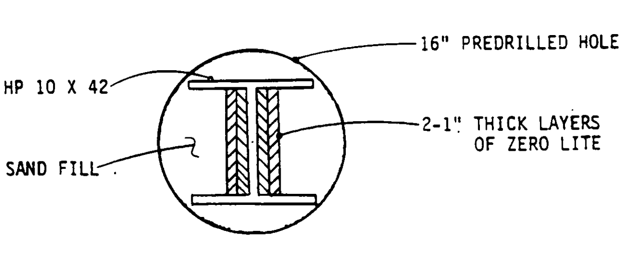

FIGURE 8 North Dakota abutment pile with compressible material on web.

Iowa

Iowa began building integral abutments on concrete bridges in 1964. One of the first bridges was constructed on Stange Road over the Squaw Creek in Ames (*12*). This prestressed beam bridge is about 230 ft long with no skew and is shown in Figure 4. Inspection of this bridge indicates no major cracks and apparent distress in the abutment walls, wingwalls, and beams caused by the thermal movement. The Iowa DOT reported (*12*) that inspection has been made yearly on 20 integral abutment bridges for about 5 years after their construction. Some of these bridges were skewed up to 23 degrees. The inspections were terminated because no stress or problems were found relating to the lack of expansion joints in the superstructure.

Iowa's designs are based on an allowable bending stress of 55 percent of yield plus a 30 percent overstress, because the loading is caused by temperature effects (*12*). The movement in the piles is found by a rigid joint frame analysis, which considers the relative stiffness of the superstructure and the piling. The piles are assumed to have an effective length of 10.5 ft; the soil resistance is not considered. Their analysis has shown that the pile deflection is about ⅜ in. (*12*).

CONCLUSION

Although the objective of this study was to investigate the methods of analysis and design details that have been implemented by the various DOTs in the United States to overcome the problems related to integral abutment bridges, it is apparent that this topic has not yet been resolved. Many research studies have been conducted recently, and work is continuing at Iowa State University. The findings of the recent research studies agree in most part with the findings presented. Some of the state DOTs have effectively solved most of the problems and have been successful in constructing long bridges with integral abutments.

One problem that no state, except North Dakota, has addressed deals with the effect of passive soil pressure that acts behind the abutments during the bridge deck expansion. Further study of their design approach is warranted.

Most states are using integral abutments in designing their bridges. The study shows that design practices followed by most state DOTs are too conservative, and much longer bridges could be constructed. However, some rational analysis is still necessary to make this design more acceptable.

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