

Performance of Jointless Bridges in Virginia

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Final Report VTRC 26-R50

Standard Title Page - Report on Federally Funded Project

1. Report No.: FHWA/VTRC 26-R50		2. Government Accession No.:		3. Recipient's Catalog No.:	
4. Title and Subtitle: Performance of Jointless Bridges in Virginia				5. Report Date: June 2026	
				6. Performing Organization Code:	
7. Author(s): Bernard L. Kassner, Ph.D., P.E. and Edward J. Hoppe, Ph.D., P.E.				8. Performing Organization Report No.: VTRC 26-R50	
9. Performing Organization and Address: Virginia Transportation Research Council 530 Edgemont Road Charlottesville, VA 22903				10. Work Unit No. (TRAIS):	
				11. Contract or Grant No.: 115767	
12. Sponsoring Agencies' Name and Address: Virginia Department of Transportation Federal Highway Administration 1221 E. Broad Street 400 North 8th Street, Room 750 Richmond, VA 23219 Richmond, VA 23219-4825				13. Type of Report and Period Covered: Final	
				14. Sponsoring Agency Code:	
15. Supplementary Notes: This is an SPR-B report					
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17 Key Words: Bridge Federal ID 4553, 20641, 24736, 25039, 25895, 25957, 25958, 25959, 26017, 26038, 26230, 26452, 26469, 26659, 27344, 27413, 27414, 27431, 27484, 27542, 27544, 27733, 28011, 28171, 28272, 28475, 28526, 28614, 28834, 29342, integral, semi-integral, abutment, deck slab extension, jointless, bridge, approach			18. Distribution Statement: No restrictions. This document is available to the public through NTIS, Springfield, VA 22161.		
19. Security Classif. (of this report): Unclassified		20. Security Classif. (of this page): Unclassified		21. No. of Pages: 37	22. Price:

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In Cooperation with the U.S. Department of Transportation
Federal Highway Administration

Virginia Transportation Research Council
(A partnership of the Virginia Department of Transportation
and the University of Virginia since 1948)

Charlottesville, Virginia

June 2026
VTRC 26-R50

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ABSTRACT

Since the early 1980s, the Virginia Department of Transportation (VDOT) has routinely developed and modified bridge abutment designs that eliminate the need for deck joints. The main objective is to decrease lifecycle maintenance costs and disruptions to the traveling public while constructing resilient structures across Virginia. VDOT's current policy is to make jointless bridges the primary design choice. A selection algorithm was developed to help select the appropriate abutment option.

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INTRODUCTION

The Virginia Department of Transportation (VDOT) is one of the leading highway agencies in the United States, focusing on the construction of jointless bridges. The preference for jointless bridges has been driven by the need to reduce maintenance expenditures and provide more resilient bridges. Leaking expansion joints on conventional bridges are a common cause of premature deterioration to beam ends, bearing assemblies, and substructures. The resulting damage typically requires substantial resources to rectify; performing the necessary rehabilitation or repair work can cause significant disruption to the traveling public. In addition, setting up and maintaining construction zones pose an additional traffic safety risk.

Although the origins of jointless bridge construction in Virginia can be traced back to the 1920s, a more systematic approach was developed and has been periodically updated by VDOT since the 1980s. Current design policy mandates joint elimination to the greatest extent possible to mitigate damage to structural elements and thus reduce overall lifecycle costs. Effectively, VDOT's policy is to make jointless bridges a primary design choice (Hoppe et al., 2016). Any new bridge design with a deck expansion joint is subject to the State Structure and Bridge Engineer's approval.

Abutment Design Hierarchy

VDOT developed the following design selection algorithm for bridge abutments, with the first option being the most preferred:

1. Full integral abutment.
2. Semi-integral abutment.
3. Conventional cantilever abutment with deck slab extension.
4. Virginia abutment.
5. Conventional abutment with expansion joints (subject to State Structure and Bridge Engineer's approval).

The full integral abutment type, also called "integral abutment" (Figure 1a), is characterized by bridge girders and deck cast monolithically with the integral backwall, which is cast on top of the footing. A moment relief hinge, consisting of a row of steel dowels connecting the footing and the backwall, is provided to reduce the bending moment in foundation piles.

Typically, a single row of steel H-piles is used, oriented along the weak axis bending and driven to a minimum of 25 feet to achieve pile tip fixity. However, prestressed concrete piles are also allowed.

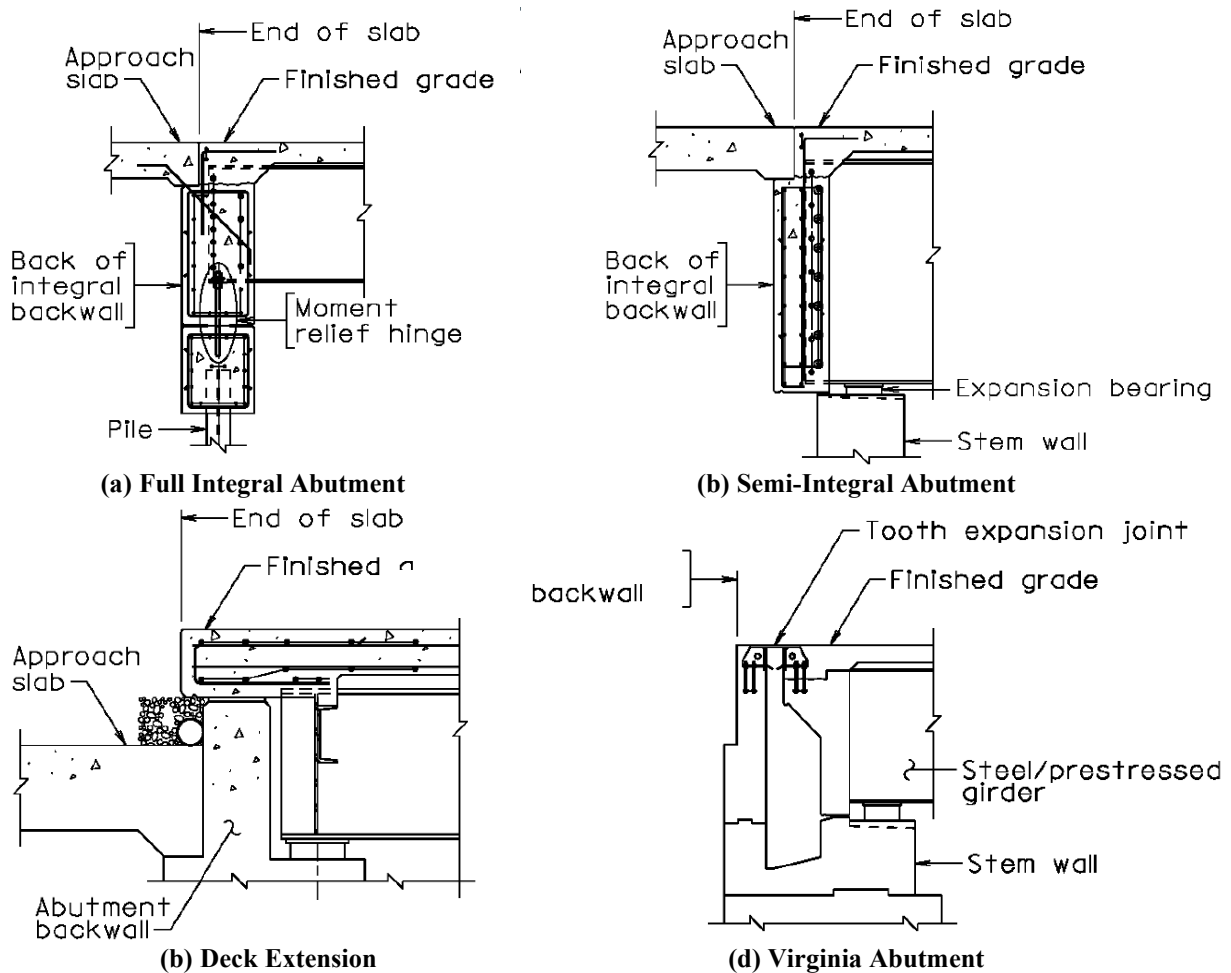


Figure 1. Jointless Abutment Types Used in Virginia (VDOT, 2024)

If the bridge parameters exceed the overall geometry and thermal movement constraints of the full integral abutments or if the foundation piles cannot be driven sufficiently deep, the next available option is a semi-integral abutment (Figure 1b). This type of design also involves the girders and deck cast monolithically with the integral backwall. However, the backwall can move relative to the stem wall of the abutment, and bearings support the girders to accommodate expansion and contraction. Thus, the superstructure and substructure are effectively independent of each other.

Current VDOT (2024) guidelines specify the use of elasticized expanded polystyrene (EPS) at the back of full integral and semi-integral abutments. This elastic inclusion, in conjunction with granular backfill material, is intended to absorb thermally induced superstructure displacements and reduce bridge approach settlement. The material is also intended to attenuate lateral earth pressure acting on the back of the abutment. VDOT uses

relatively conservative passive earth pressure coefficients, K_p values of 4 and 12 for designs with and without the EPS elastic inclusion, respectively.

The next design alternative involves a conventional cantilever abutment combined with a deck slab extension over the top of the abutment backwall (Figure 1c). Structurally, this alternative is not an integral design because the beams are not embedded in the backwall, but the design effectively provides a jointless bridge deck. With proper subsurface drainage details, water is directed away from the joint area behind the backwall. This type of design lends itself to retrofitting existing conventional bridges when deck replacement is needed.

The Virginia abutment (Figure 1d) is used when the bridge length, skew, and calculated thermal range of movement exceed the allowable limits for the aforementioned abutment types. This design provides a secondary backwall behind a parallel integral backwall. The two walls are separated by a wide, epoxy-coated service trough that collects drainage from the tooth expansion joint above it. The trough width allows sufficient maintenance access for debris cleanout. The secondary backwall is stationary, eliminating soil-structure interaction resulting from thermal effects. Although the Virginia abutment is relatively expensive to construct compared with other integral abutments, its design configuration is versatile, allowing it to be used with any bridge geometry.

PURPOSE AND SCOPE

The purpose of this study was to provide VDOT's Structure and Bridge Division feedback on the in-service performance of bridges that are near or extend beyond VDOT's limits for length and skew for various types of jointless abutments. In addition, this study identified potential areas of current design guidelines that may require further refinement. To accomplish these goals, the scope of this study consisted of visual field observations of representative structures constructed since the early 1980s, including the condition of abutment backwalls, wingwalls, and bridge approaches. The researchers also examined the archival bridge safety inspection reports to assess the reported maintenance issues.

METHODS

Overview

The following tasks were performed to achieve the study objectives:

1. Characterize VDOT's inventory of jointless bridges.
2. Conduct site visits at selected bridges.
3. Analyze information collected from the field.
4. Examine archival bridge safety inspection reports.

Characterize VDOT’s Inventory of Jointless Bridges

To characterize VDOT’s jointless bridge population, the researchers obtained the agency’s entire structure inventory database as of August 2019 in the form of an Excel spreadsheet. As a first step in selecting the jointless structures from the entire inventory, the field for State Item 19 in the *VDOT Bridge Inventory Coding Guide* (VDOT, 2019), and specifically, *ABUT_B_TYPE*, was set to “8” for integral abutments. After resetting the field for Abutment B, the researchers set the same filter for *ABUT_A_TYPE*. This designation was assumed to apply only for full integral abutments. Separately, the field for State Item 173—*Special Structure Code* in the same manual was set to “11” for integral backwalls, also known as semi-integral abutments. Unfortunately, the *Coding Manual* lacked unique designations for bridges with deck extensions and for Virginia abutments (VDOT, 2019). Therefore, VDOT’s Central Office Structure and Bridge Division provided a supplemental listing of structures, including these two design types.

After gathering the previous information, the researchers identified additional jointless bridge candidates by reviewing plan drawings of structures with conventional abutments (see the *Examine Archival Bridge Safety Inspection Reports* section). These additional bridges were not identified during the initial database search.

Having established a listing of jointless bridge candidates, subsequent examination of plan drawings for those structures confirmed the abutment type for both abutments and the following additional details for this study:

- Year designed.
- Continuous span length.
- Superstructure material.
- Skew angle.
- Height of fill above existing soil.
- Approach type.
- Backfill material.
- Drainage.
- Presence of EPS at the backwall.

The aforementioned database also provided the following information:

- Average daily traffic.
- Average daily truck traffic.
- Route classification—State Item 185, *Virginia Highway System Code*, in the *Coding Manual* (VDOT, 2019).

Using the previous information, the researchers categorized the jointless bridge inventory according to the following factors:

- Abutment type.
- Continuous span length.
- Skew angle.
- Age.
- Route classification.
- Average daily traffic.

Observe Conditions at Selected Bridges

With approximately 400 jointless bridges in VDOT's inventory, it was not feasible to visit all potential candidates. Therefore, based on the results of the first task and subsequent work with the technical review panel, the researchers selected a representative set of bridges. However, the selection prioritized those structures near or beyond the current limits for length, skew, or both, but not necessarily bridges using the latest structural details. Also, given that the Virginia abutment is near the bottom of the abutment selection hierarchy and constitutes a relatively small percentage of VDOT's bridge inventory, the focus of the site visits was directed at the other types of jointless bridges.

The researchers visited 23 bridges with jointless abutments. At the bridge sites, the researchers documented any visible signs of deterioration of the abutments, the deck-approach interface, and the approaches. The temperature during the site visits ranged from 27°F to 70°F. Continuous span lengths ranged from 110 to 815 feet, and the skew angle ranged from 0° to 45°. These bridges were constructed between 1996 and 2019. The average daily traffic ranged from 145 to 17,341. The abutment type, presence of EPS, drainage, select backfill material, and approach slab type varied across the subset. Details of the bridge sites that the researchers visited are listed in Table 1, and the geographic locations of these sites are displayed in Figure 2.

Each site was assessed through visual observations, with qualitative general condition ratings of excellent, good, fair, and poor assigned to the face of the abutment backwalls, wingwalls, and roadway approaches, including the sleeper pad-pavement interface. These qualitative ratings were based on the degree of cracking, disintegration, leaking, staining, or past repairs. The researchers documented the conditions with photographs and used tape measurements to record gaps and changes in elevation at the deck-approach and shoulder-approach slab interfaces.

Analyze Information Collected from the Field

Site-specific data were processed to determine if design type relates to field performance. Factors examined in the analysis included the following:

- Abutment type.
- Superstructure material.
- Total length affecting abutment displacement.
- Degree of skew.
- Bridge age.
- Use of a concrete slab at the approach.
- Geotechnical considerations:
 - Backfill material.
 - Drainage installation.
 - Presence of EPS.
 - Height of fill above the existing soil.

Table 1. List of Selected Bridges with Jointless Abutments and Relevant Design Characteristics

Federal ID No.	Route Name	Feature Intersected	Year Built	Super-structure Material	Contin-uous Length (ft)	Skew (°)	Abut-ment Type ^a	EPS Thick-ness (in)	Drain-age ^b	Select Backfill	Slab Type	Average Daily Traffic	Road Classi-fication
4553	US 58 (EBL), Route 221	Little Reed Island Creek	2010	steel	146	30	SI	none	none	no	surface	7,407	Primary
20641	Harpersville Road	Interstate 64	2000	steel	240	27	SI	none	pipe	no	surface	9,389	Urban
24736	Snider Branch Road	Middle Fork Holston River	1996	concrete	113	30	FI	none	none	no	none	271	Secondary
25039	Fiery Run Road	Norfolk-Southern Railroad	1997	steel	152	30	FI	none	pipe	no	none	3,699	Secondary
25895	US 58 (EBL)	Sandy Creek	2005	steel	314	0	FI	none	pipe	yes	none	5,197	Primary
25957	Dogwood Drive	Route 29	2005	steel	287	20	FI	none	pipe	no	surface	286	Secondary
25958	Izaak Walton Road	Route 29	2004	steel	275	14	FI	none	pipe	no	surface	668	Secondary
25959	Bobwhite Road	Route 29	2004	steel	282	13	FI	none	pipe	no	surface	585	Secondary
26017	EBL Bowling Green Byp.	Route 301 / Route 2	2002	steel	194	17	SI	none	pipe	no	surface	3,850	Primary
26038	Lee-Jackson Highway	Reed Creek	1999	steel	125	30	FI	none	pipe	no	surface	2,342	Primary
26230	Selma Low Moor Road	CSX Railroad	2010	steel	306	0	SI	10	none	yes	none	1,843	Secondary
26452	Nelson Hill Road	CSX Railroad	2001	steel	256	23	SI	none	pipe	no	surface	2,300	Secondary
26469	Medical Campus Blvd.	Route 37	2001	steel	214	26	SI	none	pipe	no	none	17,341	Primary
26659	Main Street	Jackson River	2000	steel	331	0	SI	6	board	yes	none	7,628	Primary
27344	Newland Road	Cat Point Creek	2008	concrete	815	0	SI	10	none	yes	none	1,773	Secondary
27413	Route 340	Jeremiahs Run	2009	steel	446	0	SI	23	pipe	yes	buried	4,389	Primary
27414	Route 340	Overall Run	2008	steel	300	0	DE	none	pipe	yes	buried	4,279	Primary
27431	Jefferson Park Avenue	Norfolk-Southern Railroad	2012	steel	94	21	SI	10	weep	yes	surface	8,810	Urban
27484	Route 712	Interstate 81	2011	steel	306	0	FI	10–17	pipe	yes	none	444	Secondary
27542	Farris Mine Road	Big Reed Island Creek	2006	steel	243	35	DE	none	weep	no	none	141	Secondary
27544	Stone Spring Road	Interstate 81	2012	steel	341	20	SI	10	pipe	no	none	211	Urban
27733	Rest Church Road	Interstate 81	2003	steel	306	0	DE	none	weep	yes	none	5,600	Secondary
28011	John Tyler Memorial Pkwy.	Chickahominy River	2009	concrete	600	0	SI	22	none	yes	surface	3,307	Primary
28171	Route 42	Jennings Branch	2004	steel	160	0	SI	12	pipe	yes	none	1,755	Primary
28272	Farris Mine Road	Big Reed Island Creek	2013	steel	226	30	SI	14	geo	yes	none	141	Secondary
28475	Potts Creek Road	Blue Spring Run	2006	steel	110	45	SI	15	pipe	yes	none	1,304	Primary
28526	Jefferson Davis Highway	CSX Railroad	2015	steel	178	48	DE	none	pipe	no	buried	17,273	Primary
28614	Prospect Hill Road	Nomini Creek	2014	concrete	449	0	FI	16	pipe	yes	none	508	Secondary
28834	Midland Trail	C&O Railroad	2008	steel	200	42	DE	none	pipe	yes	none	2,098	Primary
29342	Natural Well Road	Jackson River	2019	steel	254	0	FI/SI	24 ^c	geo	yes	none	224	Secondary

EBL = eastbound lane; EPS = expanded polystyrene. ^aDE = deck extension; FI = full integral; SI = semi-integral. ^b Drainage system: board = 4-inch drainage board with weepholes; geo = geocomposite wall drain; pipe = perforated pipe parallel to footing; weep = weepholes. ^c The full integral abutment at Bridge 29342 does not have EPS, but the semi-integral abutment does.

Examine Archival Bridge Safety Inspection Reports

For structures with evidence of deterioration and/or maintenance within the approaches, the researchers gathered archival bridge safety inspection reports to determine when deterioration began and how the condition changed over time. The data gathered from these reports included:

- Substructure and deck condition ratings and condition state quantities.
- Narrative information reflecting the condition of the following:
 - Bridge deck.
 - Approach.
 - Bearings.
 - Abutments.
 - Wingwalls.

One question that arose was whether the settlement observed at some jointless abutment-approach interfaces was substantially different from that often found at conventional abutments. The researchers compared the settlement history of a set of jointless abutment bridges with their conventional counterparts. This subtask involved identifying comparable structures having similar parameters, including:

- Era.
- Presence of approach slab.
- Backfill material.
- Drainage installation.
- Geographic setting.

State of Best Practices for Jointless Abutments

In the 1980s, VDOT in cooperation with the Virginia Transportation Research Council (VTRC) embarked on a systematic development of various integral bridge design options to reduce bridge lifecycle costs. Zuk (1981) mathematically analyzed the potential use of continuous jointless bridges allowing for flexible pier design and high cambering of the superstructure to accommodate thermal expansion. Hoppe and Gomez (1996) reported on the field performance of an instrumented semi-integral bridge on Interstate 81 in the Staunton District. The study pointed to a significant settlement of the abutment backfill material, ultimately leading to VDOT's adoption of the select backfill detail (Figure 3).

Prior to 2006, VDOT practice called for a loosely compacted, "porous backfill" behind the abutment. The intent of specifying loose placement was to avoid exerting excessive lateral loads on the backwall. This practice was revised in 2006 by mandating compacted granular fill with a California Bearing Ratio value of 30. The objective was to reduce the approach settlement.

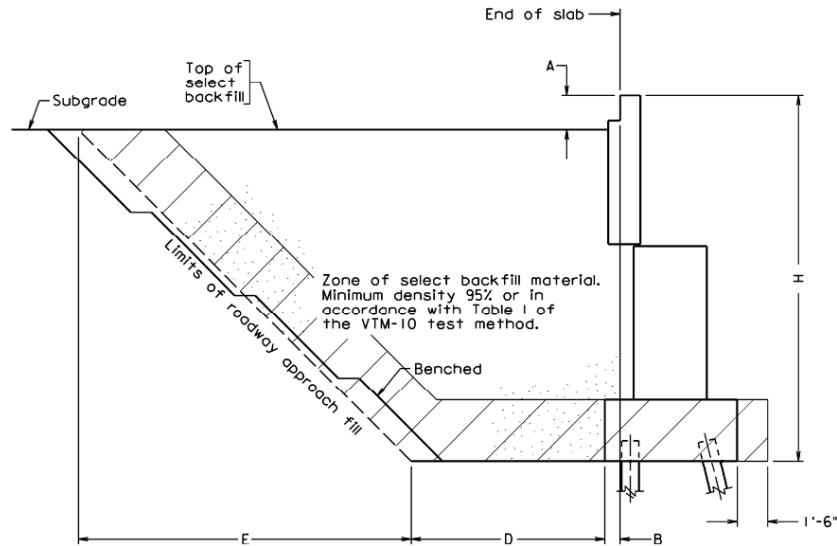


Figure 3. Select Backfill Material Behind Abutment (VDOT, 2024). VTM = Virginia Test Method.

The laboratory study by Arsoy et al. (2002) resulted in revising the moment relief hinge construction detail at full integral abutments. Field instrumentation studies of a semi-integral bridge with the elasticized EPS installed at the integral backwall and of a highly skewed semi-integral bridge provided additional performance feedback (Hoppe, 2005; Hoppe and Eichenthal, 2012). Further studies by Virginia Tech researchers included the analysis of the thermal response of integral bridges with mechanically stabilized earth wall abutments (Arenas et al., 2013). Mokwa and Duncan (2000) provided extensive guidance on calculating pile stresses at full integral abutments, accounting for the significant lateral resistance of pile caps.

Circa 2004, VDOT began using elasticized EPS. This material is intended to reduce the lateral earth pressures at abutments because the superstructure expands longitudinally against the adjoining roadway embankment. In addition, the elastic nature of the EPS material helps to accommodate bridge cyclic movements, reducing settlement and deterioration of the adjoining roadway approach.

VDOT’s comprehensive design guidelines for jointless bridges are compiled in the *Manual of the Structure and Bridge Division, Part 2*, Chapter 17 (VDOT, 2024). These guidelines include examples of design calculations and structural details developed by VDOT engineers. Table 2 presents current length and skew limits for various jointless bridges.

Table 2. Current VDOT Limits for Jointless Abutment Designs in Virginia

Abutment Type	Steel		Concrete		Total Movement at Abutment (in)
	Total Length (ft)	Skew (deg)	Total Length (ft)	Skew (deg)	
Full Integral	300	0	500	0	1.50
	150	30	250	30	1.50
Semi-Integral	450	30	750	30	2.25
Deck Slab Extension (straight beams)	450	45	750	45	2.25
Deck Slab Extension (curved beams)	300	30	N/A	N/A	1.50
Virginia Abutment	no limit	no limit	no limit	no limit	N/A

N/A = not applicable.

VDOT’s guidelines are generally in line with the practices adopted by other state departments of transportation, as the recent National Cooperative Highway Research Program Scan 19-01 report (DeRuyver et al., 2020). Table 3 details the jointless bridge span length and skew angle design limits adopted in various states. This same scan report noted that two of the more common challenges with jointless abutments are approach settlement and pavement damage (Table 4). Likewise, Dahlberg et al. (2023) indicated that deck cracking can be a problem on wide integral bridges.

Table 3. Maximum Limits for Span Length and Skew for a Given Superstructure Material for Various State Departments of Transportation, Based on Data from DeRuyver et al. (2020)

State	Common Abutment Type	Span Length (ft)		Skew (°)
		Steel	Concrete	
California	FI	NG	400	20
Iowa ^a	FI (preferred) and SI	400	575	0
		300	425	45
Louisiana	FI	NG	NG	45
Maine ^a	FI, SI, DE	300	600	20
Massachusetts ^a	FI	NL	NL	NL
Minnesota ^b	FI (preferred) and SI	300	300	20
		100	100	45
Pennsylvania ^c	FI	390	590	20
		130	130	30
		90	90	45
South Dakota ^d	FI	350	700	30
Tennessee	FI	NL	NL	NL
Utah ^e	FI and SI	NL	NL	60
Washington ^f	SI	300	450	45

DE = deck extension; FI = full integral; NG = no guidance given; NL = no set limit or thermal displacements dictates maximum span lengths; SI = semi-integral. ^a Buried slabs used or preferred. ^b Lengths listed are for integral abutments; the maximum skew limit for SI abutments with wingwalls parallel to the roadway is 30°. ^c Length limit for 20° skew applies to a single span greater than 130 feet or any structure with multiple spans; the length limits for the other skews listed apply to single-span structures. ^d Standard skew, although 35° skews do occur, with rarer instances of 45° skews. ^e Guidance recommends that skews be no more than 30°; skews greater than 45° require approval. ^f Current policy provides only for SI abutment; preferred maximum skew is 30°, but skews up to 45° may be permitted with approval.

Table 4. Common Jointless Abutment Issues Encountered by State Departments of Transportation, Based on Information from DeRuyver et al. (2020)

Issue	State								
	CA	IA	ME	MN	PA	SD	TN	UT	WA
Deck Cracking		X	X	X					
Approach Slab	X					X	X		
Pavement Damage	X	X	X	X				X ^a	
Approach Settlement		X	X	X	X	X	X		X
Undermining		X		X	X	X			

^a Proper sizing of expansion joint at the sleeper pad-pavement interface resolved the issue.

Some of the recent changes in bridge design practice reported by other state departments of transportation include the following (DeRuyver et al., 2020):

- California has phased out sleeper pads.
- Maine now requires approach slabs for all jointless bridges.

- Iowa has extended the corbel ledge for supporting the approach slab at the backwall to 15 inches and added granular backfill with drainage.
- Minnesota uses a guide lug for skew angles greater than 30° to prevent excessive lateral movement.
- South Dakota ended the practice of constructing a 6-inch void behind the backwall and no longer uses mechanically stabilized-earth reinforcement in conjunction with jointless abutments.
- Washington requires the use of concrete approach slabs at all bridges.

RESULTS

Characterization of VDOT’s Inventory of Jointless Bridges

Figures 4 through 6 show the distribution of VDOT’s jointless bridge inventory by abutment type and superstructure material type.

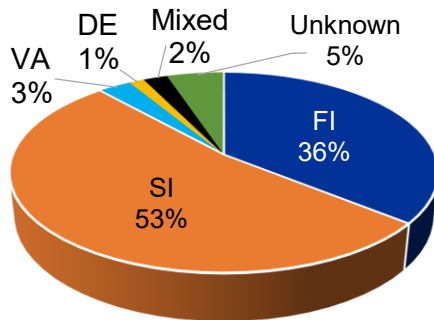


Figure 4. Abutment Types at All Jointless Structures. DE = deck extension; FI = full integral; SI = semi-integral; VA = Virginia.

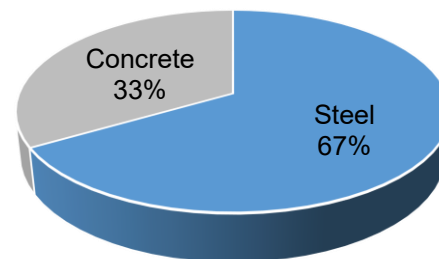
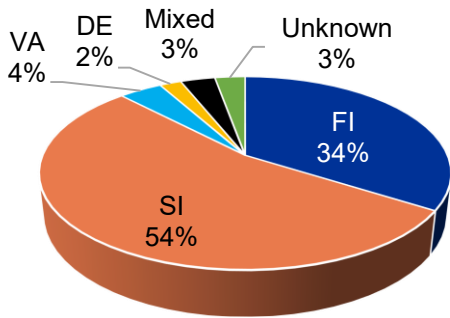
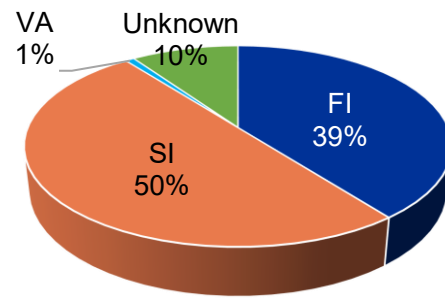


Figure 5. Superstructure Materials at All Jointless Structures



(a)



(b)

Figure 6. Abutment Types at (a) Steel and (b) Concrete Superstructures. Percentages are relative to the total inventory of bridges with jointless abutments. DE = deck extension; FI = full integral; SI = semi-integral; VA = Virginia.

Approximately two-thirds of Virginia’s jointless structures are bridges constructed with steel superstructures. The predominant abutment type is semi-integral. The “unknown” category represents bridges with incomplete documentation at the time of the study.

Figure 7 shows the temporal distribution of various types of jointless abutments. The period from 1997 to 2006 represents the peak construction activity to date, with mostly semi-integral abutments constructed at that time. Approximately 40% of the current inventory was constructed after 2006.

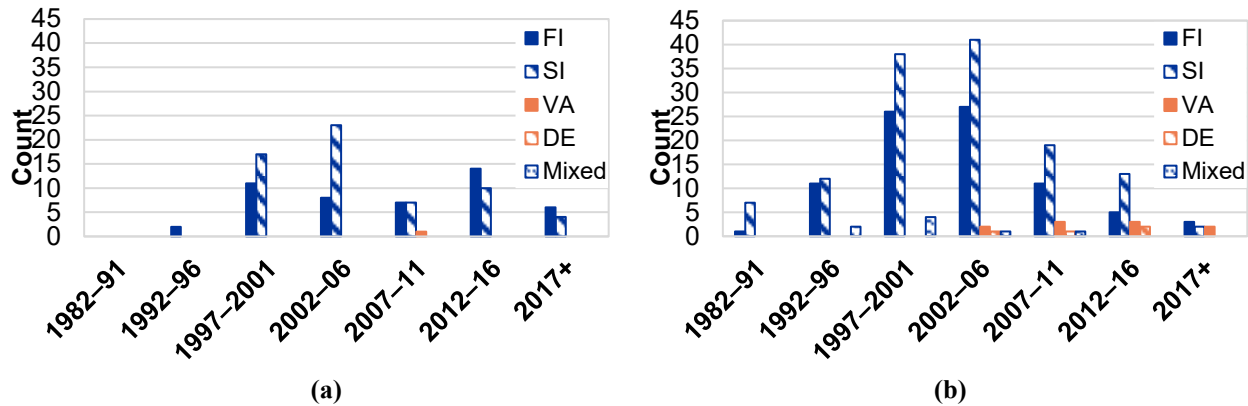


Figure 7. Count of Jointless Abutments Constructed Over Time for (a) Steel and (b) Concrete Superstructures. DE = deck extension; FI = full integral; SI = semi-integral; VA = Virginia.

Figure 8 shows the route classifications associated with various jointless abutment types. These structures, mainly semi-integral, were constructed predominantly on the primary road system, with a sizable percentage also placed on secondary roads.

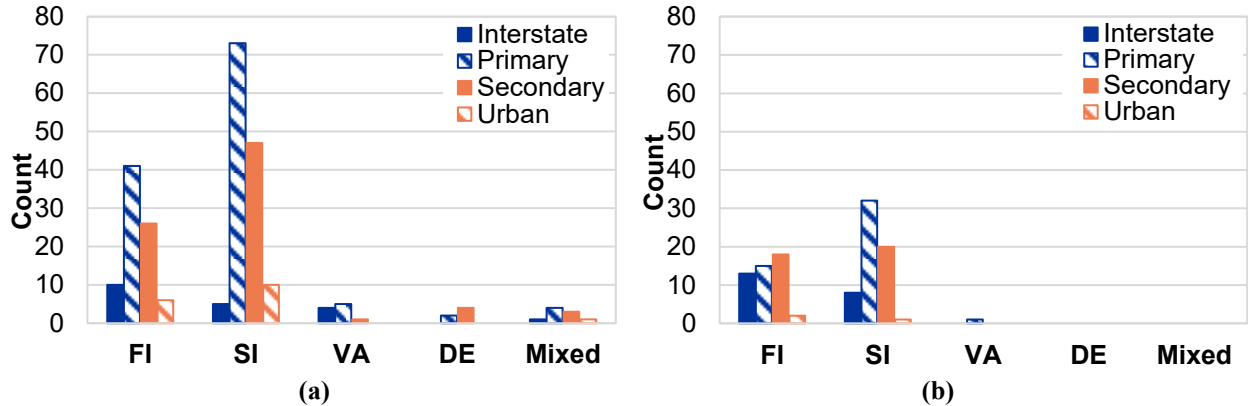


Figure 8. Count of Jointless Abutments on Given Route Classification for (a) Steel and (b) Concrete Bridges. DE = deck extension; FI = full integral; SI = semi-integral; VA = Virginia.

Figure 9 shows the entire inventory of VDOT’s jointless bridges, represented in terms of continuous span length and superstructure skew angle relative to the current design limits for various abutment types. A few structures (13 steel full integral, one steel semi-integral, one concrete semi-integral, one deck extension type of abutment) exceeded the specified span, skew limit, or both. The inventory also included four abutment types that exceeded the technical guidance limits, in which “mixed” represents any set of abutments in a bridge where the design types of the two abutments are different, jointless, or are otherwise. All the “mixed” cases involve steel superstructures. Two were supported by a combination of one full integral and one conventional abutment, with a large skew (42°), and the other with a combination of a moderate

skew and span length that placed the structure outside the limits for steel full integral-type bridges. The other two cases involved both full integral and semi-integral abutments, one with a high skew (37°) and the other with a skew-span length combination that exceeded the limits for the steel full integral-type of abutment.

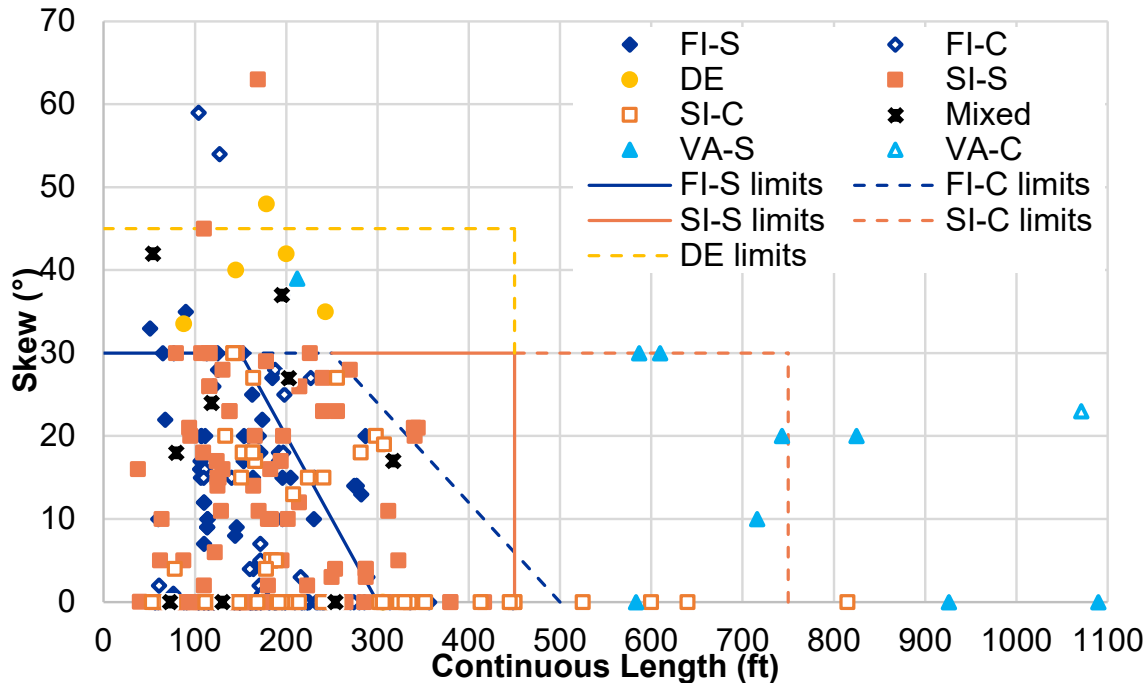


Figure 9. Entire Inventory of VDOT’s Jointless Abutments. C = concrete superstructure; DE = deck extension; FI = full integral; S = steel superstructure; SI = semi-integral; VA = Virginia.

Figure 10 shows that many jointless bridges in Virginia have concrete approach slabs. The top surface of these approach slabs is typically constructed at the same elevation as the roadway surface (at-grade approach slab). In contrast, several bridges feature buried concrete approach slabs but were constructed as experimental trials. Their number is limited because of the lack of formally established design standards. VDOT’s inventory also contains a significant quantity of bridges without concrete approach slabs. These bridges are typically constructed on low-volume secondary roads.

Figure 11 shows the number of jointless abutments equipped with the elasticized EPS installed at the backwall. Their percentage of the total population is relatively small because VDOT did not make the use of EPS a part of its formal policy until 2003.

Figure 12a shows the count of structures with a drainage system installed behind the abutment, and Figure 12b shows how many abutments were backfilled using “select material” versus those abutments backfilled with a material specified only as “porous fill.” Overall, the bridges are split evenly in terms of those with and without a clearly identified drainage system, and nearly three-fourths of all abutments were constructed with porous backfill material. The select backfill detail was not instituted as a statewide practice until 2006.

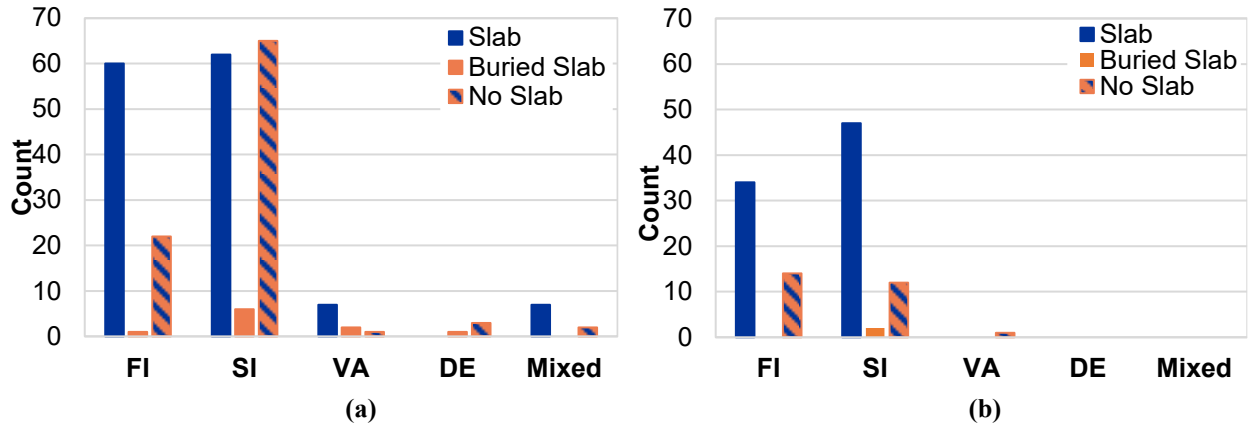


Figure 10. Count of Jointless Abutments in VDOT's Entire Inventory, by Approach Type for (a) Steel and (b) Concrete Superstructures. DE = deck extension; FI = full integral; SI = semi-integral; VA = Virginia.

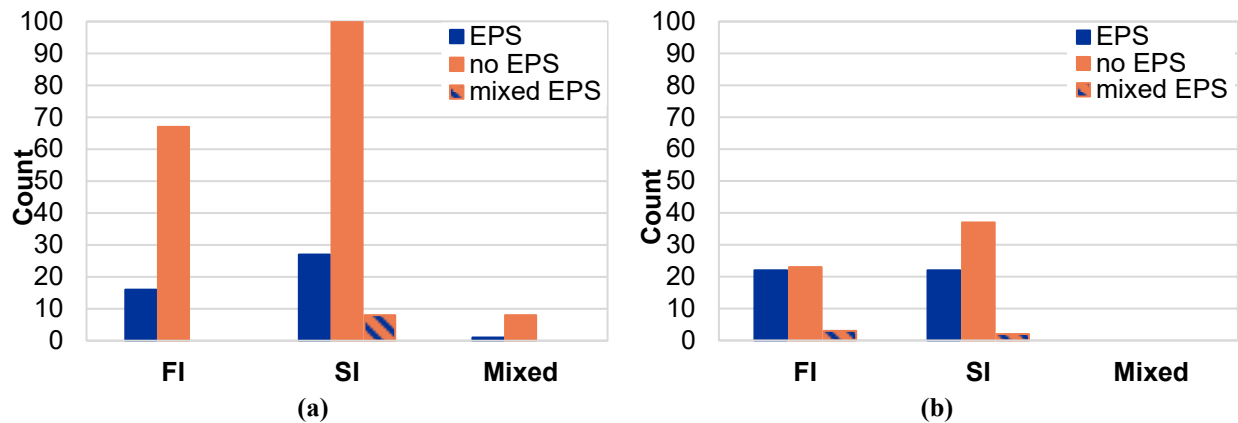


Figure 11. Count of Jointless Abutments in VDOT's Entire Inventory, by Presence of EPS: (a) Steel Superstructures; (b) Concrete Superstructures. EPS = expanded polystyrene; FI = full integral; SI = semi-integral.

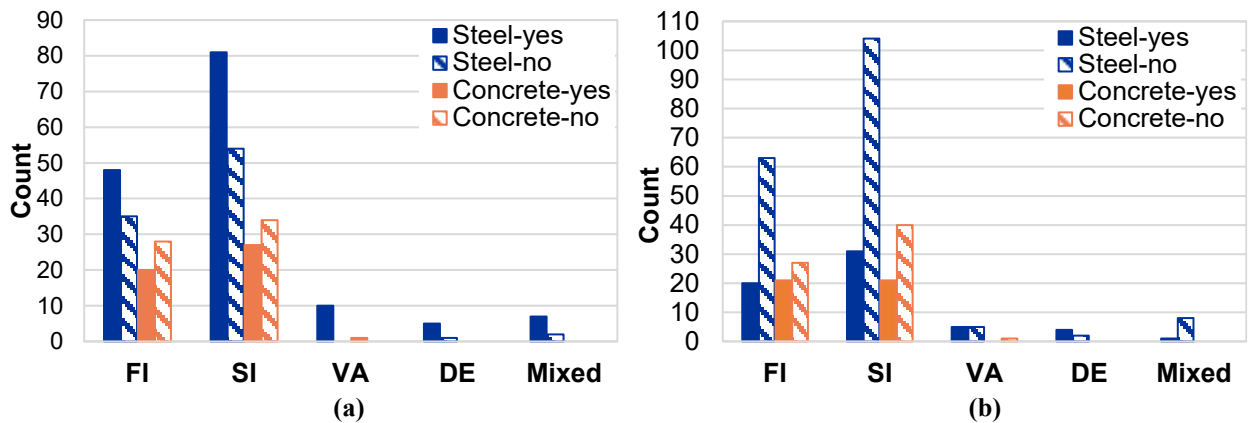


Figure 12. Count of Jointless Abutments and the Use of (a) a Drainage System and (b) Select Backfill for Steel and Concrete Bridges. DE = deck extension; FI = full integral; SI = semi-integral; VA = Virginia.

Observed Conditions at Selected Bridges

Beam Ends and Bearings

Overall, the beam ends and bearings for jointless bridges in this survey had performed well, with little signs of distress. However, some exceptions were found among the various abutment types, as the following sections discuss.

Full Integral Abutments

Site visits to full integral bridges revealed no evidence of structural distress. It appears that full integral abutments supporting steel superstructures have been performing well. All the structures visited were at or exceeded the current span or skew design limits. Visual observations of the integral backwall, including the hinge detail, did not indicate any apparent structural distress. Also, no significant cracking of the bridge deck was detected.

The only visually noticeable feature detected at many full integral bridges with uncoated weathering steel superstructure was the rust staining of concrete at the locations of contact (Figure 13). Some of the observed concrete staining was extensive.



Figure 13. Photographs of Rust Staining at Full Integral Abutments: (a) Bridge 25957; (b) Bridge 25959

It should be noted that the rust staining of concrete was observed not only at full integral bridges but also on other jointless bridge types. Moreover, it was frequently observed not only at the exterior beams, which are typically exposed to rainfall, but also near the interior beams.

Semi-Integral Abutments

In general, site visits to semi-integral bridges revealed good performance of these structures, although some recurring instances of maintenance-related problems persisted. One common occurrence in a skewed semi-integral bridge of older design is noticeable superstructure rotation in the horizontal plane, even at a relatively small skew angle. This rotation typically results in anchor bolts locking up against the sole plates (Figure 14).

At the acute end of the integral backwall, the rotation can cause widening of the gap at the vertical interface with the adjoining wingwall, resulting in the spill of crushed stone backfill material (Figure 15).



Figure 14. Photographs of Bent Anchor Bolt on Skewed Semi-Integral Abutment: (a) Bridge 20641; (b) Bridge 27544



Figure 15. Photograph of Backfill Spill at Semi-Integral Abutment (Bridge 28475)

Horizontal rotation of the superstructure and the resulting lateral interaction with the abutment can also lead to concrete cracking at the wingwall interface due to an increase in earth pressure (Figure 16).

In addition, anecdotal evidence of sporadic construction-related problems at semi-integral bridges was received from the Staunton District (Park Thompson, personal communication). The problem was traced to the integral backwall concrete formwork being attached solidly to or physically interfering with the abutment stem. After concrete placement, the precipitous drop in air temperature caused the steel beams to thermally contract and pull out of the fresh concrete. This problem can be averted through adequate construction supervision.



Figure 16. Photograph of Wingwall Crack at Semi-Integral Abutment with 45-Degree Skew (Bridge 28475)

Combined Full and Semi-Integral Bridges

One of the bridges, Structure No. 29342 in Staunton District, is an example of a jointless bridge constructed with one full integral and one semi-integral abutment (Figure 17). This design choice was imposed by limited right-of-way at the site.



Figure 17. Photograph of Structure No. 29342—Route 638 over Jackson River in Alleghany County

The initial design of Bridge 29342 included a pile-supported full integral abutment. A design change was made during construction, resulting in the abutment footing being cast directly on the bedrock stratum. With the expansion bearings installed over the pier, the bridge is effectively fixed longitudinally at the full integral abutment (A) and thermally expanding along its entire span length toward the semi-integral abutment (B), as Figure 18 shows.



Figure 18. Photograph of In-Service Expansion and Rotation Diagram at Structure No. 29342. Curved and straight arrows indicate the direction of positive and negative rotation and displacement, respectively. A = Abutment A (full integral); B = Abutment B (semi-integral); E = expansion bearing; F = fixed bearing.

Because of its unique design, VTRC personnel have been monitoring this bridge using onsite instrumentation at the request of the Staunton District Bridge Office. Field data collected during a 4-year monitoring period (since construction) do not indicate structural distress manifested by excessive stress, lateral displacement, rotation, or concrete cracking and spalling. As a part of that data collection, the distance between backwalls was measured using two laser sensors. Figure 19 shows the plot of overall backwall-to-backwall distance measurements, together with the corresponding temperatures, monitored during the 4-year period.

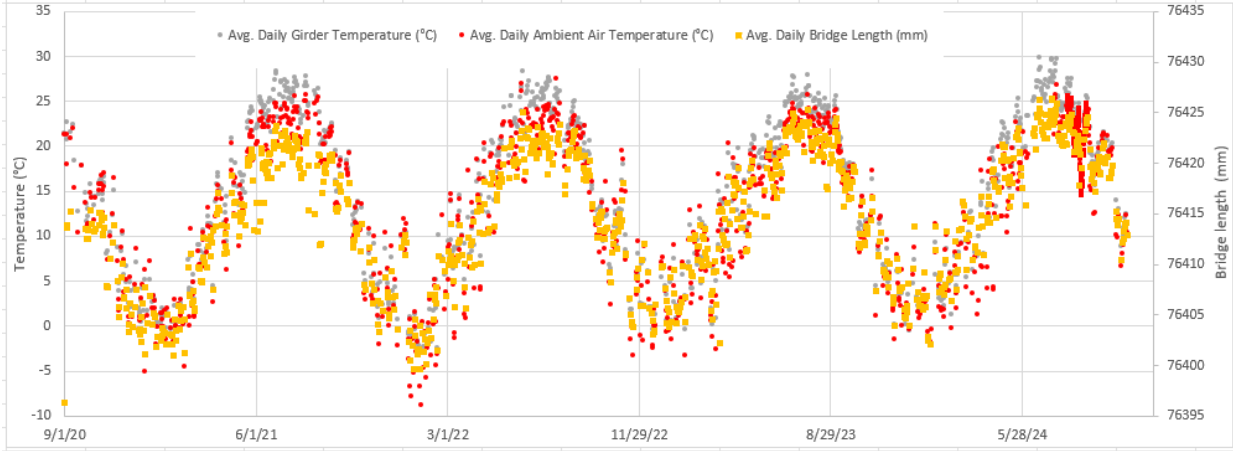


Figure 19. Backwall-to-Backwall Distance and Temperature at Structure No. 29342

As expected, the seasonal pattern of displacements is sinusoidal. A similar, diurnal pattern of thermally induced bridge displacements was also observed. The lateral displacement range is approximately 25 mm (1 inch), with a corresponding ambient air temperature range of 35°C (approximately 63°F) during the 4-year monitoring period. The entire range of movement was observed at Abutment B, whereas Abutment A remained stationary. Additional displacement sensors installed at the bearing locations over the pier and at the semi-integral abutment confirmed this magnitude of lateral movement over time.

Deck Extension

Site visits indicate that bridges with deck extensions are generally performing well. No signs of structural distress were observed. The only noticeable maintenance issue evident on some deck extensions is the leakage between the underside of the slab and the top of the abutment backwall, resulting in the drainage runoff spilling on the beam seats and bearings (Figure 20). All four deck extension bridges in this sample set had steel superstructures and were constructed between 2003 and 2008. None of these structures had the most recent VDOT drainage details.



Figure 20. Photographs of Leaking Joint at Deck Extension Down the Backwalls for Bridge 27733: (a) Abutment A; (b) Abutment B

Backwalls

Overall, integral backwalls in the jointless bridges surveyed in this study performed well, with 93% of them classified as either excellent or good condition (Figure 21). All the full integral and semi-integral backwalls supporting concrete superstructures were found in excellent condition, whereas approximately one-half of the full integral and semi-integral types supporting steel superstructures had that same rating. Of the three abutment types supporting steel beams, deck extension had the lowest performance, with only 25% in excellent visual condition, and another 63% in good condition. Generally, the best performing backwalls tended to be those at bridges with either small skews or with short span lengths (Figure 22).

The two bridges with “fair” backwalls, 26452 and 27733, were constructed in 2001 and 2003, respectively. The results in Figure 22 may not reflect the more recent construction, because these two bridges were built during a time when a contractor was permitted to use high early-strength concrete with up to 800 lb/yd³ of cementitious material. VDOT no longer allows that option for two reasons: the high cementitious content leads to substantial moisture-related shrinkage and larger temperature differentials inside the structure. Both of these effects can cause cracking that extends below the concrete surface. Also, in 2015, VDOT implemented the comprehensive use of a newly developed low-shrinkage A4 modified concrete mix to limit cracking in bridge decks, which has been reported to perform satisfactorily (Park Thompson, personal communication).



Figure 21. Count of Backwalls Observed with a Given General Condition, for a Given Abutment-Superstructure Type. C = concrete superstructure; DE = deck extension; FI = full integral; S = steel superstructure; SI = semi-integral.



Figure 22. Visual Condition of Surveyed Abutment Backwalls for Given Continuous Span Length and Skew. C = concrete superstructure; DE = deck extension; FI = full integral; S = steel superstructure; SI = semi-integral.

Roadway Approaches

Numerous signs of approach-pavement distress are present at many of Virginia’s jointless bridges. Pavement distress is a common problem related to soil-structure interaction associated with the thermally induced lateral expansion and contraction of the bridge superstructure, which interacts with the adjoining roadway approach embankment. Figure 23 shows examples of qualitative ratings. Note that the relatively long continuous span length of 815 feet, lack of concrete approach slabs (included in the original design but not constructed), and soft subsurface soils exacerbated the “poor” condition of pavement approach of the example (Bridge 27344).

Figure 24 provides a summary of visual observations related to the pavement approach surface condition observed during field inspections. Although the continuous span length and skew angle appear to be significant variables, other factors clearly influence the overall performance of bridge roadway approaches.

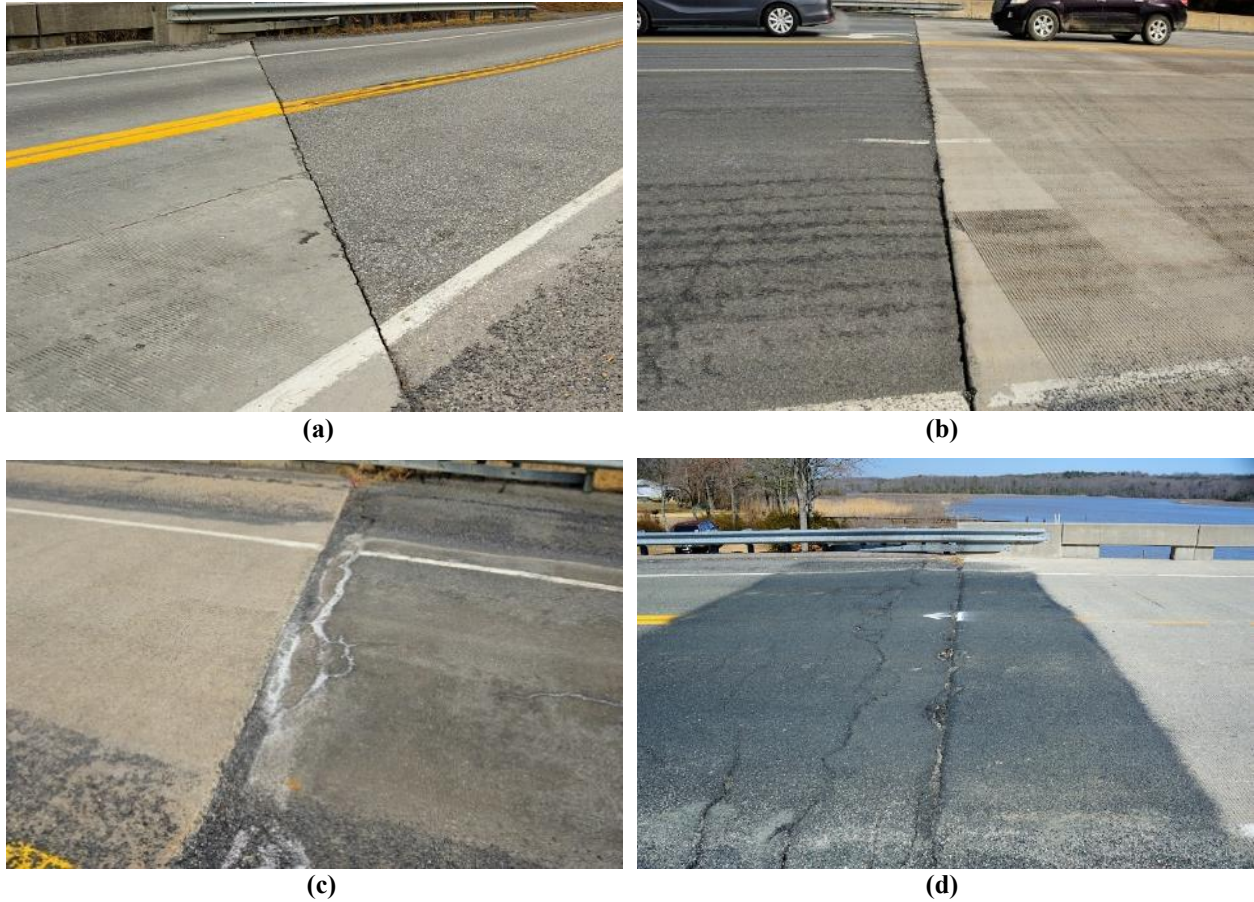


Figure 23. Photographs of Visual Reference for Qualitative Approach Condition Assessment: (a) Excellent; (b) Good; (c) Fair; (d) Poor

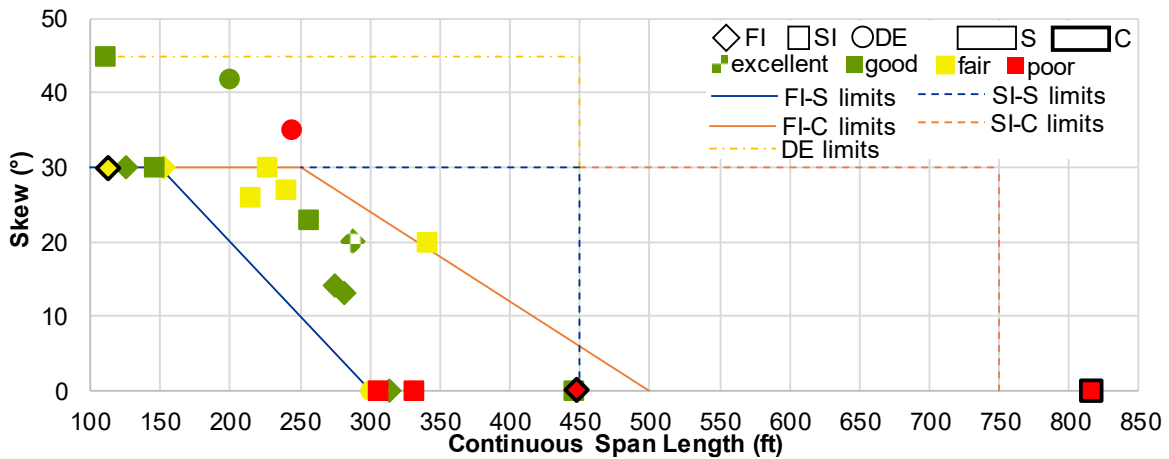


Figure 24. Visual Condition of Surveyed Pavement Approaches at Jointless Bridges. C = concrete superstructure; DE = deck extension; FI = full integral; S = steel superstructure; SI = semi-integral.

in the fair to poor subset. Thus, in general, bridge approaches on primary roads have been performing somewhat better than those on secondary roads. The relative difference in pavement section design may explain this trend. Comprehensive review of pavement structure data in VDOT's Pavement Management System database indicates that primary and secondary roads

across Virginia have an average asphalt thickness of 5 and 1.5 inches, respectively (Kalyan Asam, personal communication). Anecdotal evidence from the Staunton District indicates approximately 8 and 2 inches of asphalt pavement are typically used for primary and secondary roadway approaches, respectively (Park Thompson, personal communication).

Table 5 lists the observed roadway approach conditions in the decreasing order of excellent to poor visual rankings at the time of site inspection. Overall, 8 primary and 5 secondary roads were classified in the excellent to good subset, with urban roads equivalent to primary roads. In contrast, 8 primary and 9 secondary roads were observed in the fair to poor subset. Thus, in general, bridge approaches on primary roads have been performing somewhat better than those on secondary roads. The relative difference in pavement section design may explain this trend. Comprehensive review of pavement structure data in VDOT’s Pavement Management System database indicates that primary and secondary roads across Virginia have an average asphalt thickness of 5 and 1.5 inches, respectively (Kalyan Asam, personal communication). Anecdotal evidence from the Staunton District indicates approximately 8 and 2 inches of asphalt pavement are typically used for primary and secondary roadway approaches, respectively (Park Thompson, personal communication).

Table 5. Roadway Approach Condition at Surveyed Bridges

Federal ID No.	Year Built	Super-structure Material	Contin - uous Length (ft)	Skew (°)	Abutment Type ^a	EPS Thick-ness (in)	Drain - age ^b	Select Back-fill	Slab Type	Road Classi-fication	Interface Condi-tion ^c
25957	2005	steel	287	20	FI	none	weep	no	surface	Secondary	Excellent
4553	2010	steel	146	30	SI	none	none	no	surface	Primary	Good
25895	2005	steel	314	0	FI	none	pipe	yes	none	Primary	
25958	2004	steel	275	14	FI	none	geo	no	surface	Secondary	
25959	2004	steel	282	13	FI	none	yes	no	surface	Secondary	
26038	1999	steel	125	30	FI	none	yes	no	surface	Primary	
26452	2001	steel	256	23	SI	none	yes	no	surface	Secondary	
27413	2009	steel	446	0	SI	23	yes	yes	buried	Primary	
27431	2012	steel	94	21	SI	10	yes	yes	surface	Urban	
27484	2011	steel	306	0	FI	10–17	yes	yes	none	Secondary	
28171	2004	steel	160	0	SI	12	yes	yes	none	Primary	
28475	2006	steel	110	45	SI	15	yes	yes	none	Primary	
28834	2008	steel	200	42	DE	none	yes	yes	none	Primary	
20641	2000	steel	240	27	SI	none	yes	no	surface	Urban	
24736	1996	concrete	113	30	FI	none	no	no	none	Secondary	
25039	1997	steel	152	30	FI	none	yes	no	none	Secondary	
26017	2002	steel	194	17	SI	none	yes	no	surface	Primary	
26469	2001	steel	214	26	SI	none	yes	no	none	Primary	
27414	2008	steel	300	0	DE	none	yes	yes	buried	Primary	
27544	2012	steel	341	20	SI	10	yes	no	none	Urban	
27733	2003	steel	306	0	DE	none	yes	yes	none	Secondary	
28011	2009	concrete	600	0	SI	22	no	yes	surface	Primary	
28272	2013	steel	226	30	SI	14	yes	yes	none	Secondary	
28526	2015	steel	178	48	DE	none	yes	no	buried	Primary	
26230	2010	steel	306	0	SI	10	no	yes	none	Secondary	
26659	2000	steel	331	0	SI	6	yes	yes	none	Primary	
27344	2008	concrete	815	0	SI	10	no	yes	none	Secondary	
27542	2006	steel	243	35	DE	none	yes	no	none	Secondary	

Federal ID No.	Year Built	Super-structure Material	Contin - uous Length (ft)	Skew (°)	Abut - ment Type ^a	EPS Thick - ness (in)	Drain - age ^b	Select Back - fill	Slab Type	Road Classi - fication	Interface Condi - tion ^c
28614	2014	concrete	449	0	FI	16	yes	yes	none	Secondary	
29342	2019	steel	254	0	FI/SI	24 ^d	yes	yes	none	Secondary	

^a DE = deck extension; FI = full integral; SI = semi-integral.

^b board = 4-inch drainage board with weepholes; geo = geocomposite wall drain; pipe = perforated pipe parallel to footing; weep = weepholes.

^c Qualitative joint condition for the worst deck–approach interface at the given structure.

^d The full integral abutment at Bridge 29342 does not have EPS, but the semi-integral abutment does.

EPS = expanded polystyrene.

Base material underlying asphalt pavement typically consists of unbound crushed stone. The pavement section is designed for appropriate traffic loading but not for longitudinal loading. Figure 25 shows the long-term visual comparison of pavement approach conditions at Structure



Figure 25. Photographs Comparing the Approach Condition at Abutment A (Full Integral) versus Abutment B (Semi-Integral) for Bridge 29342
 No. 29342. The same roadway pavement section performed well at the stationary Abutment A while experiencing localized distress at Abutment B undergoing lateral cyclic movement in Figure 19.

DISCUSSION

The need to construct resilient bridges that require less extensive maintenance during their service life motivates a trend toward jointless bridge design. Figure 26 shows an example of a conventional bridge encountered during the study. This bridge with joints exemplifies the commonly occurring maintenance problem of leaking deck expansion joints and the resulting damage to beam ends and substructure elements.



Figure 26. Photographs Showing Deteriorated Beam Ends in Bridge 4554, a Structure with Joints at Abutments and Piers: (a) Abutment A; (b) Pier 1

From a structural perspective, field observations of jointless bridges visited during this study indicated that they perform remarkably well relative to their conventional counterparts. One potentially beneficial area to explore further may be wider implementation of full integral abutments due to their relatively low maintenance requirements. Full integral abutments are also the least expensive abutment type to construct. Furthermore, VDOT's length and skew limits for full integral steel structures are relatively conservative compared with most other states (Figure 27). Recall that Virginia's limit for steel bridges supported on full integral abutments at 0° skew is 300 feet. This length is comparatively shorter than for Iowa and Pennsylvania, which allow similar structures to span 390 and 400 feet, respectively. Likewise, Virginia's limit of 500 feet for concrete bridges with full integral abutments and no skew is less than that of Iowa, Pennsylvania, Maine, and South Dakota, which allow spans ranging from 575 to 700 feet.

In fact, Virginia has had successful experiences with integral abutment structures spanning longer distances than the current limits. Consider Bridge 27413, which is a 446-foot-long steel bridge, and Bridge 27344, an 815-foot-long concrete bridge; both structures have no skew. Although Bridge 27344 certainly had issues at the approaches, the abutments for both bridges exhibited excellent structural performance, with no evidence of cracking or corrosion in the abutment, beam ends, or the deck. Although the scope of this study did not include calculations specifically supporting the case for longer span lengths, Virginia's experience with full integral abutments supporting longer-span bridges, coupled with observations from other state departments of transportation, suggests that VDOT could increase the span limits of full integral steel and concrete bridges to 400 and 600 feet, respectively, when those structures are on a zero-degree skew.

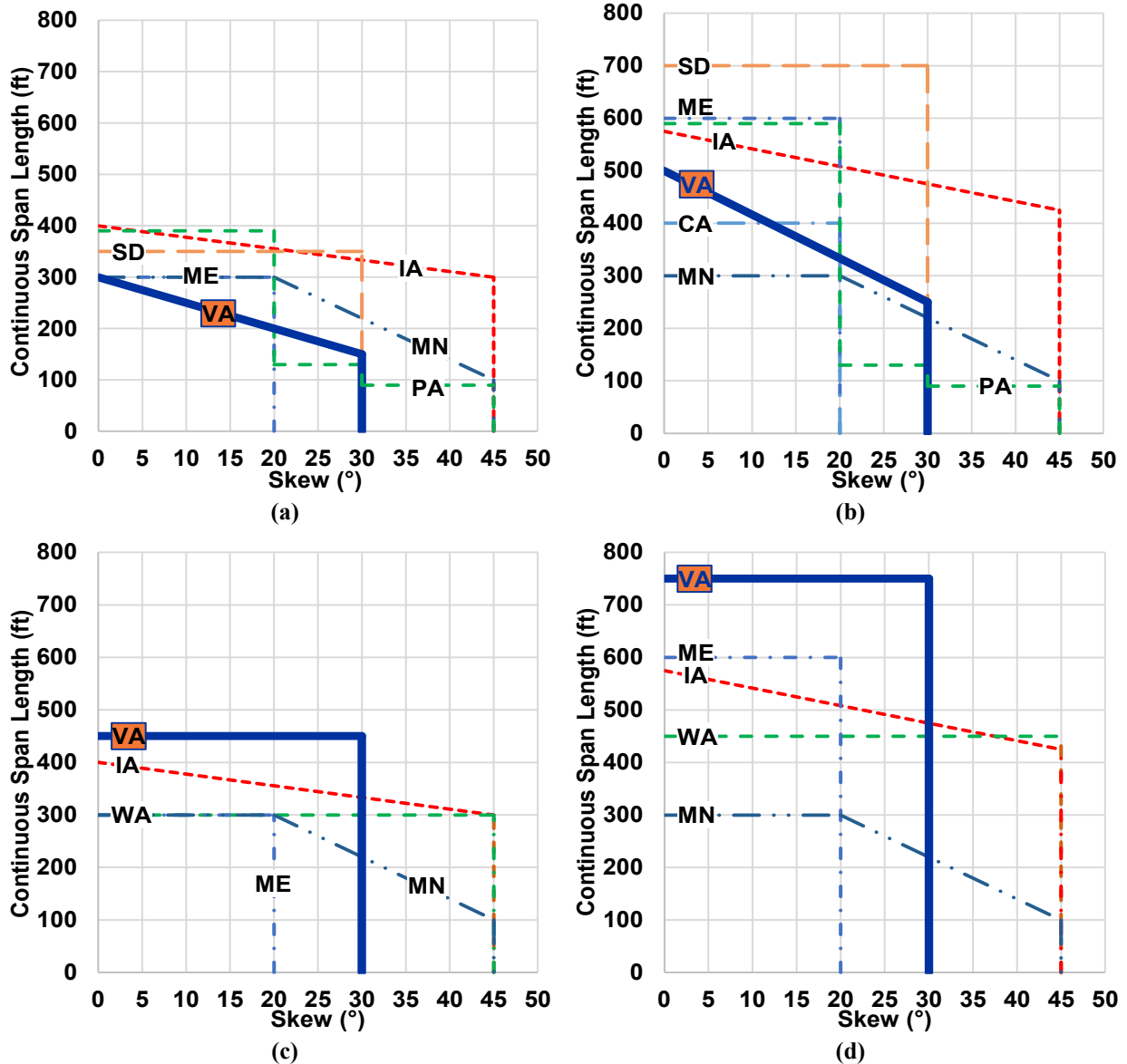


Figure 27. Comparison of VDOT's Current Design Limits versus Other Departments of Transportation: (a) Full Integral—Steel Superstructure; (b) Full Integral—Concrete Superstructure; (c) Semi-Integral—Steel Superstructure; (d) Semi-Integral—Concrete Superstructure

One component worth exploring is the embedment length for piles in full integral abutments. VDOT's current guidelines mandate a minimum of 25 feet, regardless of the subsurface conditions. This embedment length requirement may be an area for revision, addressing specific concerns about pile fixity, allowable pile design stress, and potentially excessive deck rotation at skewed full integral bridges resulting from the use of a single row of foundation piles.

The problem of excessive superstructure rotation at skewed semi-integral and deck extension bridges has been resolved by the introduction of the load buttress with steel rub plates at the abutment-wingwall vertical interface (Figure 28). Implemented by VDOT in 2007, this detail is intended to restrain the horizontal rotation of a skewed superstructure because of the

generation of noncollinear reaction forces (Figure 29). Figure 29 indicates that a tangential force component is needed to balance the rotation. Theoretically, friction generated at the interface between the backwall and the adjoining backfill material provides this force. In practice, however, this frictional resistance cannot be relied on under all service conditions. As the bridge contracts, water runoff and deicing chlorides can enter and lubricate the interface gap before the bridge starts expanding. During the subsequent expansion, no effective frictional resistance can be mobilized. The resulting movement of the skewed superstructure is particularly apparent on single-span semi-integral bridges with no rotational restraint provided by the pier bearings.

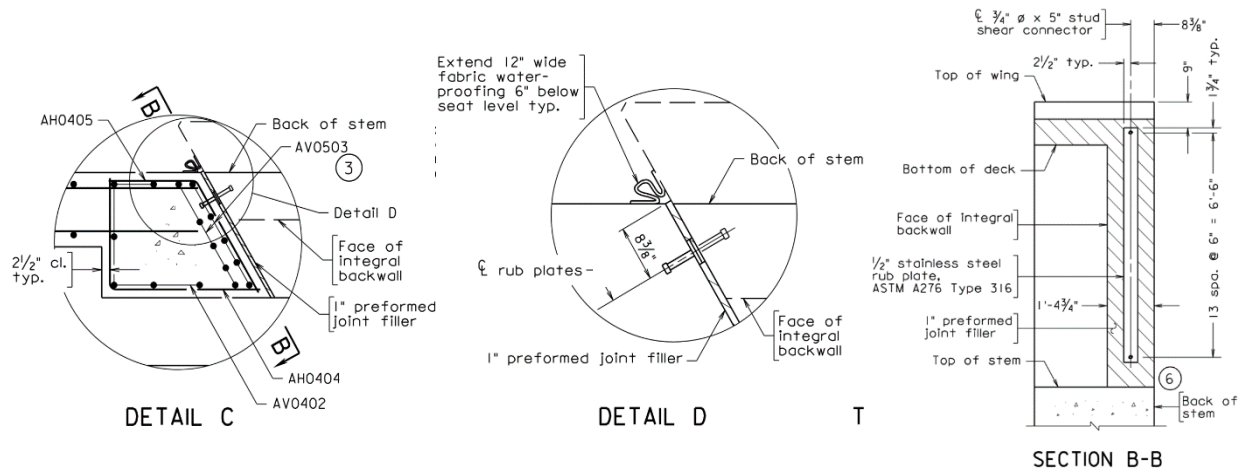


Figure 28. Various Views of VDOT's Current Load Buttress Detail (VDOT, 2024)

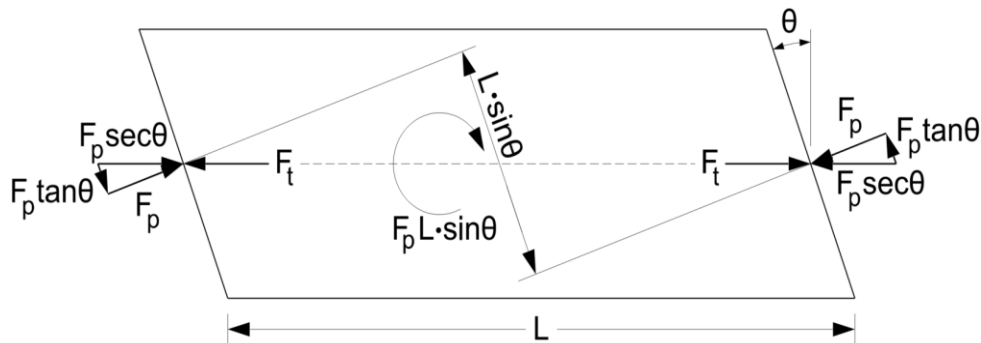
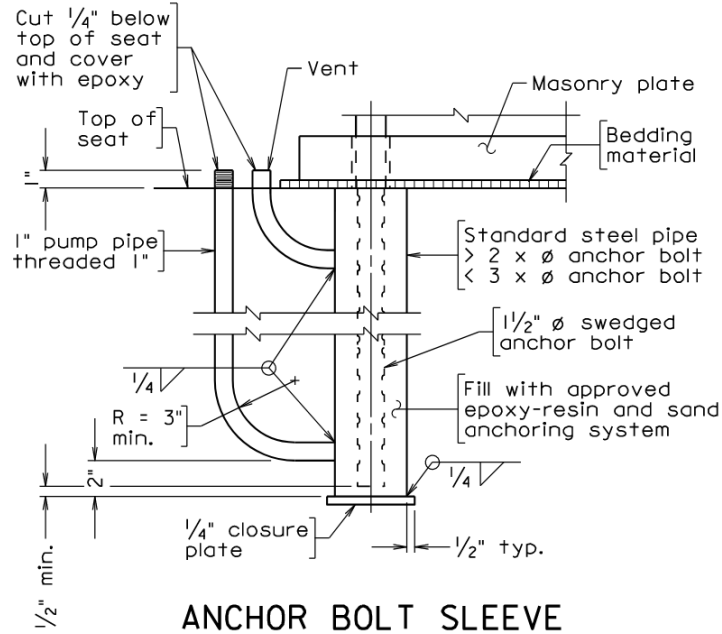


Figure 29. Free Body Diagram of the Reaction Forces on a Bridge Deck Caused by Thermal Expansion. F_p is the perpendicular reaction against the deck caused by F_t , the thermal expansive forces in the deck; L is the length of the deck; and θ is the skew angle relative to the line perpendicular to the bridge's centerline.

In addition to the load buttress, constructing U-back wingwalls on skewed bridges provides significantly increased lateral resistance to superstructure rotation. Also, wider slots in the sole plates have been allowed to provide additional clearance for anchor bolts. To ensure proper centering of the anchor bolt within the wide slot, steel sleeves are embedded in the abutment seat (Figure 30).



ANCHOR BOLT SLEEVE

Figure 30. VDOT Standard Drawing Suggested for Use in Ordinary Bearings to Enable Properly Set Anchor Bolts During Construction (VDOT, 2024)

Figure 31 shows the proposed revision of the sealing detail at the vertical gap between the backwall and adjoining wingwall, developed in Staunton District. The intent is to prevent unbound backfill material from spilling through the gap. A geotextile fabric combined with a polyvinyl chloride pipe provides an effective seal at the interface. Internal end plugs are installed in the pipe to ensure full interface contact along the entire gap length.

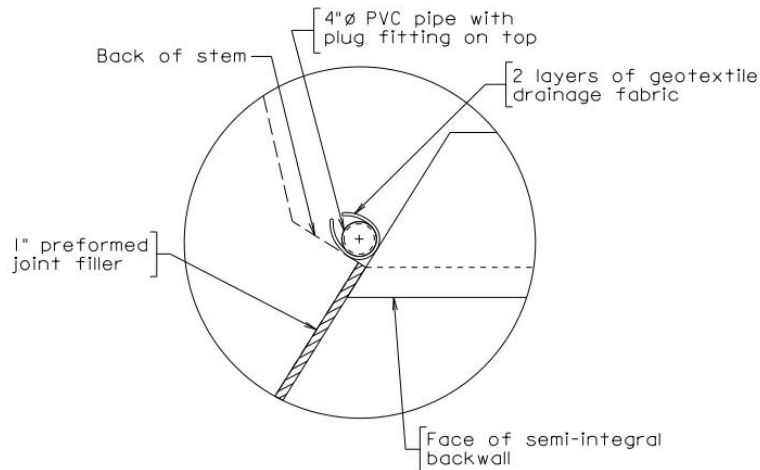
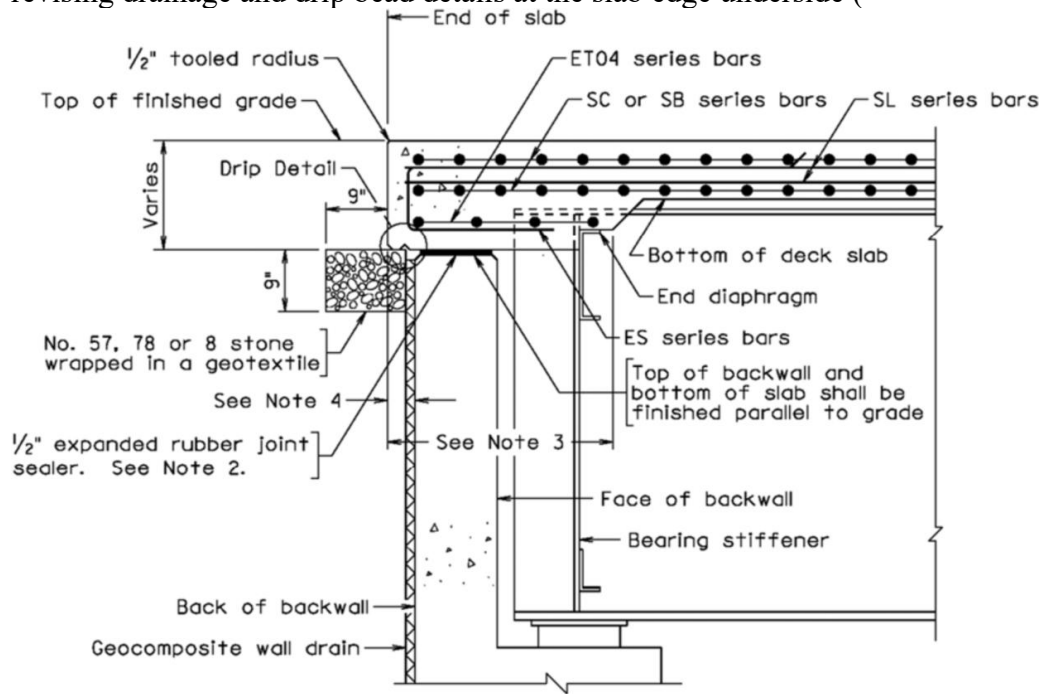


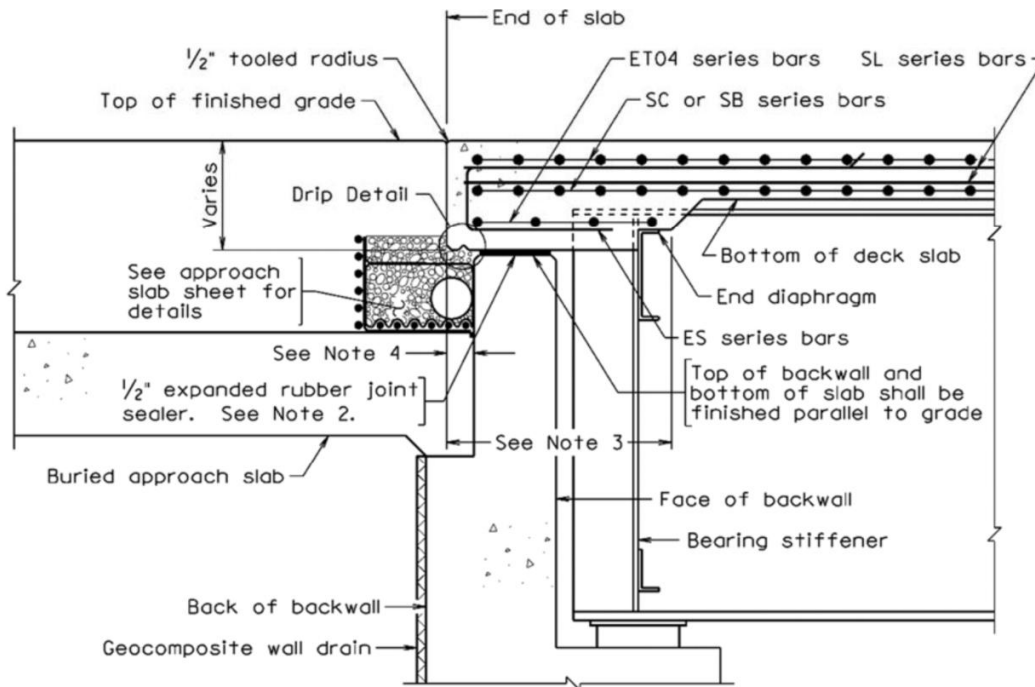
Figure 31. Proposed Joint Detail at the Backwall-Wingwall Interface

The water leakage problem at the deck slab extension has been largely addressed by revising drainage and drip bead details at the slab edge underside (



(b)

Figure 32). This current detail for deck extensions is based on the modified Virginia Department of Highways standard for steel beam bridges, originally developed in 1951 (Park Thompson, personal communication). The original standard was improved by lengthening the deck extension beyond the backwall and adding drainage detail.



(a)

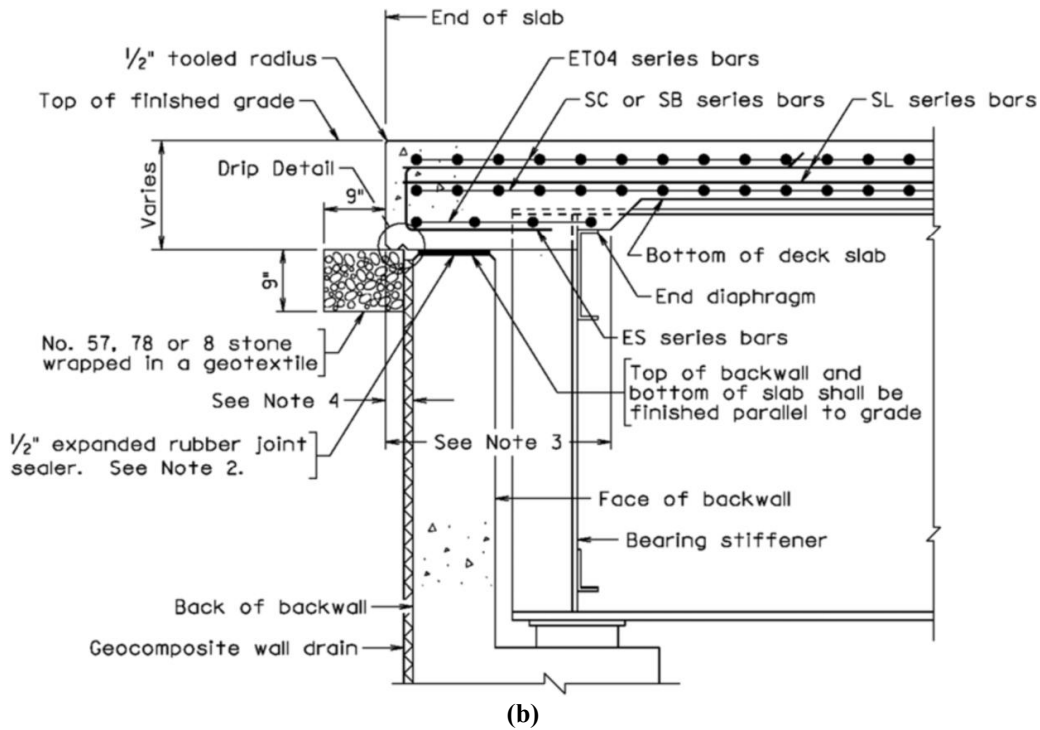


Figure 32. Deck Extension Drainage Detail with (a) Buried Approach Slab and (b) No Approach Slab

The elasticized EPS layer reduces the lateral pressure acting on the abutment and accommodates thermally induced lateral displacements. Its design thickness is based on the estimated range of abutment thermal movement, subject to a minimum specified thickness of 10 inches. EPS typically terminates at the approach slab seat elevation. On a bridge without concrete approach slabs, the integral abutment cyclic movement results in a localized pavement distress in the immediate vicinity of the bridge deck, as the red outline directly above the 24-inch-thick EPS layer shows in Figure 33a. This region is a relatively shallow zone of direct vertical contact (approximately 20 inches deep) between the bridge and the pavement section. In contrast, the approach embankment zone below the pavement section interacts with the bridge abutment through the 24-inch elasticized EPS layer, which absorbs lateral deformations.

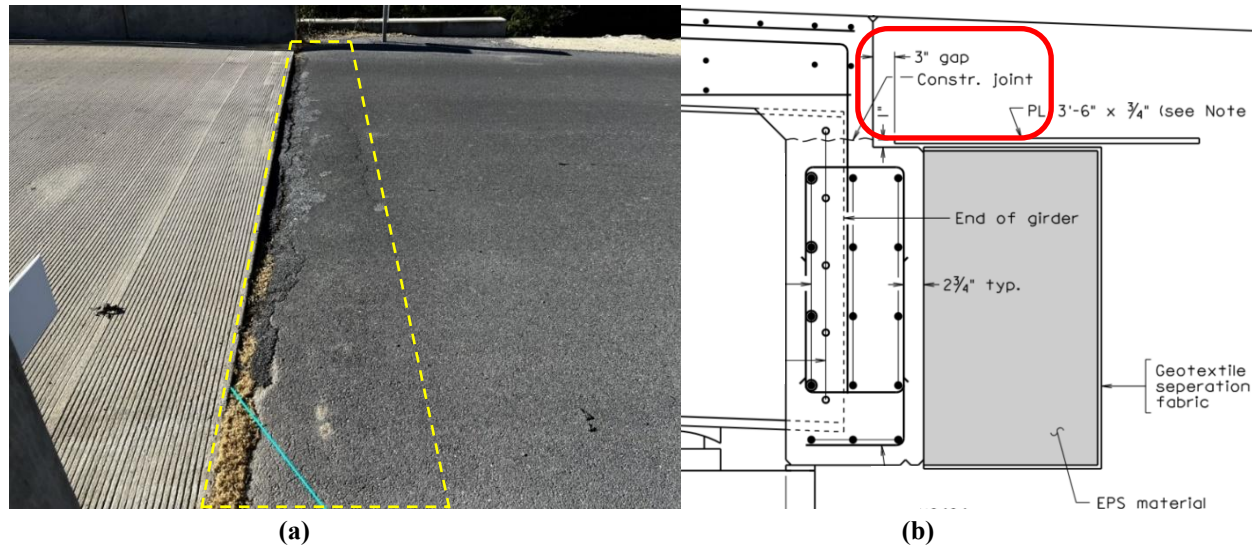


Figure 33. Semi-Integral Abutment in Bridge 29342, with (a) Visible Pavement Distress [Highlighted Within Dashed Yellow Lines] Because the Elasticized EPS Cannot Extend to the Surface, as the Region Encircled in Red Indicates in the (b)Section Drawing of the Abutment Design Showing Vertical Limits of Elasticized EPS to Alleviate Pressures at the Bridge Deck-Asphalt Approach Interface. EPS = expanded polystyrene.

The use of a steel plate above the EPS, acting as a mini approach slab, is intended to prevent local vertical deformation from wheel loads. This steel plate is specified only when no concrete approach slab is above the EPS layer. The efficacy of this detail needs to be evaluated against the more common practice of reducing the EPS design thickness in the upper portion. Without approach slabs, two competing requirements govern the EPS thickness: providing a sufficiently thick elastic layer to absorb cyclic horizontal deformations versus avoiding a wide soft spot that may be prone to settlement under traffic loading.

Buried approach slabs can also be used to mitigate differential settlement. However, these slabs do not entirely solve the problem of direct interaction between concrete abutment and asphalt pavement. For example, Structure Nos. 27413 and 27414, with buried concrete approach slabs, were found to have 0.25-inch and 0.75-inch deck-pavement elevation differentials, respectively.

The problem of unsightly rust staining on concrete calls for verifying current detailing practices. Guidelines for the recommended use of weathering steel are compiled by the National Steel Bridge Alliance (2022). To limit staining, the National Steel Bridge Alliance recommends using temporary protection in the form of plastic sheeting over the substructure concrete during the first few months the weathering steel is in service. Also, when the beam ends are cast integral with the abutment backwall, the National Steel Bridge Alliance recommends coating the encased portion of the beam with a protective primer. The primer or primed and painted coating must project past the backwall-beam interface for a sufficient distance to protect against the moisture buildup caused by the temperature differential between the backwall and the beam exposed to the atmosphere. Additional guidance on the proper use of unpainted weathering steel is provided in the Federal Highway Administration (FHWA) Technical Advisory 5140.22 and in the FHWA-HRT-23-116 report (FHWA, 1989, 2024). The UK detailing practice calls for suitable sealant along all interfaces between weathering steel and concrete to prevent excessive corrosion

(Highways England, 2019). It is not evident if the rust stains observed at interior beams originated during construction, before the deck was cast, or if they subsequently developed on a bridge in service.

Generally speaking, the structural elements in jointless bridges have performed satisfactorily. Beyond the bridge elements, this study has shown that no single jointless abutment type interacts with the roadway pavement substantially better than another in terms of the joint at the concrete-asphalt interface, cracking in the approaches, or differential settlement between the concrete and asphalt pavement. A supplemental document contains additional data comparing the performance of these elements versus various design or construction factors, such as span length, skew, backfill type, EPS thickness, drainage, fill height, route classification, and construction era (see the link to the VTRC's report supplements library in this report's Abstract section). Also, with regard to settlement, the data show that jointless abutments exhibit a similar history of settlement at the concrete-asphalt interface as conventional abutments. Thus, the problem of reconciling thermally induced superstructure displacements with the resulting distress at the adjoining pavement section is perhaps the most pressing matter that remains to be resolved. Apart from the issue of deck cracking, which was not detected in this study, the remaining issues in Table 4 were identified in this survey and are directly linked to bridge-pavement interaction. Although repairing a damaged roadway approach is arguably much less expensive than maintaining bridges with leaking expansion joints, improving the existing bridge-pavement transition detail would significantly reduce VDOT's maintenance workload and improve the rideability. Ultimately, the effective solution will require a multi-disciplinary approach involving structural, pavement, and geotechnical expertise.

CONCLUSIONS

- *Based on the visible structural condition of jointless abutments, VDOT's current limits on the length and skew of its jointless bridges could be extended.* Generally, jointless bridges in Virginia are in good condition, even those structures that exceed those limits. The use of full integral abutments, semi-integral abutments, and deck extensions effectively addresses maintenance problems associated with leaking expansion joints in conventional structures, namely deteriorated beam ends and bearings.
- *Integral designs frequently result in pavement distress at bridge approaches.* The thermally induced cyclic longitudinal movements of the jointless superstructure interacting with the adjoining roadway amplifies the recurring problem of the "bump at the end of the bridge."
- *No single jointless abutment type, with or without an approach slab, has an advantage over the others in terms of the performance of the approach pavement.*
- *Relaxing current length and skew limitations without addressing the soil-structure interaction problems may amplify pavement distress.*

RECOMMENDATIONS

1. *VDOT's Structure and Bridge Division should consider the design and construction of jointless bridges and their abutments, assuming lengths and skew that exceed the agency's current limits.*
2. *VTRC should work with VDOT's Structure and Bridge Division in conducting a follow-up project to design an improved approach pavement for jointless bridges. Ultimately, the successful implementation of jointless bridge technology will be evaluated in conjunction with the roadway approach interaction, and this research gap needs to be addressed.*

IMPLEMENTATION AND BENEFITS

The researcher and the technical review panel (listed in the Acknowledgments) for the project collaborate to craft a plan to implement the study recommendations and determine the benefits of doing so. This process is to ensure that the implementation plan is developed and approved with the participation and support of those involved with VDOT operations. The implementation plan and the accompanying benefits are provided here.

Implementation

Regarding Recommendation 1, VDOT's Structure and Bridge Division will amend Chapter 17 of VDOT's Manual of the Structure and Bridge Division, Part 2: Design Guidelines to allow for the use of full integral abutments at 0° skew for steel and concrete bridges up to 400 and 600 feet long, respectively. This task will be completed by October 30, 2026.

Regarding Recommendation 2, VTRC will support a research project or a pilot project addressing the rehabilitation of up to two approach pavements for an existing jointless bridge. VTRC's Pavements Team will work with VDOT's Structure and Bridge Division and the selected district to redesign the approaches and support their construction. The target completion date for the approach rehabilitation is August 30, 2028.

Benefits

The Staunton District recently provided an estimate for constructing a 40-foot-long full integral abutment costing about \$79,000 less than a semi-integral abutment of similar length. The projects constructed during the 3-year period from 2022 to 2024 included one bridge with 0° skew that could have benefited from a 100-foot increase in maximum length for full integral abutments but that particular bridge had a shorter abutment length. That is, the length of the abutment perpendicular to the direction of traffic was shorter than the Staunton District case. Assuming a similar rate of construction of full integral bridges that have 0° skew and are between 300 and 400 feet long, implementing Recommendation 1 would likely save about \$106,000 annually for the construction of two abutments per bridge. If VDOT were to apply the same maximum length for skewed bridges (that is, use linear interpolation between a 400-foot length at 0° skew and a 150-foot length at 30° skew to determine the maximum length), then

three additional semi-integral, skewed structures potentially could have been full integral abutment structures instead, soil conditions permitting. Again, assuming a similar construction rate, VDOT could achieve additional annual savings of \$316,000, based on 2025 dollars.

The second recommendation from this study calls for designing new approach pavements to address the “bump at the end of the bridge” and cracking in the approaches. Based on the scan in this current study, 28% of VDOT paved approaches for jointless bridges were in poor condition, and another 34% were in fair condition. Of those poor approaches, one, or 3% of the total sampled approaches without slabs, had relatively short lengths (up to 2 feet) of deterioration that could be repaired for \$5,000 to \$10,000 each. The remaining eight approaches, or 25% of the observed approaches without slabs, had more extensive damage that would require contractors with heavy equipment and cost approximately \$20,000 to \$30,000 each. The observations show that most approaches using current pavement designs will need similar repairs in the future. The inventory database indicated at least 139 “jointless” bridges with paved approaches (that is, 278 paved approaches at jointless abutments). If the new approach designs can avoid this need for cyclical repair, and assuming a similar percentage of inventoried approaches need repair as those observed in this study and the repair costs are at the middle of the given ranges, the department could save an additional \$1.8 million in approach maintenance costs. Once an approach has been identified as needing repair, residencies tend to address the issue within 1 or 2 years. However, rehabilitations may take longer, depending on the number of other projects a residency has at one time and the amount of funding available. Furthermore, residencies tend to initiate approach rehabilitations following citizen complaints instead of bridge safety inspection reports. Note that this savings estimate does not account for the possibility that those approaches that are currently in fair condition may eventually degrade into a poor condition.

Summing all the previously detailed benefits, VDOT could potentially save \$2.2 million during a 4-year period. Given that the budget for this 2-year project was \$265,000, those savings would equate to approximately a 730% return on investment.

In addition to saving money by improving the state of the practice in design and construction, safety is also an important element to consider. Maintenance work on bridge approaches invariably requires prolonged lane closures, leading to traffic congestion and the associated potential for construction-related accidents.

ACKNOWLEDGMENTS

The Federal Highway Administration supported this study. The authors gratefully acknowledge members of the technical review panel for their helpful guidance and support. The panel is composed of the following VDOT engineers: Junyi Meng, Structure and Bridge Division, Central Office; Bruce Shepard, Structure and Bridge Division, Central Office; Park Thompson, Structure and Bridge Office, Staunton District; and Ilker Boz, Research Scientist,

VTRC. Special thanks are extended to Nathan Maupin, Research Assistant at VTRC, for his assistance with processing data and preparing the tables and figures.

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