

Identification and Evaluation of Pavement-Bridge Interface Ride Quality Improvement and Corrective Strategies



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Prepared in cooperation with the
Ohio Department of Transportation
and the

U.S. Department of Transportation,
Federal Highway Administration

Report Number FHWA/OH-2011/1

State Job Number 134375

January 2011

IOWA STATE UNIVERSITY

Institute for Transportation



SI* (MODERN METRIC) CONVERSION FACTORS				
APPROXIMATE CONVERSIONS TO SI UNITS				
Symbol	When You Know	Multiply By	To Find	Symbol
LENGTH				
in	inches	25.4	millimeters	mm
ft	feet	0.305	meters	m
yd	yards	0.914	meters	m
mi	miles	1.61	kilometers	km
AREA				
in ²	square inches	645.2	square millimeters	mm ²
ft ²	square feet	0.093	square meters	m ²
yd ²	square yard	0.836	square meters	m ²
ac	acres	0.405	hectares	ha
mi ²	square miles	2.59	square kilometers	km ²
VOLUME				
fl oz	fluid ounces	29.57	milliliters	mL
gal	gallons	3.785	liters	L
ft ³	cubic feet	0.028	cubic meters	m ³
yd ³	cubic yards	0.765	cubic meters	m ³
NOTE: volumes greater than 1000 L shall be shown in m ³				
MASS				
oz	ounces	28.35	grams	g
lb	pounds	0.454	kilograms	kg
T	short tons (2000 lb)	0.907	megagrams (or "metric ton")	Mg (or "t")
TEMPERATURE (exact degrees)				
°F	Fahrenheit	5 (F-32)/9 or (F-32)/1.8	Celsius	°C
ILLUMINATION				
fc	foot-candles	10.76	lux	lx
fl	foot-Lamberts	3.426	candela/m ²	cd/m ²
FORCE and PRESSURE or STRESS				
lbf	poundforce	4.45	newtons	N
lbf/in ²	poundforce per square inch	6.89	kilopascals	kPa
APPROXIMATE CONVERSIONS FROM SI UNITS				
Symbol	When You Know	Multiply By	To Find	Symbol
LENGTH				
mm	millimeters	0.039	inches	in
m	meters	3.28	feet	ft
m	meters	1.09	yards	yd
km	kilometers	0.621	miles	mi
AREA				
mm ²	square millimeters	0.0016	square inches	in ²
m ²	square meters	10.764	square feet	ft ²
m ²	square meters	1.195	square yards	yd ²
ha	hectares	2.47	acres	ac
km ²	square kilometers	0.386	square miles	mi ²
VOLUME				
mL	milliliters	0.034	fluid ounces	fl oz
L	liters	0.264	gallons	gal
m ³	cubic meters	35.314	cubic feet	ft ³
m ³	cubic meters	1.307	cubic yards	yd ³
MASS				
g	grams	0.035	ounces	oz
kg	kilograms	2.202	pounds	lb
Mg (or "t")	megagrams (or "metric ton")	1.103	short tons (2000 lb)	T
TEMPERATURE (exact degrees)				
°C	Celsius	1.8C+32	Fahrenheit	°F
ILLUMINATION				
lx	lux	0.0929	foot-candles	fc
cd/m ²	candela/m ²	0.2919	foot-Lamberts	fl
FORCE and PRESSURE or STRESS				
N	newtons	0.225	poundforce	lbf
kPa	kilopascals	0.145	poundforce per square inch	lbf/in ²

*SI is the symbol for the International System of Units. Appropriate rounding should be made to comply with Section 4 of ASTM E380.
(Revised March 2003)

(Conversion Factors, 2010)

Technical Report Documentation Page

1. Report No. FHWA/OH-2011/1		2. Government Accession No.		3. Recipient's Catalog No.	
4. Title and Subtitle Identification and Evaluation of Pavement-Bridge Interface Ride Quality Improvement and Corrective Strategies				5. Report Date January 2011	
				6. Performing Organization Code	
7. Author(s) Brent Phares, David White, Jake Bigelow, Mark Berns, and Jiake Zhang				8. Performing Organization Report No.	
9. Performing Organization Name and Address Institute for Transportation Iowa State University 2711 South Loop Drive, Suite 4700 Ames, IA 50010-8664				10. Work Unit No. (TRAIS)	
				11. Contract or Grant No. State Job No. 134375	
12. Sponsoring Organization Name and Address Ohio Department of Transportation 1980 West Broad Street Columbus, OH 43223				13. Type of Report and Period Covered Final Report	
				14. Sponsoring Agency Code	
15. Supplementary Notes Visit www.intrans.iastate.edu for color PDF files of this and other research reports.					
16. Abstract <p>Bridge owners have long recognized that the approach pavement at bridges is prone to exhibiting both settlement and cracking, which manifest as the "bump at the end of the bridge." This deterioration requires considerable on-going maintenance expenditures, added risk to maintenance workers, increased distraction to drivers, reduced steering control, increased damage to vehicles, a negative public perception of the highway system, and a shortened useful bridge life. This problem has recently begun to receive significant national attention, as bridge owners have increased the priority of dealing with this recurring problem.</p> <p>No single factor, in and of itself (individually), leads to significant problems. Rather, it is an interaction between multiple factors that typically leads to problematic conditions. As such, solutions to the problem require interdisciplinary thinking and implementation. The bridge-abutment interface is a highly-complex region and an effective "bump at the end of the bridge" solution must address the structural, geotechnical, hydraulic, and construction engineering disciplines. Various design alternatives, construction practices, and maintenance methods exist to minimize bridge approach settlement, but each has its own drawbacks, such as cost, limited effectiveness, or inconvenience to the public.</p> <p>The objective of this work is to assist the Ohio Department of Transportation in the development of pre-construction, construction, and post-construction strategies that will help eliminate or minimize the "bump at the end of the bridge." Implementation of the details and procedures described herein will provide a tangible benefit to both the Ohio Department of Transportation and the traveling public, in the form of smoother bridge transitions, reduced maintenance costs, and a safer driving environment.</p> <p>As a result of this work, several conclusions and recommendations were made. Generally, these could be grouped into three categories: general, structural, and geotechnical. In some cases, the recommendations may require notable changes to the Ohio Department of Transportation bridge design policy. Suggestions for such changes have been made.</p>					
17. Key Words bridge-abutment bump—bridge approach deterioration— bridge-pavement interface—bridge ride quality—pavement-bridge interface—pavement settling—ride quality improvement				18. Distribution Statement No restrictions. This document is available to the public through the National Technical Information Service, Springfield, Virginia 22161	
19. Security Classification (of this report) Unclassified.		20. Security Classification (of this page) Unclassified.		21. No. of Pages 264	
				22. Price	

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**Final Report
January 2011**

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Sponsored by the
Ohio Department of Transportation
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Report Number FHWA/OH-2011/1
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Acknowledgments

The authors acknowledge and thank the following Ohio Department of Transportation (ODOT) personnel for their assistance over the course of this project: Brian Schleppi, data administrator manager; Chris Pridemore, engineer; Jeff Crace, structural engineer; Keith Geiger, district construction engineer; Scott LeBlanc; Gary Middleton; Sean Meddles; and Patrick Luff, former ODOT intern.

TABLE OF CONTENTS

EXECUTIVE SUMMARY	ix
1. GENERAL INFORMATION.....	1
1.1. Introduction.....	1
1.2. Background.....	1
1.3. Project Objectives	3
1.4. Project Scope	3
1.5. Report Content.....	3
2. GENERAL LITERATURE REVIEW.....	5
2.1. Integral Abutments.....	5
2.2. Approach Slabs	6
2.3. Approach Slab to Bridge Interface	6
2.4. Embankment Design and Construction	11
2.5. Abutment Slabs.....	14
3. OHIO BRIDGE DESIGN AND CONSTRUCTION STATE OF PRACTICE.....	16
3.1. Standard Bridge Drawings and Bridge Design Manual.....	16
3.2. Ohio Current and Past Research	30
4. OTHER STATES' BRIDGE DESIGN AND CONSTRUCTION STATE OF PRACTICE	33
4.1. Colorado.....	33
4.2. Illinois	36
4.3. Iowa.....	39
4.4. Kansas	49
4.5. Kentucky	50
4.6. Louisiana.....	53
4.7. Massachusetts	54
4.8. Michigan	60
4.9. Minnesota.....	61
4.10. Missouri	68
4.11. Nebraska	69
4.12. New Hampshire	70
4.13. New Mexico.....	71
4.14. New York.....	73
4.15. North Dakota.....	74
4.16. Pennsylvania	76
4.17. South Dakota.....	85
4.18. Tennessee.....	86
4.19. Virginia	93
4.20. Washington	95
4.21. Wisconsin.....	98

5. IN-SERVICE BRIDGE TESTING AND PERFORMANCE.....	99
5.1. Geometric Bridge Testing and Support System Evaluation	99
5.2. Live Load Testing.....	101
5.3. FAI 33-14.17.....	104
5.4. MUS 16-7.69.....	116
5.5. RIC 430-9.98.....	124
5.6. FRA 317-8.09	131
5.7. PRE 70-12.49.....	138
5.8. LIC 40-12.53.....	145
5.9. WYA 30-22.40.....	151
5.10. FRA 270-32.36	157
5.11. ERI 60-2.39.....	169
6. IN SITU EVALUATION OF NEW BRIDGE APPROACH FILL MATERIALS.....	181
6.1. Introduction.....	181
6.2. Test Methods.....	181
6.3. Laboratory Test Results	185
6.4. Field Study Results	203
6.5. Summary of Key Findings	220
7. SUMMARY, CONCLUSIONS, AND RECOMMENDATIONS.....	221
7.1. Summary	221
7.2. Conclusions/Findings.....	222
7.3. Recommendations.....	224
REFERENCES	243

LIST OF FIGURES

Figure 2.1. Long approach slab T shape and reinforcement (Cai et al., 2005B)	8
Figure 2.2. Deck steel extension connection (Greimann et al., 2008)	9
Figure 2.3. Abutment steel connection (Greimann et al., 2008)	9
Figure 2.4. Abutment with no connection (Greimann et al., 2008)	10
Figure 2.5. Problems leading to the formation of the bump (Briaud et al., 1997)	12
Figure 2.6. Temperature induced movement of an integral abutment bridge (Greimann et al., 2008)	13
Figure 3.1. Typical abutment detail for simply supported girders (ODOT, 2009)	17
Figure 3.2. Integral abutment detail (ODOT, 2009)	18
Figure 3.3. Integral abutment connection to pile cap beam (ODOT, 2009)	18
Figure 3.4. Semi-integral abutment configuration (ODOT, 2009)	19
Figure 3.5. Typical concrete approach slab (ODOT, 2009)	20
Figure 3.6. Abutment to approach slab joint detail (ODOT, 2009)	21
Figure 3.7. Abutment to approach slab joint detail (cont.) (ODOT, 2009)	22
Figure 3.8. Compression seal expansion joint for steel girder bridge (ODOT, 2009)	23
Figure 3.9. Compression seal expansion joint for concrete box girder (ODOT, 2009)	23
Figure 3.10. Strip seal expansion joint detail for steel girder bridges (ODOT, 2009)	24
Figure 3.11. Pressure relief joint (ODOT, 2009)	24
Figure 4.1. CDOT approach slab with expansion joint at the sleeper slab plan and section (2009)	33
Figure 4.2. Approach slab connection and bearing at bridge (CDOT, 2009)	34
Figure 4.3. Approach slab expansion joint at sleeper slab (CDOT, 2009)	34
Figure 4.4. CDOT approach slab section with expansion joint located at bridge joint (2009)	35
Figure 4.5. Approach slab with asphalt roadway and 3 in. asphalt overlay (CDOT, 2009)	35
Figure 4.6. Typical Illinois detail (Greimann et al., 2008)	37
Figure 4.7. Iowa DOT typical integral abutment design (2009)	40
Figure 4.8. Iowa DOT typical approach slab plan and profile (2009)	40
Figure 4.9. Iowa DOT typical expansion joint details (2009)	41
Figure 4.10. Common problems seen at bridge sites in Iowa (White et al., 2005)	43
Figure 4.11. Effective surface drain detail (White et al., 2005)	44
Figure 4.12. ISU water management bridge approach model schematic (White et al., 2005)	45
Figure 4.13. ISU water management bridge approach model (White et al., 2005)	45
Figure 4.14. Precast paving notch system selected for field implementation (BEC, 2008)	47
Figure 4.15. Typical Kansas detail (Greimann et al., 2008)	49
Figure 4.16. Kentucky typical approach slab details and abutment connection (KYTC, 2009)	52
Figure 4.17. Kentucky typical expansion joint detail (KYTC, 2009)	53
Figure 4.18. Typical LA DOTD pile approach slab support (DAS et al., 1999)	54
Figure 4.19. MassDOT integral abutment reinforcement (2009)	55
Figure 4.20. MassDOT approach slab plan (2009)	56
Figure 4.21. MassDOT typical approach slab details: approach slab Type 1 detail (2009)	56
Figure 4.22. MassDOT typical approach slab details: approach slab Type 2 detail (2009)	57
Figure 4.23. MassDOT typical approach slab details: approach slab Type 3 detail (2009)	57
Figure 4.24. MassDOT paving notch details for lowered approach slabs (2009)	58
Figure 4.25. MDOT strip seal expansion joints (2009)	60

Figure 4.26. Mn/DOT typical approach slab fixed at abutment and bituminous mainline pavement (2009)	62
Figure 4.27. Mn/DOT typical approach slab fixed at abutment and concrete mainline pavement (2009)	63
Figure 4.28. Mn/DOT typical sleeper slab details with E&S expansion joint (2009)	64
Figure 4.29. Mn/DOT typical expansion joint details (2009).....	65
Figure 4.30. Typical Minnesota detail (Greimann et al., 2008).....	66
Figure 4.31. Mn/DOT integral abutment finished grading section (2009)	67
Figure 4.32. MoDOT typical approach slab plan (2009).....	68
Figure 4.33. MoDOT typical approach slab section (2009)	68
Figure 4.34. Typical Missouri detail (Greimann et al., 2008)	69
Figure 4.35. Typical Nebraska detail (Greimann et al., 2008)	70
Figure 4.36. NHDOT typical strip seal detail (2009)	70
Figure 4.37. Drainage gutter used in New Mexico for moving water away from the bridge embankment (Lenke, 2006)	72
Figure 4.38. Settlement around drainage structure that is next to departure slab and in travel lane (Lenke, 2006)	72
Figure 4.39. Typical New York detail (2009).....	73
Figure 4.40. Typical North Dakota detail (Greimann et al., 2008).....	74
Figure 4.41. NDDOT approach slab drainage (2009).....	75
Figure 4.42. PennDOT typical integral abutment detail (2009)	76
Figure 4.43. PennDOT approach slab connection to integral abutment detail (2009)	77
Figure 4.44. PennDOT Type 1 typical approach slab configuration for simply supported bridge girders (2009).....	77
Figure 4.45. PennDOT typical expansion joints for simply supported girders (2009).....	78
Figure 4.46. PennDOT typical sleeper slab joints for approach slab (2009).....	79
Figure 4.47. PennDOT Type 3 approach slab section for connection at girder with a backwall (2009)	80
Figure 4.48. PennDOT Type 4 approach slab section for connection at girder with a backwall and a drain trough at end of approach slab (2009)	81
Figure 4.49. PennDOT Approach slab connection to girder when a abutment backwall is present (2009)	81
Figure 4.50. PennDOT typical expansion joint details at end of approach slab (2009)	83
Figure 4.51. PennDOT tooth expansion joint at approach slab drain trough (2009).....	83
Figure 4.52. PennDOT Type 5 approach slab section used at integral abutment bridges (2009)	84
Figure 4.53. Typical South Dakota detail (Greimann et al., 2008).....	86
Figure 4.54. TDOT simply supported girder abutment with strip seal expansion joint (2010).....	87
Figure 4.55. TDOT simply supported girder with connected bridge deck (2010)	88
Figure 4.56. TDOT simply supported girder with alternate expansion device (2010)	89
Figure 4.57. TDOT integral abutment (2010).....	90
Figure 4.58. TDOT typical approach slab (2010).....	91
Figure 4.59. TDOT various sleeper slab configurations for the approach slab end (2010).....	92
Figure 4.60. Approach slab distress attributable to foundation soil settlement (Route 10 westbound lane over the Appomattox River) (Hoppe, 1999)	94
Figure 4.61. VDOT proposed approach slab details for non-integral bridges (Hoppe, 1999)	94
Figure 4.62. Erosion control design detail for bridge without approach slabs (Hoppe, 1999)	95

Figure 4.63. WSDOT typical approach slab detail (2009)	96
Figure 4.64. WSDOT approach slab rigid connection to the bridge abutment (2009)	96
Figure 4.65. WSDOT expansion joint connection at approach slab-bridge joint interface (2009)	97
Figure 4.66. WSDOT expansion joint details for anchor head and compression seal (2009)	97
Figure 4.67. WSDOT approach slab to mainline pavement joint typical detail (2009)	97
Figure 4.68. WSDOT paving notch field replacement detail (2009)	98
Figure 5.1. Location of in-service bridges tested	99
Figure 5.2. Bridge global geometric evaluation system	100
Figure 5.3. ODOT falling weight deflectometer (FWD) used for approach testing	101
Figure 5.4. Typical strain and displacement transducers approach slab layout	102
Figure 5.5. Displacement transducer at approach-pavement joint	102
Figure 5.6. Abutment horizontal translation and rotation monitoring	103
Figure 5.7. Girder strain monitoring	103
Figure 5.8. FAI 33-14.17 bridge	104
Figure 5.9. FAI 33-14.17 approach slab cracking	105
Figure 5.10. FAI 33-14.17 asphalt wedge and oil staining located at exit of bridge	105
Figure 5.11. FAI 33-14.17 bridge geometric testing results	110
Figure 5.12. FAI 33-14.17 bridge live load testing instrumentation layout	112
Figure 5.13. FAI 33-14.17 bridge live load testing results	115
Figure 5.14. MUS 16-7.69 profile and MSE wall	116
Figure 5.15. MUS 16-7.69 approach slab settlement relative to main line pavement	117
Figure 5.16. Approach to pavement joint filled with grout causing a hump	117
Figure 5.17. Dip in pavement when exiting the bridge	118
Figure 5.18. Fill loss between the MSE wall and the bridge abutment	118
Figure 5.19. MUS 16-7.69 bridge geometric testing results	123
Figure 5.20. RIC 430-9.98 crossing over I-71	124
Figure 5.21. Abutment conditions at RIC 430-9.98	124
Figure 5.22. RIC 430-9.98 newly placed asphalt wedge and mud-jacked slab	125
Figure 5.23. Tapered vertical gap at bridge to approach slab joint indicating settlement	125
Figure 5.24. Void under approach	126
Figure 5.25. RIC 430-9.98 bridge geometric testing results	130
Figure 5.26. Profile view of FRA 317-8.09	131
Figure 5.27. Asphalt wedges at bridge abutment	132
Figure 5.28. Differential settlement at pavement and bridge abutment	132
Figure 5.29. FRA 317-8.09 bridge geometric testing results	137
Figure 5.30. Profile of PRE 70-12.49 prior to widening	138
Figure 5.31. PRE 70-12.49 after bridge widening	138
Figure 5.32. Condition of bridge and approach joints	139
Figure 5.33. Barrier rail joint at approach slab showing settlement of approach slab	139
Figure 5.34. Oil staining on bridge surface caused by bump at pavement-to-approach joint	140
Figure 5.35. PRE 70-12.49 bridge geometric testing results	144
Figure 5.36. LIC 40-12.53 profile view	145
Figure 5.37. Pavement-to-approach joint condition at LIC 40-12.53	146
Figure 5.38. Condition of LIC 40-12.53 bridge surface	146
Figure 5.39. LIC 40-12.53 bridge geometric testing results	150

Figure 5.40. WYA 30-22.40 profile view	151
Figure 5.41. WYA 30-22.40 pressure relief joint deterioration.....	152
Figure 5.42. Bar grates and curb provide good water drainage and eliminates erosion at the edge of the shoulder	152
Figure 5.43. WYA 30-22.40 bridge geometric testing results	157
Figure 5.44. FRA 270-32.36 profile view.....	158
Figure 5.45. 12 in. pressure relief joint condition.....	158
Figure 5.46. Erosion from bridge embankment.....	159
Figure 5.47. FRA 270-32.36 bridge geometric testing results.....	163
Figure 5.48. FRA 270-32.36 live load testing instrumentation	165
Figure 5.49. FRA 270-32.36 bridge live load testing results.....	168
Figure 5.50. Five spans of ERI 60-2.39	169
Figure 5.51. Girder bearing at abutment.....	170
Figure 5.52. Approach slab joint at the pavement and bridge	170
Figure 5.53. New asphalt pavement up to approach slab	171
Figure 5.54. Deterioration of strip seal	171
Figure 5.55. ERI 60-2.39 bridge geometric testing results.....	176
Figure 5.56. ERI 60-2.39 live load testing instrumentation.....	177
Figure 5.57. ERI 60-2.39 bridge live load testing results	180
Figure 6.1. Laboratory evaluation of the collapse potential of the backfill materials	182
Figure 6.2. In situ testing methods/devices.....	183
Figure 6.3. E_{V1} and E_{V2} determination from static plate load test.....	185
Figure 6.4. Grain-size distribution curves for materials sampled from Marzane at Perryville ...	188
Figure 6.5. Grain-size distribution curves for materials sampled from Shelly at Newark and West Mill Grove	188
Figure 6.6. Grain-size distribution curves for material sampled from Bridge #1	189
Figure 6.7. Grain-size distribution curve for material sampled from Bridge #2	189
Figure 6.8. Grain-size distribution curves for materials sampled from Bridge #3	190
Figure 6.9. Grain-size distribution curves for materials sampled from Bridge #4	190
Figure 6.10. Grain-size distribution curves for materials sampled from Bridge #5	191
Figure 6.11. Grain-size distribution curve for material sampled from Bridge #6	191
Figure 6.12. Moisture and dry unit weight relationships developed by using vibratory compaction (bulking moisture contents in the range of about 6%)	192
Figure 6.13. Collapse potential tests results for material - Shelly at Newark natural sand	193
Figure 6.14. Collapse potential tests results for material – West Mill Grove MF sand.....	194
Figure 6.15. Collapse potential tests results for material – Marzane at Perryville Sand 1	195
Figure 6.16. Collapse potential tests results for material – Marzane at Perryville gravel	196
Figure 6.17. Collapse potential tests results for material – Marzane at Perryville Sand 2	197
Figure 6.18. Pre-saturation and post-saturation modulus versus moisture content	198
Figure 6.19. Collapse potentials versus moisture content.....	199
Figure 6.20. Collapse potential versus moisture contents – Marzane at Perryville Sand 2	200
Figure 6.21. Collapse potential versus moisture contents – Marzane at Perryville Sand 2	201
Figure 6.22. Collapse potential and dry unit weight versus moisture content for Marzane at Perryville Sand 2.....	202
Figure 6.23. Bridge #1 (BUT-75-0660) at I-75 and SR 129 interchange	204

Figure 6.24. DCP-CBR profiles at selected distances away from the NE and SW MSE walls – Bridge #1	204
Figure 6.25. CBR at different depths from the top of the MSE wall at various distances away from the walls – Bridge #1	205
Figure 6.26. Moisture and dry density measurements at selected distances away from the MSE walls – Bridge #1	205
Figure 6.27. Bridge #2 (CL1-73-0985) at Wilmington.....	207
Figure 6.28. DCP-CBR profiles at selected distances away from the MSE wall – Bridge #2 (USCS: SW-SM).....	207
Figure 6.29. Bridge #3 (MOT-75-1393) at downtown Dayton	208
Figure 6.30. DCP-CBR profiles at selected distances away from south wall and paving notch at east abutment – Bridge #3	209
Figure 6.31. E_{LWD-Z2} measurements at selected distances away from paving notch and south wall on the east abutment – Bridge #3	209
Figure 6.32. Bridge #4 (FRA-670-0904B) near the Columbus airport.....	210
Figure 6.33. DCP-CBR profiles at selected distances away from the abutment – Bridge #4	210
Figure 6.34. E_{LWD-Z2} at selected distances away from the approach slab – Bridge #4	211
Figure 6.35. Bridge #5 (LIC-37-1225L) at Licking 161 over Moots Run.....	212
Figure 6.36. Proctor curve and field moisture and dry density measurement – Bridge #5	213
Figure 6.37. DCP-CBR profiles at test locations away from the east and west abutments – Bridge #5.....	213
Figure 6.38. E_{LWD-Z2} measurements at test locations away from the east and west abutments – Bridge #5	214
Figure 6.39. Moisture and dry density measurements at test locations away from the east and west abutments – Bridge #5	214
Figure 6.40. Bridge #6 (MED-71-0729) at I-71 and I-76 interchange	216
Figure 6.41. DCP-CBR profiles at test locations away from the south abutment for west and east lanes – Bridge #6 (USCS: GP-GM).....	217
Figure 6.42. E_{LWD-Z2} at test locations away from the south abutment on east and west lanes – Bridge #6 (USCS: GP-GM).....	217
Figure 6.43. Stress-strain curves for static plate load tests – Bridge #6 (USCS: GP-GM)	218
Figure 6.44. Bridge #7 (MED-71-0750) at I-71 and I-76 interchange	219
Figure 6.45. DCP-CBR profiles at test locations away from east abutment – Bridge #7.....	219
Figure 6.46. E_{LWD-Z2} at test locations away from east abutment – Bridge #7.....	220
Figure 7.1. Alternative integral bridge approach drainage detail with porous backfill (White et al., 2005)	230
Figure 7.2. Alternative integral bridge approach drainage detail with geocomposite (White et al., 2005)	231
Figure 7.3. Alternative integral bridge approach drainage detail with geotextile reinforcement (White et al., 2005).....	232
Figure 7.4. Alternative integral bridge approach drainage detail with tire chip backfill (White et al., 2005)	233

LIST OF TABLES

Table 2.1. Reinforcement ratio of slab under different settlement	7
Table 3.1. Item 304 gradation (ODOT, 2009)	25
Table 3.2. Item 411 gradation (ODOT, 2009)	26
Table 3.3. Item 617 gradation (ODOT, 2009)	26
Table 3.4. Granular Material Type C gradation (ODOT, 2009)	26
Table 3.5. Granular Material Type D gradation (ODOT, 2009)	26
Table 3.6. Size of coarse aggregate (AASHTO M43) (ODOT, 2009)	27
Table 3.7. Embankment compaction requirements (ODOT, 2009)	28
Table 3.8. Structural backfill gradation (ODOT, 2009)	29
Table 3.9. Granulated slag gradation requirements (ODOT, 2009)	29
Table 4.1. Porous granular coarse aggregate gradation (IDOT, 2010)	37
Table 4.2. Porous granular fine aggregate quality (IDOT, 2010)	38
Table 4.3. Porous granular fine aggregate material (IDOT, 2010)	38
Table 4.4. Iowa DOT criteria for using and integral or stub abutment (2009)	39
Table 4.5. Aggregate quality specifications (granular backfill materials) (Iowa DOT, 2009)	42
Table 4.6. Embankment gradation classification (KDOT, 2009)	49
Table 4.7. Various types of soil compaction requirements by KDOT (2009)	50
Table 4.8. Special borrow crushed rock gradation (MassDOT, 2008)	59
Table 4.9. Gravel borrow gradation requirements (MassDOT, 2008)	59
Table 4.10. Crushed stone gradation (MassDOT, 2008)	59
Table 4.11. Granular Material Gradation	61
Table 4.12. PennDOT select granular material	85
Table 6.1. Locations for evaluation of under-construction bridge approach backfill characteristics	181
Table 6.2. Summary of in situ testing at different bridge locations	184
Table 6.3. Soil index properties of bridge approach fill materials tested in situ	186
Table 6.4. Summary of index properties of bridge approach fill materials tested in situ	187
Table 7.1. Summary of geotechnical-related specifications reviewed and suggestions for future specification updates	228
Table 7.2. Summary bump identification metrics and troubleshooting	235
Table 7.3. Evaluation of bridge rideability and corrective strategies	241

EXECUTIVE SUMMARY

Bridge owners have long recognized that the approach pavement at bridges is prone to exhibiting both settlement and cracking, which manifest as the “bump at the end of the bridge.” This deterioration requires considerable on-going maintenance expenditures, added risk to maintenance workers, increased distraction to drivers, reduced steering control, increased damage to vehicles, a negative public perception of the highway system, and shortened useful bridge life. This problem has recently begun to receive significant national attention as bridge owners have increased the priority of dealing with this recurring problem.

No single factor, in and of itself (individually), leads to significant problems. Rather, it is an interaction between multiple factors that typically leads to problematic conditions. As such, solutions to the problem require interdisciplinary thinking and implementation. The bridge-abutment interface is a highly complex region and an effective “bump at the end of the bridge” solution must address the structural, geotechnical, hydraulic, and construction engineering disciplines. Various design alternatives, construction practices, and maintenance methods exist to minimize bridge approach settlement, but each has its own drawbacks, such as cost, limited effectiveness, or inconvenience to the public.

The objective of this work is to assist the Ohio Department of Transportation (ODOT) in the development of pre-construction, construction, and post-construction strategies that will help eliminate or minimize the “bump at the end of the bridge.” Implementation of the details and procedures described herein will provide a tangible benefit to both ODOT and the traveling public in the form of smoother bridge transitions, reduced maintenance costs, and a safer driving environment.

To achieve the project goals, the following general activities were performed:

- Review of the ODOT design and construction standards and specifications
- Literature review
- Review and summary of current, nationwide state-of-the-practice
- Field investigation of the behavior and condition of in service bridges
- Laboratory and field testing of bridge embankment materials
- Compilation and comparison of collected information
- Development of recommendations

The following recommendations were developed as a result of this work:

- In addition to profiling bridges, it is recommended that ODOT begin a program of measuring the gross vertical geometry of all bridges.
- It is recommended that all new bridges be profiled and have the gross vertical geometry measured immediately after construction.
- It is recommended that all bridges be profiled and have the gross vertical geometry measured on a long-term recurrence schedule (at least every 10 years) and when ride quality begins to degrade. Furthermore, the approach pavement slope should be

calculated and examined for changes. Similarly, ODOT should begin also calculating the Bridge Approach Performance Index so that it may be examined for changes over time.

- A specification that ensures a minimum ride quality at the time of construction should be created and adopted by ODOT. It is suggested that the specification should contain two parts: one for maximum global roughness and one for maximum local roughness.
- On structures where unusual/unproven construction practices are required (or requested by the contractor), the bridge deck should be constructed with a minimum of 1/2 in. of additional sacrificial thickness (beyond that already provided for wearing surface considerations), such that corrective grinding can, without question, occur.
- Improve the stiffness compatibility between the bridge superstructure, substructure, approach slab, and supporting materials:
 - Follow the geotechnical recommendations below.
 - Use integral abutments whenever possible and revise the integral abutment details such that the superstructure and piles are rigidly connected (so they rotate and translate as a unit).
 - Consider revising the approach slab to bridge connection detail to provide for rotational fixity.
 - Regardless of the mainline approach type, support the approach slab on a sleeper slab.
- Minimize the frictional resistance between the approach slab and supporting materials by casting the slab on a low-friction material, such as polyethylene sheeting. The use of a friction-reducing material will help to reduce the forces induced on the bridge superstructure and approach slab-to-bridge connection.
- Strive to limit bridge skew to 30 degrees to minimize the magnitude and lateral eccentricity of the longitudinal forces.
- Design the approach slab with sufficient strength to bridge settlement extending from the bridge abutment to the recommended sleeper slab. Furthermore, consider designing the approach slab with stiffness sufficient to minimize any deflection with such settlement.
- Replace the current ODOT approach slab to mainline pavement joint detail with an expansion joint that is sized to accommodate the expected bridge and approach slab expansion and contraction.
- Actively maintain the recommended expansion joint to prevent the development of high stresses in the approach slab and bridge.
- Develop a lab test protocol to determine the bulking moisture content for granular backfill materials and establish a practice to field control the moisture content to avoid bulking moisture contents.
- Consider the use of alternative backfill materials, such as geosynthetic-reinforced soil, geofabric, or flowable fill, as an alternative to collapsible backfill.
- Improve compaction effort with 5 ft of the abutment backfill using thin lifts with a light vibratory compactor. If concerns exist due to compaction equipment being next to the wall, instrument a wall (or walls of different configurations) to monitor stress development and movement during compaction and during service loading to conclusively determine the impact of compaction loading. In general, vibratory compactors should be used to compact granular backfill materials.

- Water drainage needs to be an integral part of the bridge and embankment design. The bridge and embankment need to be detailed to drain water away from the bridge deck, joints, and embankment without causing erosion or changes in the soil properties.

1. GENERAL INFORMATION

1.1. Introduction

Bridge owners have long recognized that the approach pavement at bridges is prone to exhibiting both settlement and cracking, which manifest as the “bump at the end of the bridge.” This deterioration requires considerable on-going maintenance expenditures, added risk to maintenance workers, increased distraction to drivers, reduced steering control, increased damage to vehicles, a negative public perception of the highway system, and a shortened useful bridge life. This problem has recently begun to receive significant national attention as bridge owners have increased the priority of dealing with this recurring problem.

No single factor, in and of itself (individually), leads to significant problems. Rather, it is an interaction between multiple factors that typically leads to problematic conditions. As such, solutions to the problem require interdisciplinary thinking and implementation. The bridge-abutment interface is a highly-complex region and an effective “bump at the end of the bridge” solution must address the structural, geotechnical, hydraulic, and construction engineering disciplines. Various design alternatives, construction practices, and maintenance methods exist to minimize bridge approach settlement, but each has its own drawbacks, such as cost, limited effectiveness, or inconvenience to the public.

1.2. Background

Bridge approach settlement can be caused by a number of factors including: 1.) Seasonal temperature changes causing horizontal movements of abutments; 2.) Loss of backfill material by erosion; 3.) Poor construction practices (e.g., poor joint and drainage system construction, poor compaction of backfill materials, etc.); 4.) Settlement of the foundation soils; 5.) High traffic loads; and 6.) Incompatibility in the vertical system stiffness. The two primary causes reported in the literature are lateral movement of the bridge and settlement of site soils.

Seasonal ambient temperature cycles between summer and winter and the corresponding thermal movements of the bridge superstructure and abutment (in the case of integral bridges) can displace the soil behind the abutment and lead to void development under the approach slab. With water infiltration into the void, erosion and loss of backfill material occurs. To prevent this, researchers and state departments of transportation (DOTs) have recommended various design alternatives, as follows:

- Connect the approach slab to the bridge, reduce the expansion joint widths, and use various alternative joint sealers. An investigation of the practices in 37 states reveals that 30% tie the bridge approach slab to the bridge abutment for integral abutments and about 60% for non-integral abutments. In addition to varying joint

widths, other states use alternative joint sealing materials, which are reportedly effective in preventing water infiltration.

- Use drainable and compressible elastic material to reduce the effects of abutment lateral movement on the surrounding soil.
- Use geosynthetic-reinforced backfill and geotextile-wrapped backfill layers to increase backfill load carrying capacity and reduce erosion. This design creates a stiffer backfill response and can reduce the vertical strain incompatibility between the pile-supported abutment and surrounding soil. Some states use backfill with layers of geosynthetic reinforcement in combination with shallow foundations to support the bridge abutment.
- Use an improved drainage system around the abutment to minimize erosion and void development.

As a result of poor construction practices that may lead to improper placement of steel reinforcement, the failure potential of unreinforced concrete segments in a pavement notch and the bridge end region of an approach slab have also been found to be especially problematic. Generally, it has been found that good inspection and quality control procedures should be followed during the construction of bridge abutments, pavement notches, and approach slabs to minimize the possibility of paving failure.

An inherent incompatibility exists in the vertical support system stiffness at and around bridges. To illustrate this concept, visualize these support conditions as approaching a bridge:

- Far away from the bridge is a natural soil that has only been modified by providing compaction to near-optimum conditions.
- Closer to the bridge are embankment materials that may have been imported from other sites for their desirable characteristics.
- Supporting one end of an approach slab is a sleeper slab with a specially-designed footing. This footing has notably higher vertical stiffness than the soil surrounding it.
- Under the approach slab are select backfill materials that would have been selected for their superior performance characteristics.
- Just adjacent to the bridge abutment are select materials that have not been optimized due to the difficulties in obtaining good compaction.
- Immediately over the bridge abutment, the vertical stiffness approaches infinity due to the deep foundation systems usually used and the desire to minimize bridge settlement.
- Once onto the bridge, the vertical stiffness gradually decreases as approaching midspan, where it gradually increases thereon.

Clearly, the above scenario creates a situation where the vertical stiffness is continually changing. In some cases, the vertical stiffness is radically different very close to one

another. This situation sets the stage for very complex behaviors that can lead to poor ride quality.

1.3. Project Objectives

The objective of this work is to assist the Ohio DOT (ODOT) in the development of pre-construction, construction, and post-construction strategies that will help eliminate or minimize the “bump at the end of the bridge.” Implementation of the details and procedures described herein will provide a tangible benefit to both ODOT and the traveling public in the form of smoother bridge transitions, reduced maintenance costs, and a safer driving environment. The specific objectives of this work are to:

- Develop recommendations and proposed specification changes for design and construction of future bridge approaches.

1.4. Project Scope

A literature review and review of other state approach designs were conducted to find current practices on bridge approach design, construction, and maintenance. An in-depth review of ODOT bridge approach practices was also performed. Field investigations at a number of in-service and under-construction Ohio bridges were conducted to collect quantitative information on the behavior and construction of various bridge types from geographically different locations. In some cases, laboratory tests were conducted on samples taken during the field investigations.

1.5. Report Content

Chapter 2 presents the findings of a formal literature review that was focused on the causes and current solutions for the problem of the “bump at the end of the bridge.” The state of practice for Ohio bridge approach construction, maintenance, and repair is discussed in Chapter 3. Bridge approach state-of-practice descriptions for other states are presented in Chapter 4. An overview of the various in-service bridge testing procedures used in the study is given in Chapter 5. Chapter 5 also includes descriptions of each individual bridge tested, as well as the results of all tests carried out. Chapter 6 contains information about in situ evaluation of new bridge approach backfill materials. Chapter 7 summarizes the results of this work and presents conclusions and recommendations. Although the results of this study, in many cases, are cross-cutting and have ties to multiple aspects of improving bridge-pavement interface ride quality, the following is intended to guide the reader to the results of the various research tasks.

Phase 1 – Pre-Construction and Construction Strategies

- Task 1A – Researching the Department’s Designs: Chapters 3, 5, and 6
- Task 1B – Researching the Department’s Specifications: Chapters 3, 5, 6, and 7
- Task 1C – Researching and Identifying Best Practices: Chapters 2 and 4
- Task 1D – Recommendation of Design and Specification Improvements:
Chapter 7

Phase 2 – Post-Construction Corrective Strategies

Task 2A – Researching the Department’s Corrective Strategies: Chapter 5 plus
personal interaction with ODOT personnel

Task 2B – Researching and Identifying Best Practices: Chapters 2 and 4

Task 2C – Testing Best Practices: Chapter 7

Task 2D – Recommendation of Preferred Corrective Strategy(s): Chapter 7

Task 2E – Implementation of Corrective Strategies

2. GENERAL LITERATURE REVIEW

2.1. Integral Abutments

Hassiotis and Roman (2005) referenced arguments against using approach slabs at bridges. The approach slabs eventually crack in flexure due to the combined effects of soil settlement and traffic compaction. In addition, evaluations of integral abutment designs without approach slabs have shown that regular maintenance of the bridge surface can sufficiently account for any approach roadway settlement. Most states, however, use approach slabs to compensate for embankment settlement and to provide a smooth transition between the bridge and pavement.

Hassiotis and Roman indicate that integral abutment bridges are a cost effective alternative to bridges with complicated moving joints and slide. It wasn't until the 1960s, however, that integral abutment bridges in the US were widely used on the National Interstate Highway system. Eliminating maintenance and the associated cost caused by having deck joints and bearings is the leading advantage of integral abutment bridges. Additional advantages for using integral abutments include:

1. Increased end-span ratios resulting from the elimination of uplift due to dead loads.
2. Increased capacity during seismic events.
3. Integral abutments have reserve live-load capacity. During over-load conditions, the full-depth diaphragm at the bridge ends distribute the load to other girders.
4. Better suited for rapid construction techniques.
5. Integral abutments can simplify bridge replacement. Integral abutments can be constructed behind existing buried foundations due to their smaller footprint.

Although integral abutment bridges are advantageous over the use of traditional bearing-type bridges, they still possess limitations. Bridges with integral abutments must have an approach slab with an expansion joint at the pavement interface to accommodate bridge expansion and contraction. Due to the relative newness of integral abutments, uncertainty also exists on the stresses imposed on the piles and what lengths and skews are acceptable.

Kunin and Alampalli (2000) performed a study of current integral abutment practices in the US and Canada. The authors reported the results of a 1996 survey, of which 31 agencies responded to having experience with integral abutment bridges. In addition, they found that, by 1996, more than 9,770 integral abutment bridges had been built. The popularity of integral abutment bridges stems from the many advantages that they offer (Brena et al., 2007; Burke, 1993; Lawver et al., 2000; Kunin and Alampalli, 2000). Both initial construction cost and long-term maintenance cost is the biggest benefit derived from integral abutment designs, due to the elimination of expansion joints and bearings. Generally, integral abutment bridges experience less deterioration from de-icing chemicals and snowplows, decreased impact loads, improved ride quality, simpler construction, and improved structural resistance to seismic events. Burke (1993)

concludes that integral abutment bridges should be used whenever applicable, because of the many advantages over the few disadvantages.

2.2. Approach Slabs

An evaluation of current practices found that approach slabs were generally considered to be successful when good pavement joints were used and when the slab was designed to prevent cracking (Dupont and Allen, 2002). With improved backfill materials and construction practices, integral abutments were cited by the authors as being the best performing abutment type. They also found that embankment design and construction varied greatly between states. They also found that some states have implemented specifications pertaining to backfill compaction and material selection limits; however, others seem to believe the solution to ride quality issues is the approach slab. Several states reported having drainage or erosion issues at the abutment, stemming from inadequate drainage provisions.

Through finite element research, Cai et al. (2005B) evaluated different strength scenarios for various lengths of flat approach slabs and ribbed approach slabs. For the conventional Louisiana Department of Transportation and Development (LA DOTD) 20 ft long 12 in. thick approach slab, Cai et al. determined that the use of #7@6 in. is required for the bottom mat of reinforcing to develop strength sufficient to bridge any settlement amount. Slabs with longer lengths of 40 ft or 60 ft need increased depths and reinforcing to meet the American Association of State Highway and Transportation Officials (AASHTO) strength requirements. Table 2.1 shows the slab thickness and reinforcement requirements for 40 ft and 60 ft slab lengths with various thickness and embankment settlements.

Ribbed approach slabs were evaluated both as pre-stressed and as regular reinforced concrete beams with varying rib spacing (12 ft, 16 ft, and 32 ft) for slab lengths of 60 ft and 80 ft. As expected, the required pre-stressing in the ribs was found to increase as the settlement increases. However, when settlement exceeds 3 in. the 32 ft rib spacing does not work due to exceeding the allowable number of strands. For cases where settlement is larger than 3 in., a rib spacing of 16 ft and 12 ft can provide adequate capacity. Although pre-stressed ribs create an efficient cross-section, the constructability is a major disadvantage. The most economical cast-in-place reinforced concrete rib approach slab is shown in Figure 2.1. The slabs shown in Figure 2.1 meet strength and design requirements for both AASHTO Standard and Load and Resistance Factor Design (LRFD) Specifications.

2.3. Approach Slab to Bridge Interface

Failure mechanisms for the interface of the paving notch and approach slab connection were investigated using finite element analysis by Cai et al. They found the large stresses occur at the connection of the approach slab due to large embankment settlements (e.g. 6 in. for 40 ft slab). This can cause damage to the dowel bars, crushing of the concrete

bearing, and abutment cracking. Cai et al. suggest the use of an inclined bar, which allows rotation of the approach slab, but prevents differential longitudinal movement.

Table 2.1. Reinforcement ratio of slab under different settlement
($f'_c=4,000$ psi, $f_y=60,000$ psi) (Cai et al.)

Differential settlement (in)	40-ft. Slab		60-ft. Slab	
	ρ for thickness of 18 in.	ρ for thickness of 24 in.	ρ for thickness of 21 in.	ρ for thickness of 27 in.
0	0.0025 #7@16" ($\rho = 0.0025$)	0.0014 ($\rho_{min}=0.0018$) #6@11" ($\rho = 0.0019$)	0.0035 #6@7" ($\rho = 0.0035$)	0.0022 #6@8" ($\rho = 0.0023$)
0.6	0.0081 #8@6" ($\rho = 0.0088$)	0.0058 #8@6.5" ($\rho = 0.0058$)	0.0060 #8@7" ($\rho = 0.0062$)	0.0046 #8@7" ($\rho = 0.0047$)
1.2	0.0114 #9@5.5" ($\rho = 0.0121$)	0.0074 #10@8" ($\rho = 0.0076$)	0.0083 #9@6.5" ($\rho = 0.0085$)	0.0065 #9@6.5" ($\rho = 0.0065$)
2.4	0.0143 #10@6.0" ($\rho = 0.0141$)	0.0080 #10@8.5" ($\rho = 0.0081$)	0.0121 #10@5.5" ($\rho = 0.0128$)	0.0093 #10@5.5" ($\rho = 0.0096$)
3.6	0.0151 #10@5.5" ($\rho = 0.0154$)	0.0081 #10@8.5" ($\rho = 0.0081$)	0.0151 #10@4.5" ($\rho = 0.0156$)	0.0110 #10@4.5" ($\rho = 0.0117$)
4.8	0.0153 #10@5.5" ($\rho = 0.0154$)	0.0081 #10@8.5" ($\rho = 0.0081$)	0.0174 #10@4" ($\rho = 0.0176$)	0.0120 #10@4" ($\rho = 0.0132$)
6	0.0154 #10@5.5" ($\rho = 0.0154$)	0.0081 #10@8.5" ($\rho = 0.0081$)	0.0191 #10@3.5" ($\rho = 0.0202$)	0.0126 #10@4" ($\rho = 0.0132$)
7.2	0.0154 #10@5.5" ($\rho = 0.0154$)	0.0081 #10@8.5" ($\rho = 0.0081$)	0.0204 #10@3.5" ($\rho = 0.0202$)	0.0130 #10@4" ($\rho = 0.0132$)

Design of reinforced beam (adopted new design)
 $(f'_c = 4000 \text{ psi}, f_s = 60,000 \text{ psi})$

Differential Settlement (in)	60-ft. Span		80-ft. Span	
	Interior Beam Spaced at 16 ft.	Interior Beam Spaced at 12 ft.	Interior Beam Spaced at 16 ft.	Interior Beam Spaced at 12 ft.
	# of #10 Bars	# of #10 Bars	# of #11 Bars	# of #11 Bars
0	2	2	2	2
0.5	7	6	6	5
1	8	7	8	7
2	12	10	10	9
3	15	13	13	11
6	15	13	17	13

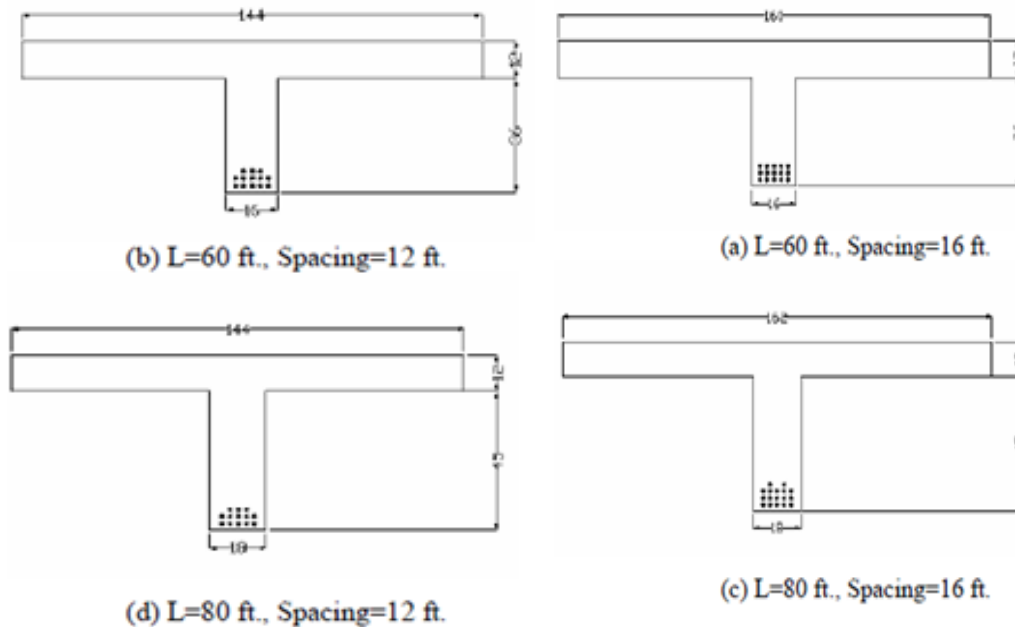


Figure 2.1. Long approach slab T shape and reinforcement (Cai et al., 2005B)

Kunin and Alampalli (2000) found two main approach slab-to-bridge connection construction techniques. The first is to connect the slab reinforcement to the bridge through extension of the deck steel (See Figure 2.2). The second technique uses reinforcing steel to connect the slab to the corbel or abutment (See Figure 2.3). Another option cited by Kunin and Alampalli is to have the approach slab simply rest on the paving notch (See Figure 2.4). Hoppe (1999) reports that 71% of the state DOTs using integral abutment bridges use a mechanical connection between the approach slab and bridge.

A more recent survey conducted by Maruri and Petro (2005) found practices similar to those found by Kunin and Alampalli. Maruri and Petro suggest that standardization and guidelines would be beneficial for abutment/approach slab connections. They also found that 31% of the respondents use sleeper slabs, 26% place the slab directly on the fill, and 30% do both.

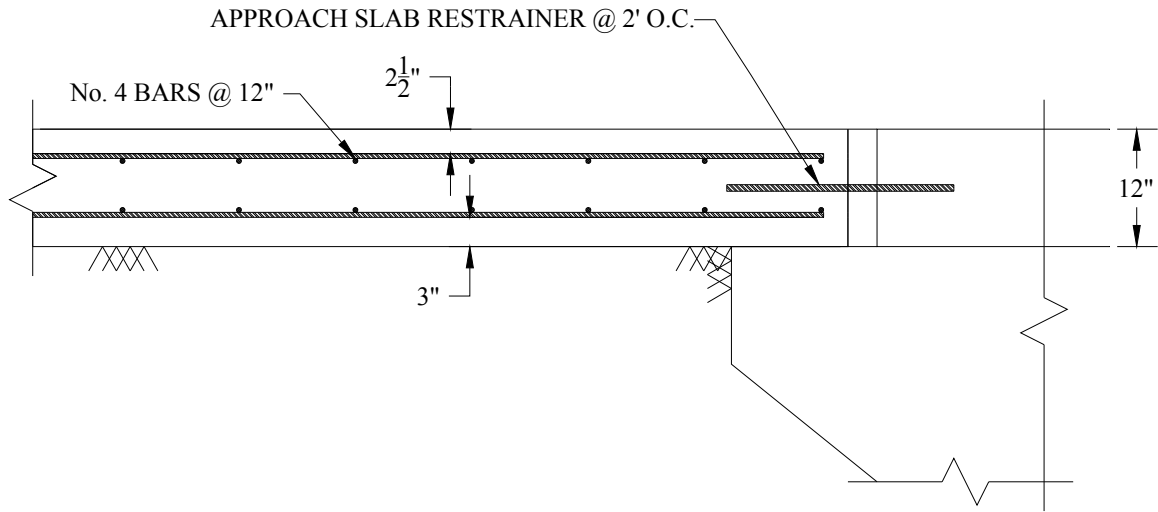


Figure 2.2. Deck steel extension connection (Greimann et al., 2008)

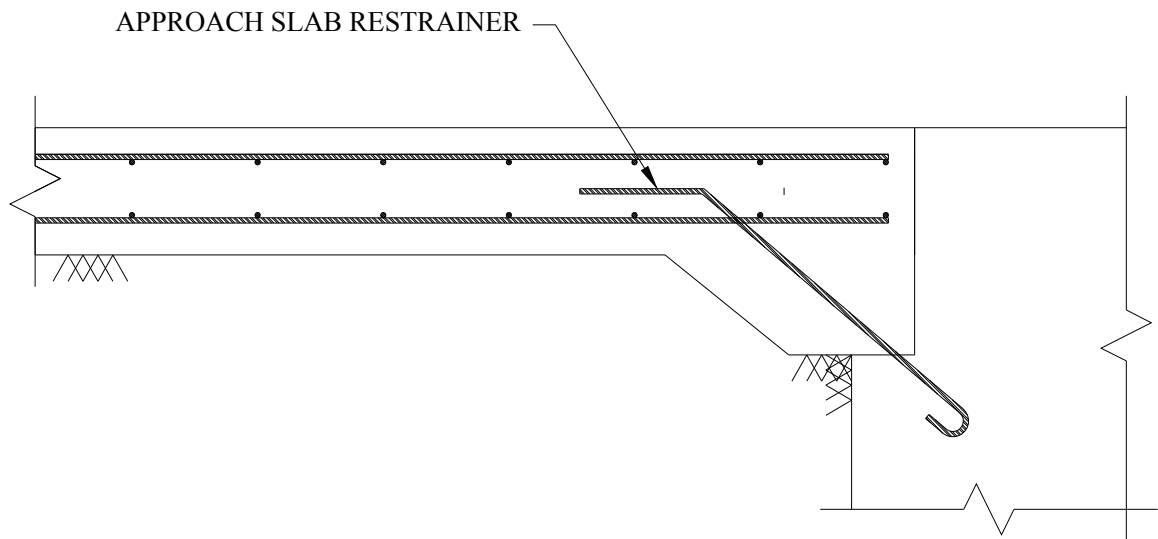


Figure 2.3. Abutment steel connection (Greimann et al., 2008)

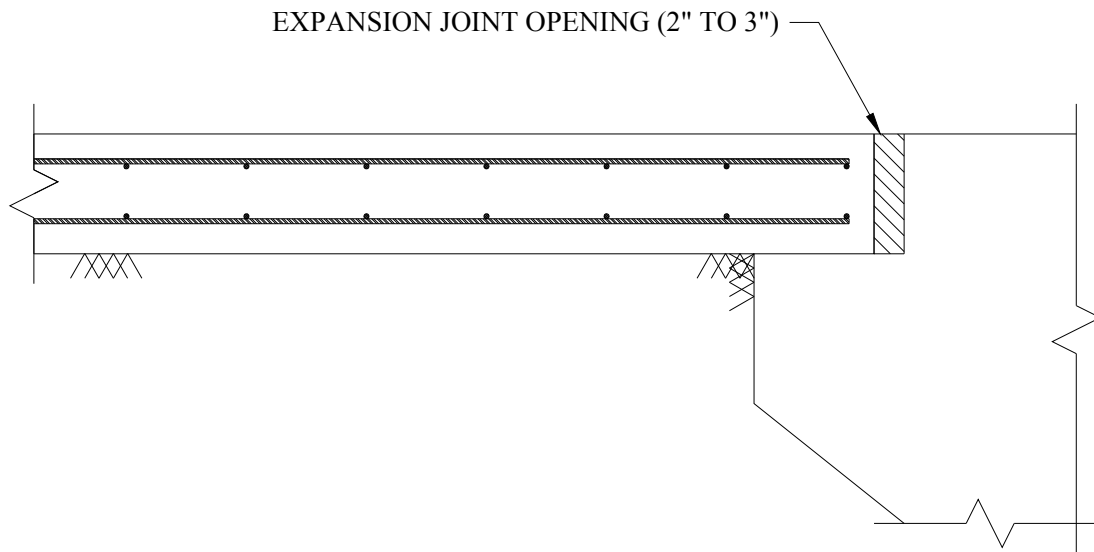


Figure 2.4. Abutment with no connection (Greimann et al., 2008)

Burke (1993) indicates “full width approach slabs should be provided for most integral abutments and should be tied to the bridge to avoid being shoved off their seat by the horizontal cycle action of the bridge as it responds to daily temperature changes.” Burke also indicates, with regards to approach slab to bridge connections, “approach slabs tied to bridges become part of the bridge, responding to moisture and temperature changes. They increase the overall structure length and require cycle control joints with greater ranges.” The cycle control joints are important, because they relieve resistance pressures that are a result of the lengthening/shortening of the bridge. As the bridge moves, it is resisted by the approach slab in the form of a pressure. That pressure is distributed to both the slab and the bridge, but is a much greater problem for the pavement, which has a smaller area. As a result, fracturing and buckling (e.g., blowouts) can occur in the approach pavement. Therefore, cycle control joints must be designed and used. Burke also suggests another method to minimize the force required to move the approach slabs: “They should be cast on smooth, low-friction surfaces such as polyethylene or filter fabric.”

Similar to the above, Mistry (2005) recommends the following with regard to approach slabs:

- Make installation of the approach slab a joint decision between the Bridge/Structures group and the Geotechnical group.
- Standardize the practice of using sleeper slabs, as cracking and settlement typically develop at the slab/pavement joint.
- Use well-drained granular backfill to accommodate the expansion/contraction.
- Tie approach slabs to abutments with hinge-type reinforcing.
- Provide layers of polyethylene sheets or fabric under approach slabs to minimize friction against horizontal movement.

- Limit skew to less than 30 degrees to minimize the magnitude and lateral eccentricity of longitudinal forces.

The above recommendations reinforce the use of proper backfill and friction-reducing material under the approach slab. More importantly, Mistry's recommendations reinforce the importance of integrally connecting approach slabs to the bridge.

2.4. Embankment Design and Construction

The Kentucky Transportation Center (KTC) conducted an extensive literature summary and a survey of state bump problems and their current practices (Dupont and Allen, 2002). The embankment foundation was found to be one of the most significant factors for the occurrence of bridge approach settlement. The foundation problems were generally found to occur when embankments were constructed on compressible cohesive soils. The settlement was generally found to occur in two phases. The initial settlement phase occurs almost instantaneously when a load is applied on the soil. Primary settlement is caused by the gradual escape of water from voids. The primary phase generally makes up the largest percentage of the total settlement. This settlement can take several years to occur in certain cohesive soils. The final settlement, called secondary, is the readjustment of soil and water particles within the foundation soil that are continuously loaded. With highly organic or very soft clays, the secondary settlement can be as large as the primary settlement.

In a literature review and survey of various state DOTs, Briaud et al. (1997) summarized causes of the bump and offered potential solutions. According to the report, “the bump develops when there is a differential settlement or movements between the bridge abutment and the pavement of the approach embankment.” This problem was estimated to impact 25% of the bridges in the country. Three main causes for the bump can be taken from Briaud’s report. Figure 2.5 conceptually shows these causes:

1. Differential settlement between the top of the embankment and the abutment due to the different loads on the natural soil and compression of embankment soils, typically because of insufficient compaction.
2. Void development under the pavement due to erosion of embankment fill because of poor drainage.
3. Abutment displacement due to pavement growth, embankment slope instability, and temperature cycles on integral abutments.

While the above items seem to suggest the problem is geotechnical and construction-related in nature, a structural issue is actually present. Integral abutment bridges are called out as a distinct issue, with “many engineers responding to the survey believing the bump worsens with integral abutment bridges” (Briaud et al., 1997). Thermal cycles are a key behavior with integral abutment bridges, because they do not have expansion joints and expand and contract with the thermal cycles. When integral abutment bridges

expand, the fill material is compacted, creating a void that increases when the bridge contracts.

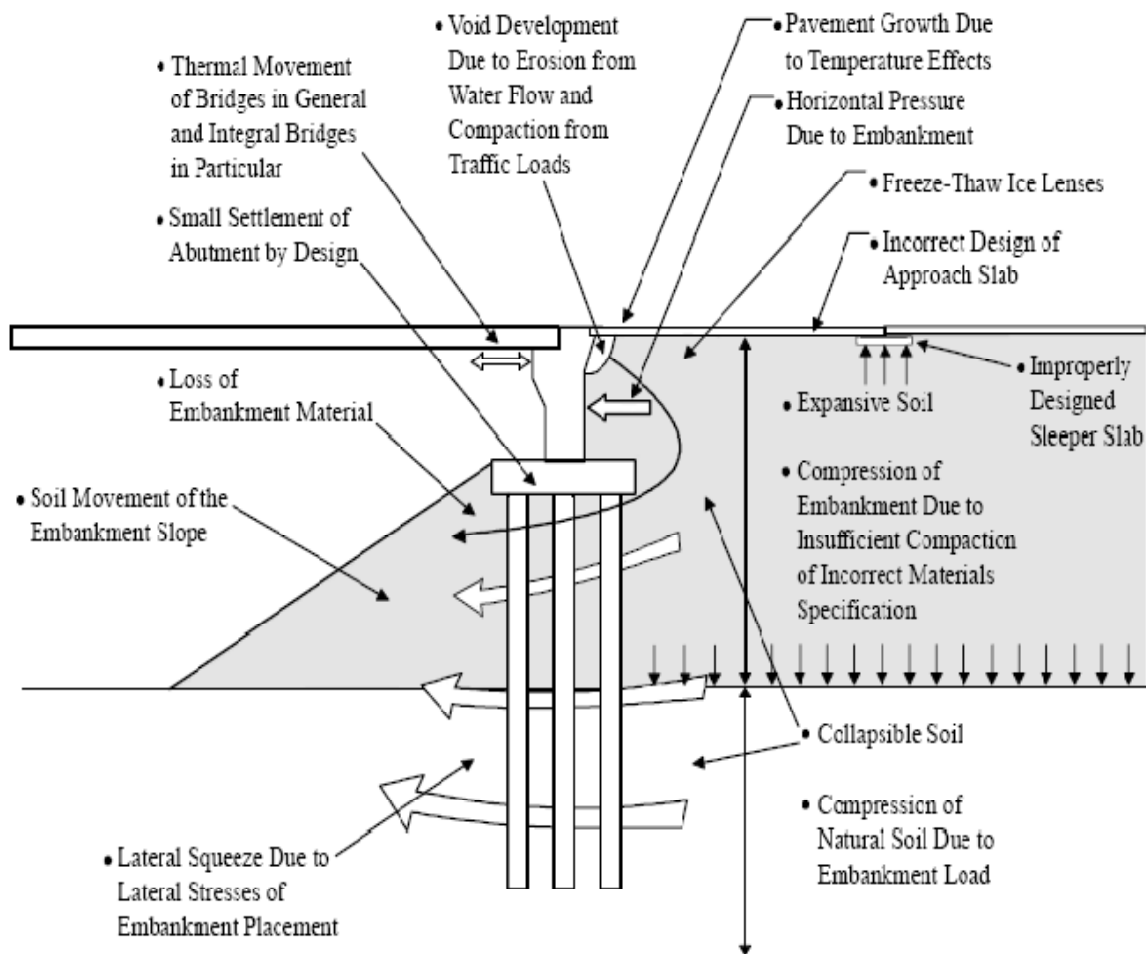


Figure 2.5. Problems leading to the formation of the bump (Briaud et al., 1997)

Schaefer and Koch (1992) conducted a model study consisting of small two girder integral abutment test bridges located near the South Dakota DOT (SDDOT) office near Brookings. The bridge was constructed so hydraulic jacks could push and pull the abutment to simulate thermal expansion and contraction of the bridge girders. The model study indicated the void space below the approach slab is a direct result of thermal-induced movement of the integral abutment system. In addition, large earth pressures were measured in the backfill, large longitudinal movements were measured in the backfill, lateral movement of the backfill occurred, cracks developed in the approach embankment, and the approach slab was pushed upward. Effects of the abutment movement in the model study corresponded to the observations made during a complimentary field study. Figure 2.6 shows the movement of typical integral abutment bridges.

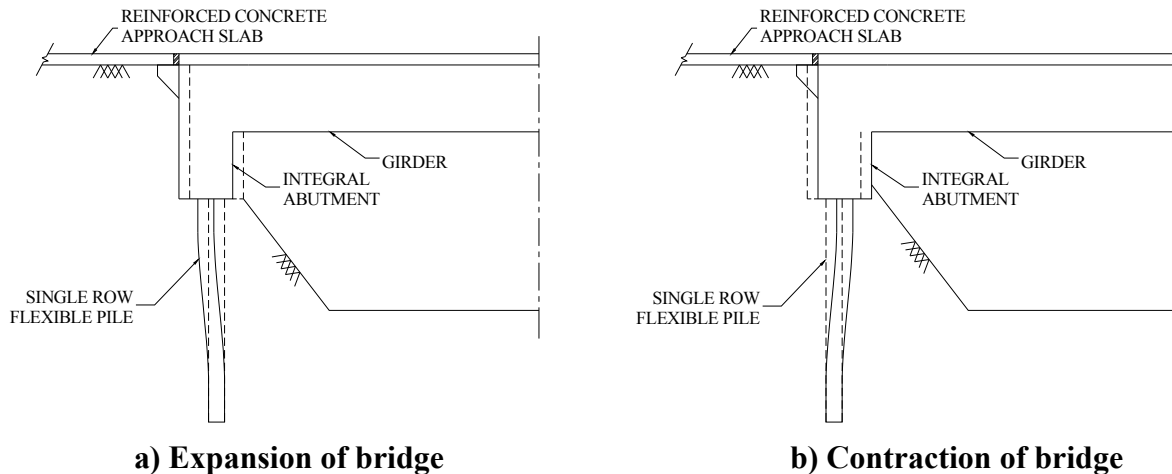


Figure 2.6. Temperature induced movement of an integral abutment bridge (Greimann et al., 2008)

Reid et al. (1999) studied if placing the backfill behind an integral abutment with a vertical gap between the backfill and abutment would reduce the void under the approach slab. A field study was constructed with a geotextile-reinforced wall behind the integral abutments. The study found that a 6 in. gap between the abutment and geotextile wall is adequate to prevent passive pressures on the reinforced soil. The placement of a gap and geotextile-reinforced soil behind the abutment did reduce the development of voids under the approach slab unlike the abutment. The soil at the wing wall was not constructed with a gap. As a result, the cyclic movement of the wing walls led to erosion of the wing wall soils into the gap behind the abutment. Reid et al. indicate that, over time, the gap at the abutment will silt in, causing conditions where no gap exists and lead to the development of voids.

A model test (Reid et al., 1999), similar to the one conducted by Schaefer (1992), was also conducted to investigate alternative backfill designs to reduce void development under approach slabs. The use of a vertical layer of rubber tire chips behind the integral abutment was not only found to reduce the passive earth pressure on the retained fill, but also reduced the development of voids under the approach slab. The study did find, however, that after several abutment movement cycles, the rubber tire chip layer rearranged and consolidated causing movement of the adjacent soil, once again resulting in void development.

Briaud et al. (1997) also give several recommendations for best practices associated with minimizing bridge approach rideability issues. The recommendations are:

1. Make the bump a design issue with prevention as the goal.
2. Assign the design issue to an engineer.
3. Encourage teamwork and open-mindedness between geotechnical, structural, pavement, construction, and maintenance engineers.
4. Carry out proper settlement vs. time calculations.
5. Design an approach pavement slab for excessive settlement.

6. Provide for expansion/contraction between the structure and the approach roadway.
7. Design a proper drainage and erosion protection system.
8. Use and enforce proper specifications.
9. Choose knowledgeable inspectors, particularly on geotechnical aspects.
10. Perform inspections including joints, grade specifications, and drainage.

Dupont and Allen (2002) provided recommendations for design and construction practices that should be taken to best prevent the bump problem altogether:

- Lower the approach slabs to allow for an asphalt overlay riding surface. By designing the approach slab to have an overlay allows for a smoother transition and makes future maintenance easier.
- Require surcharge and settlement periods prior to construction to reduce the amount of primary foundation settlement.
- Design maintenance plans concurrent with construction plans. Many states believe the bump is a bridge issue that cannot be eliminated completely and must be scheduled into the life of the bridge.
- Have specifications that require select fill be placed adjacent to abutments.
- Eliminate erosion near abutments and on approach slopes by designing adequate drainage.
- Implement bridge approach warranties for newly constructed bridges. This could be a difficult approach to sell; however, it could cause better teamwork, better review of drawings and specifications, and more input on design alternatives.
- Reduce the side slope embankments, which are more resistant to settlement and lateral movement of both the foundation and embankment.
- Improve approach slab design by providing longer slabs with stronger reinforcement. By providing longer slabs, the slope of the slab caused by settlement is decreased.

2.5. Abutment Slabs

One aspect associated with ride quality issues found to have very few guidelines or code specifications is the structural design of approach slabs. However, Cai et al. (2005A) have conducted research for LA DOTD on the approach slab performance under a given embankment settlement and developed design aids for structural evaluation and design. The researchers developed a three dimensional (3D) finite element model (FEM) of the approach slab and embankment and performed a parametric study to examine the interaction of variable embankment settlements and the performance of the approach slab. With a 40 ft approach slab, it was found that as the settlement increased the deflection and internal moments increased until a settlement of approximately 6 in. was reached. After 6 in. of settlement, the slab was found to perform as a simply supported beam having no contact with the soil. Cai et al. (2005A) also found that, as the slab moved from being uniformly supported to simply supported due to embankment settlement, more stress was applied to the sleeper slab and in turn increased the stress in

the soil. The soil stresses were seen to increase even after the slab became simply supported, due to the geometry of the soil around the sleeper slab changing. From the results of the finite element study, the research team determined that the LA DOTD approach slab design was only good for 0 to 6 in. of settlement. Design coefficients were developed corresponding to the simple beam moment that can be used to design the slab reinforcement for a given settlement. Even with improved structural design and long term performance of the approach slab, Cai et al. believe that the magnitude of the bump is still a function of the total settlement. A more rigid approach slab will decrease the change in slope of the slab; however, it may also increase the faulting deflection caused by increased soil pressure beneath the sleeper slab.

KTC determined that the greatest expenses associated with repairing bridge approach problems goes toward placing asphalt wedges, asphalt overlays, mud-jacking, or replacing the approach slab (Dupont and Allen, 2002). Each option has a varying cost, ranging from under \$1,000 for the asphalt wedges to more than \$10,000 to replace an approach slab. The longevity of each repair also varies from being temporary to five years or more for a new approach slab.

3. OHIO BRIDGE DESIGN AND CONSTRUCTION STATE OF PRACTICE

3.1. Standard Bridge Drawings and Bridge Design Manual

ODOT Office of Structural Engineering Standard Bridge Drawings are followed or used whenever it is practical. The bridge case studies investigated either are referenced to or have similar details to these standards. The ODOT Bridge Design Manual provides preliminary design information with descriptive parameters and guidelines for selecting various options during bridge design. Some of the details that may be important to understanding how current bridge design standards impact ride quality are described below.

3.1.1. Typical Abutment Details

Figure 3.1 shows the typical detail used at simply supported girders. The abutment has a back wall extending upward from the girder bearing to prevent the backfill material from coming in contact with the superstructure. The top of the back wall has a 6 in. inset for the approach slab bearing. According to the Bridge Design Manual, this abutment type should be used only when integral or semi-integral abutments cannot be used, due to either cost or length limitations. On the back side of the abutment, a vertical 2 ft wide section of porous backfill, wrapped in filter fabric, is constricted to facilitate water drainage.

The typical ODOT integral abutment is shown in Figure 3.2. The design manual states the integral abutment must be placed on a flexible abutment to accommodate longitudinal movements. The integral abutment can be used for bridges with lengths up to 400 ft and skews of less than 30 degrees. The superstructure is connected to the abutment by two opposing diagonal #6 bars that cross at the centerline of the beam bearing. Figure 3.3 shows an enlarged view of the connection. The connection of the superstructure to the abutment is different from that of most other DOTs. Many other DOTs use vertical reinforcing bars located near the outside faces of the structure. The two diagonal bars and expansion joint material allow a hinge to form at the interface between the abutment and the superstructure. Many of the bridges investigated and summarized herein had this type of abutment detail.

The semi-integral abutment detail is shown in Figure 3.4. The detail is similar to the integral abutment detail; however, the girders rest on an elastomeric bearing pad and no reinforcing is used at the joint. The support for the semi-integral abutment is rigid and does not allow longitudinal movement.

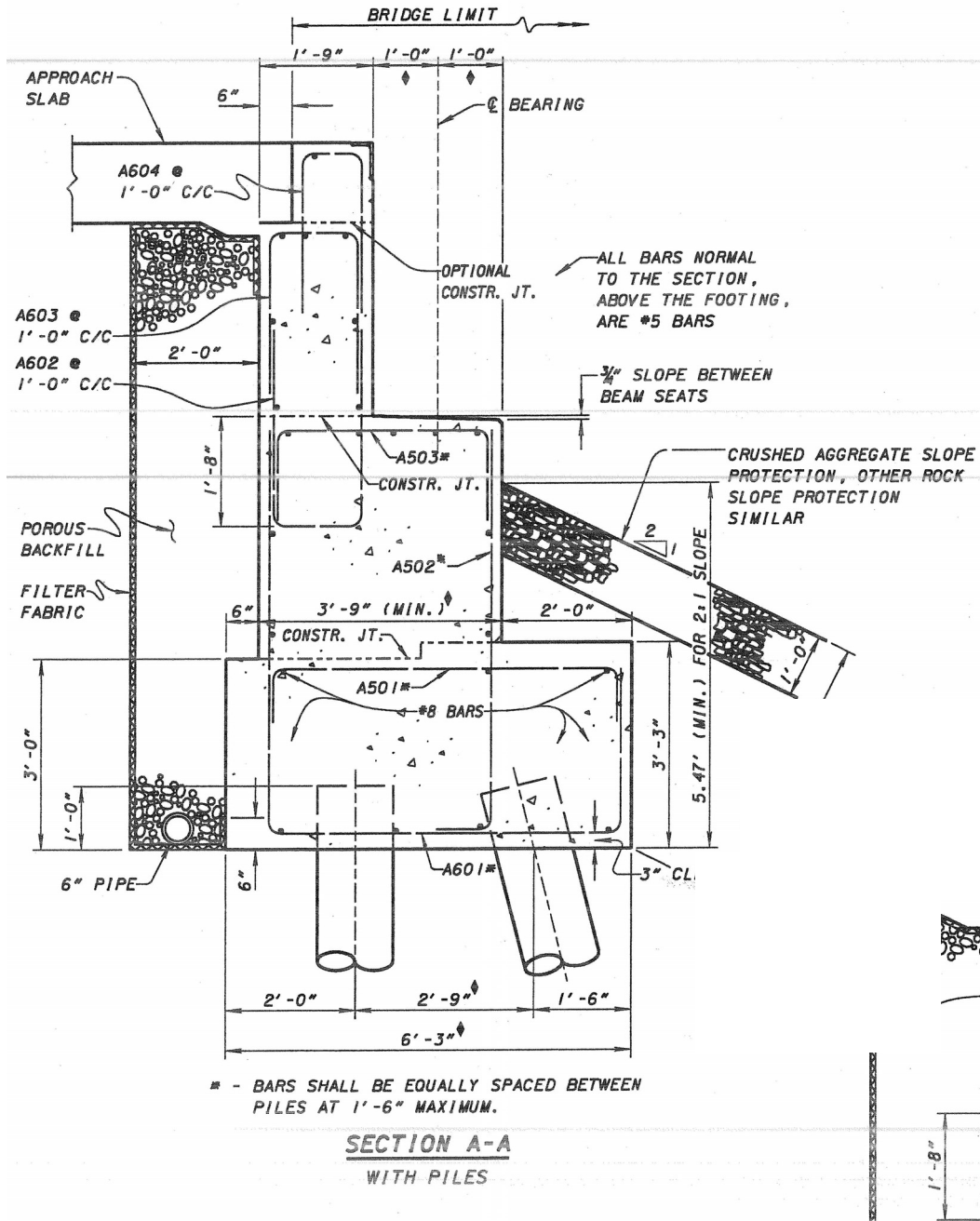


Figure 3.1. Typical abutment detail for simply supported girders (ODOT, 2009)

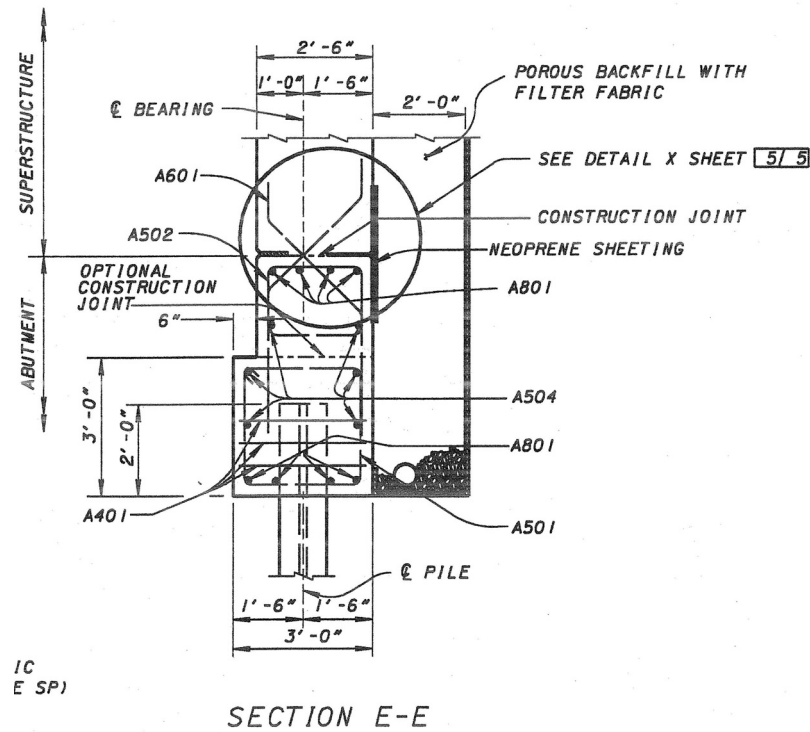


Figure 3.2. Integral abutment detail (ODOT, 2009)

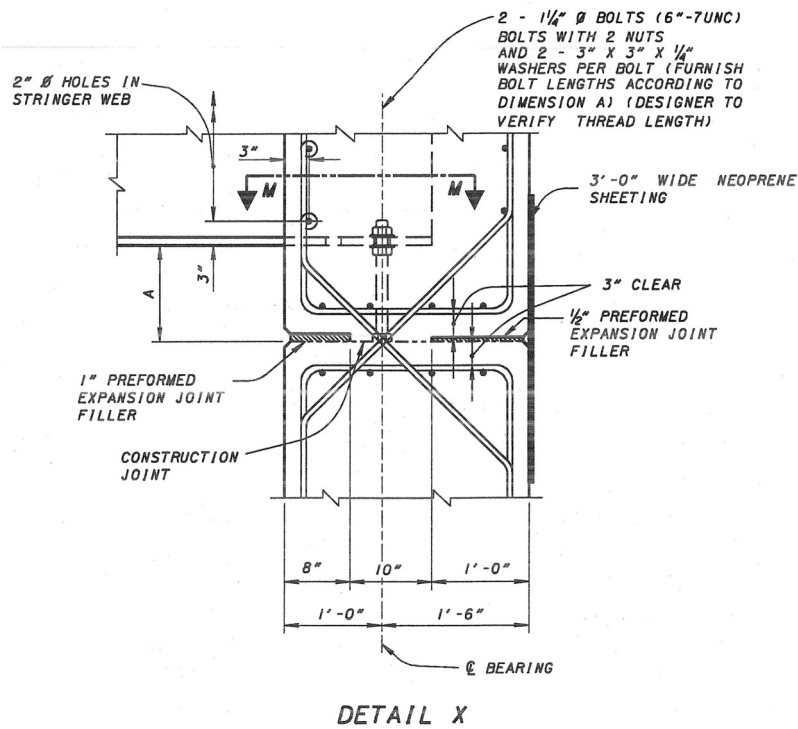


Figure 3.3. Integral abutment connection to pile cap beam (ODOT, 2009)

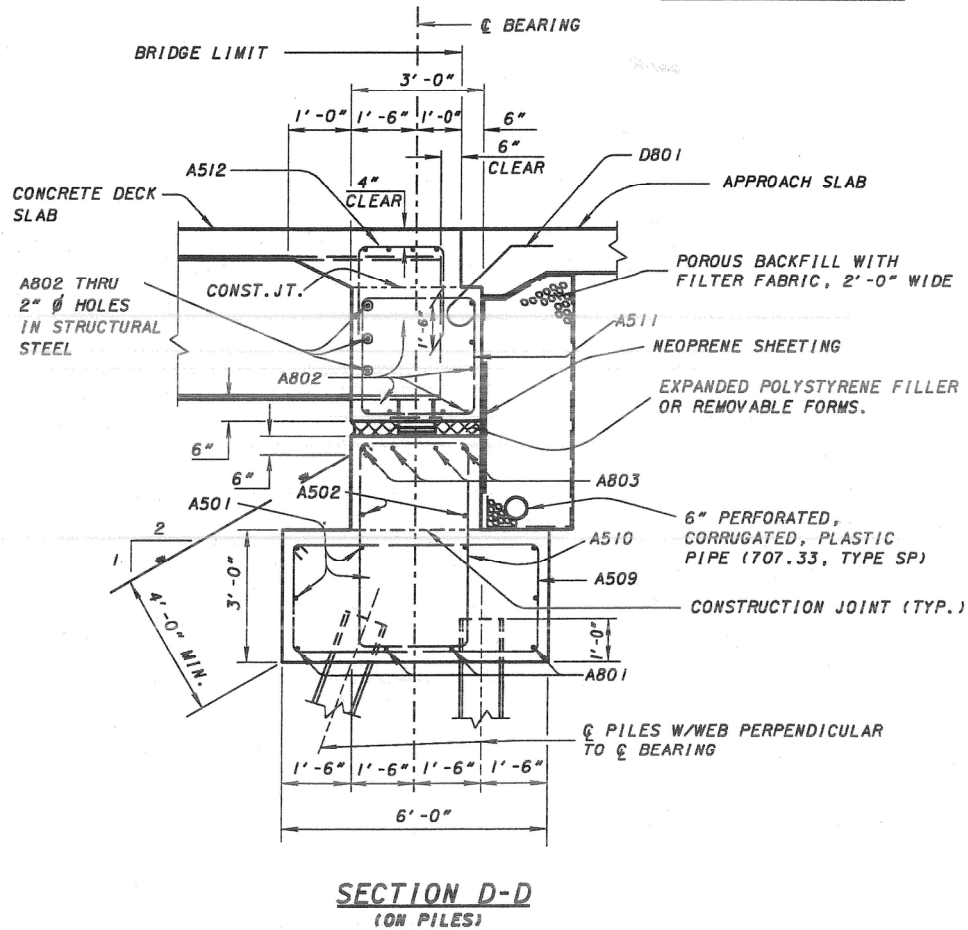


Figure 3.4. Semi-integral abutment configuration (ODOT, 2009)

3.1.2. Typical Approach Slab Details

The typical reinforced concrete approach slab details are shown in Figure 3.5. The approach slab is connected to the bridge back wall or abutment with an angled # 8 hooked bar. For the bridges investigated in this work, the typical length of the approach slab was 25 ft. The design guide states that for four-lane divided highways built on new embankments, the minimum approach length shall be 25 ft and for structures with mechanically stabilized earth (MSE) walls, a minimum of 30 ft should be used. A formula for determining the length of the approach slab is presented in the Bridge Design Manual and shown in equation 3.1.

$$L = [1.5(H + h + 1.5)] / \cos \theta \leq 30 \text{ ft} \quad (3.1)$$

where:

L = Length of the approach slab measured along the centerline of the roadway rounded up to the nearest 5 ft

H = Height of the embankment measured from the bottom of the footing to the bottom of the approach slab

h = Width of the footing heel

θ = Skew angle

At the joint of the abutment and approach slab, there are several different standard joint detail configurations, as shown in Figures 3.6 and 3.7. Although six different options are presented in the standard details, several of the bridges investigated here used a continuous pour of the approach slab with the bridge deck. The approach slab to the deck joint location is then saw cut to allow independent movement. The continuous pour method was implemented to eliminate the lip typically caused when stopping the concrete screed just short of the joint.

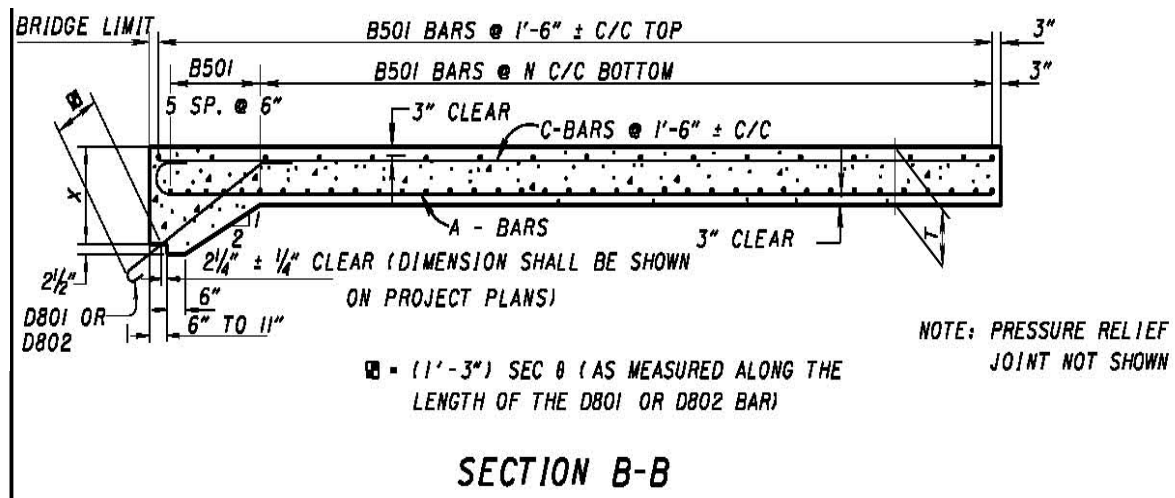


Figure 3.5. Typical concrete approach slab (ODOT, 2009)

Figures 3.8 through 3.10 show different configurations of compression and strip seal expansion joints. The use of the expansion joint generally occurs with simply supported superstructures with abutment designs similar to Figure 3.1. The expansion joint is located between the bridge deck and the backwall to allow for longitudinal movement of the bridge.

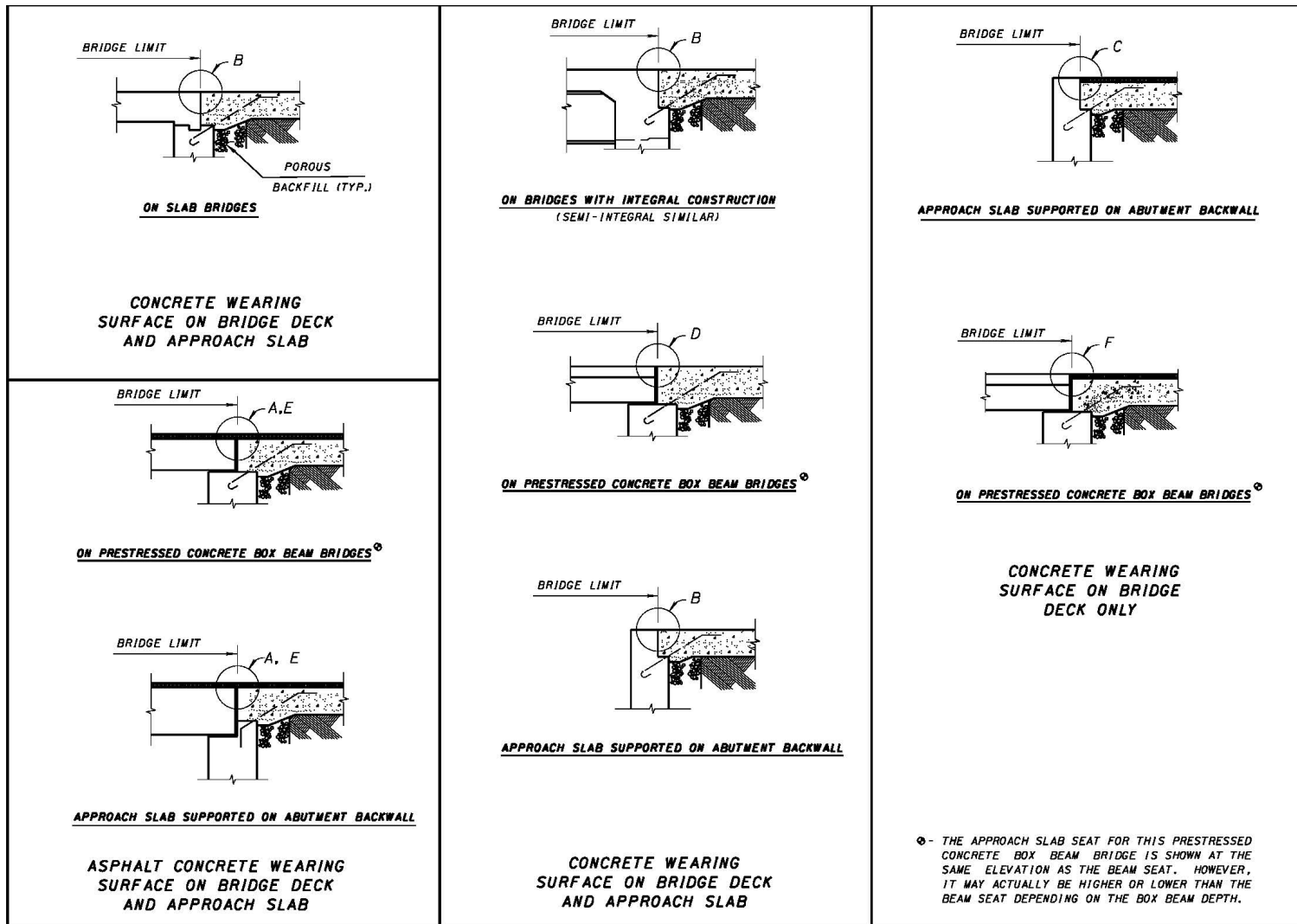
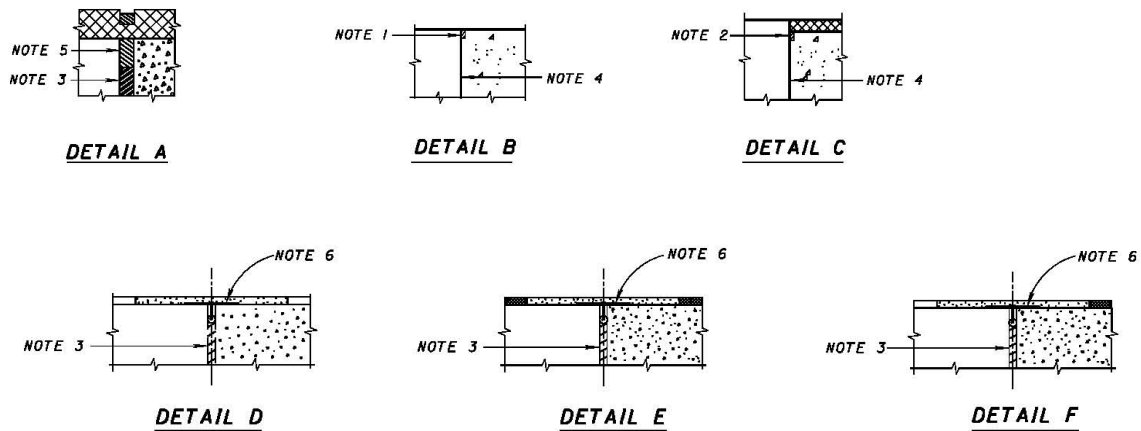


Figure 3.6. Abutment to approach slab joint detail (ODOT, 2009)



NOTE 1: PREFORMED ELASTOMERIC COMPRESSION JOINT SEAL, 705.11 (1 $\frac{1}{4}$ " WIDE FOR A $\frac{1}{2}$ " WIDE GROOVE) PLACED IN $\frac{1}{2}$ " x $\frac{3}{4}$ " GROOVE.

NOTE 2: PREFORMED ELASTOMERIC COMPRESSION JOINT SEAL, 705.11 (1 $\frac{1}{4}$ " WIDE FOR A $\frac{1}{2}$ " WIDE GROOVE) PLACED IN $\frac{1}{2}$ " x $\frac{2}{8}$ " GROOVE.

NOTE 3: 1" PREFORMED EXPANSION JOINT FILLER, 705.03.

NOTE 4: TYPE "A" WATERPROOFING.

NOTE 5: SEE PLAN INSERT SHEET, "ABUTMENT JOINTS IN BITUMINOUS CONCRETE, BOX BEAM BRIDGES."

NOTE 6: SEE PLAN INSERT SHEET, "POLYMER MODIFIED ASPHALT EXPANSION JOINT SYSTEM."

Figure 3.7. Abutment to approach slab joint detail (cont.) (ODOT, 2009)

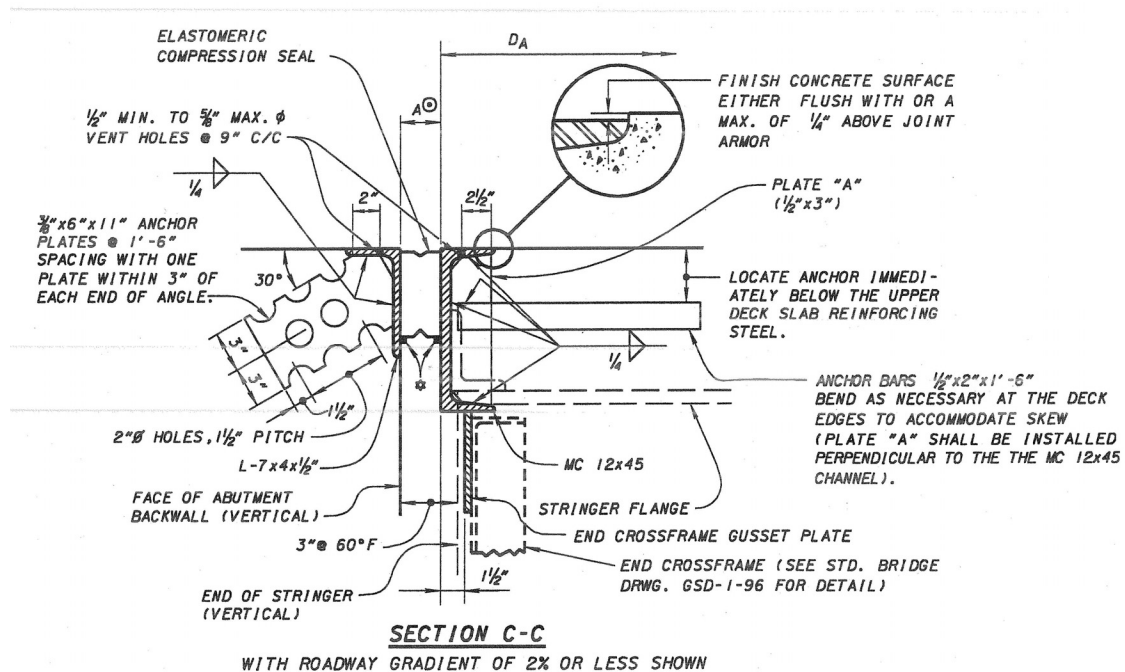


Figure 3.8. Compression seal expansion joint for steel girder bridge (ODOT, 2009)

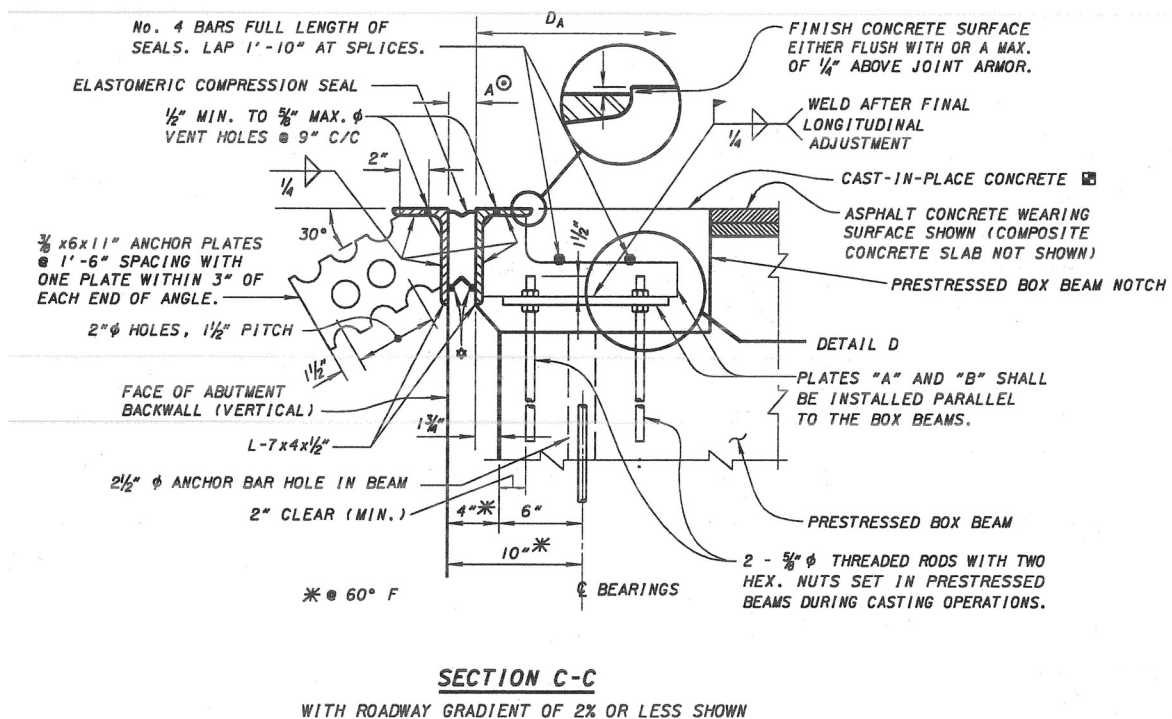


Figure 3.9. Compression seal expansion joint for concrete box girder (ODOT, 2009)

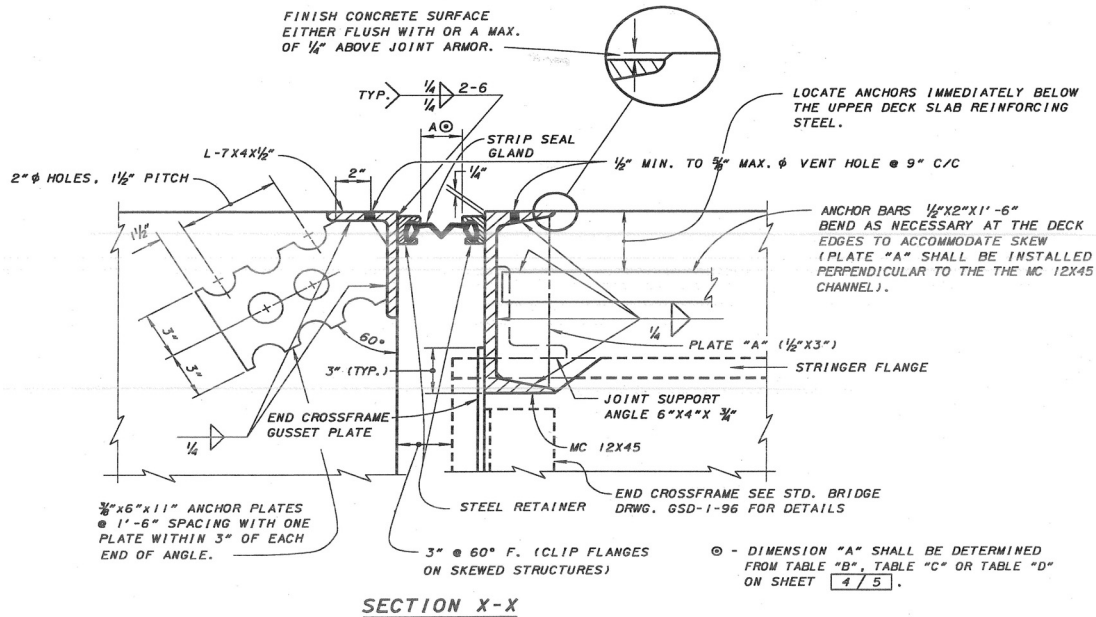


Figure 3.10. Strip seal expansion joint detail for steel girder bridges (ODOT, 2009)

In addition to the Strand Bridge Drawings, the Office of Pavement Engineering has standard pavement drawings that detail the transition between the approach pavement and the mainline pavement. Currently, when asphalt concrete pavements are used, the asphalt is butted directly up against the face of the approach slab. When concrete pavement is used, a pressure relief joint, shown in Figure 3.11, is used at the end of the approach slab or in the mainline pavement within 100 ft of the approach slab end. The 4 ft asphalt joint is placed on a sleeper slab and is used as an expansion joint for the bridge, approach, and pavement movement. In many cases, this method of creating an expansion joint was explained to cause an extra bump prior to reaching the approach slab, which compounds the bump at the bridge.

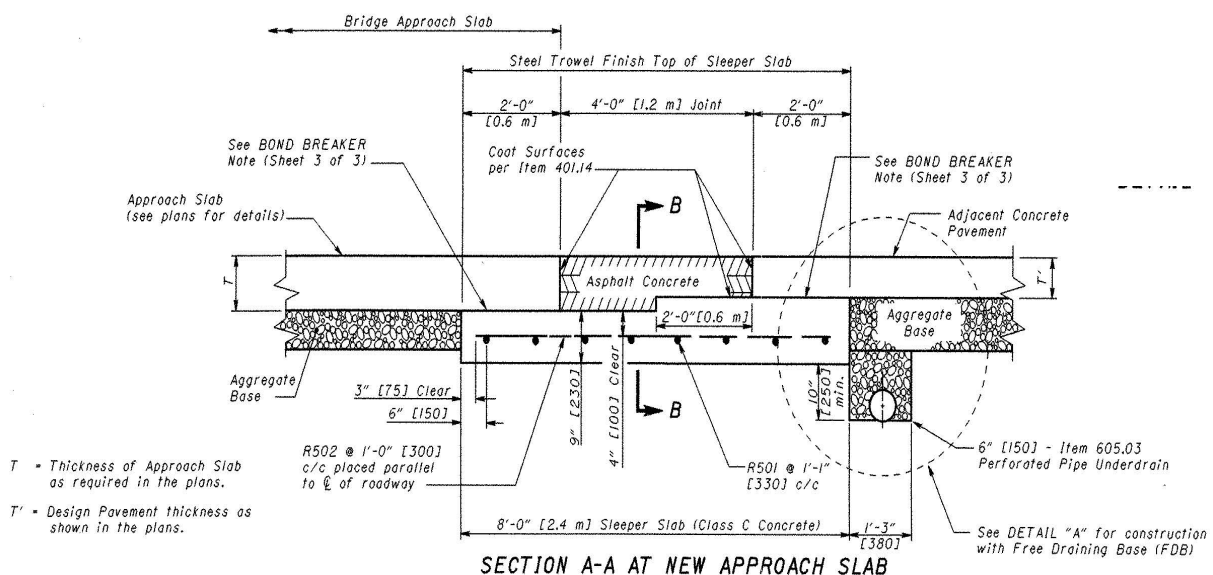


Figure 3.11. Pressure relief joint (ODOT, 2009)

3.1.3. Embankment Material

The ODOT Construction and Material Specifications (ODOT, 2008) has three basic embankment material specifications: Item 203 Embankment, Item 203 Granular Embankment, and Item 203 Granular Materials Type A, B, C, D, E, or F. The Item 203 Embankment allows natural soil, natural granular material, granular material types, slag material, brick shale, rock, random material, reclaimed asphalt concrete pavement (RACP), recycled Portland cement concrete (RPCC), or petroleum contaminated soil (PCS) to be used for embankment construction. When Item 203 Granular Embankment is specified, the material must fall under specification 703.16B or 703.16C. Under specification 703.16B, the Granular Embankment can be natural granular materials that include broken or crushed rock, gravel, sand, durable siltstone, and durable sandstone that can be placed in 8 in. lifts. The listed material must also be classified as Department Group Classification A-1-a, A-1-b, A-3, A-3-a, A-2-4, A-2-6, or A-2-7. Under Specification 703.16C the Granular Embankment is allowed to be crushed carbonate stone (CCS), gravel, air cooled blast furnace slag (ACBFS), durable sandstone, durable siltstone, granulated slag (GS), or blended natural soil or natural granular materials with open hearth slag (OH), basic oxygen furnace slag (BOF), electric arc furnace slag (EAF), or RPCC. The durable sandstone and siltstone must have a slake durability index greater than 90%. With the exception of GS, the 703.16C material can have gradation Type A through F described as following.

Type A: Material with less than 25% by weight of the grains passing the No. 200 Sieve.

Type B: Type B material can be one of three different possible gradations. The gradations are similar to Items 304, 411, 617, but, 0 to 20% of the material is allowed the pass the No. 200 sieve. Tables 3.1 through 3.3 show the gradation of material for Items 304, 411, and 617, respectively.

Type C: Type C must be a well graded material that meets the gradation shown in Table 3.4.

Type D: Type D material must meet the gradation shown in Table 3.5.

Type E: Type E material must be furnished from any of the coarse aggregates from No. 1 to No. 67 inclusive shown in Table 3.6.

Type F: Type F material must be well graded with a top size from 8 in. to 3 in. and a bottom size of No. 200 sieve. The material must be evenly graded material between the top and bottom size, compactable, stable, and serves the intended use.

Table 3.1. Item 304 gradation (ODOT, 2009)

Sieve Size	Total Percent Passing
2 inch	100
1 inch	70 to 100
3/4 inch	50 to 90
No. 4	30 to 60
No. 30	9 to 33
No. 200	0 to 15

Table 3.2. Item 411 gradation (ODOT, 2009)

Sieve Size	Total Percent Passing
1 1/2 inch	100
1 inch	75 to 100
3/4 inch	60 to 100
3/8 inch	35 to 70
No. 4	30 to 60
No. 30	7 to 30
No. 200	3 to 15

Table 3.3. Item 617 gradation (ODOT, 2009)

Sieve Size	Total Percent Passing
1 inch	100
3/4 inch	60 to 100
3/8 inch	35 to 70
No. 4	30 to 60
No. 30	9 to 33
No. 200	0 to 15

Table 3.4. Granular Material Type C gradation (ODOT, 2009)

Sieve Size	Total Percent Passing
3 inch	100
2 inch	75 to 90
1/2 inch	30 to 60
No. 200	0 to 13

Table 3.5. Granular Material Type D gradation (ODOT, 2009)

Sieve Size	Total Percent Passing
8 inch	100
3 inch	less than 60
3/4 inch	Less than 40
No. 200	0 to 20

Table 3.6. Size of coarse aggregate (AASHTO M43) (ODOT, 2009)

Size No.	Nominal size (1)	Amounts finer than each laboratory sieve (square openings), percentage by weight																
		4	3-1/2	3	2-1/2	2	1-1/2	1	3/4	1/2	3/8	No. 4	No. 8	No. 18	No. 50	No. 100		
1	3-1/2 to 1-1/2.	100	90 to 100		25 to 60	0 to 15		0 to 5										
2	2-1/2 to 1-1/2				90 to 100	35 to 70	0 to 15		0 to 5									
24	3-1/2 to 3/4.				90 to 100		25 to 60		0 to 10	0 to 5								
3	2 to 1.				90 to 100	35 to 70	0 to 15		0 to 5									
357	2 to No. 4.			100	95 to 100		35 to 70		10 to 30		0 to 5							
4	1-1/2 to 3/4.				100	90 to 100	20 to 55	0 to 15		0 to 5								
467	1-1/2 to No. 4.				100	95 to 100		35 to 70		10 to 30	0 to 5							
5	1 to 1/2					100	90 to 100	20 to 55	0 to 10	0 to 5								
56	1 to 3/8					100	90 to 100	40 to 75	15 to 35	0 to 15	0 to 5							
57	1 to No. 4.					100	95 to 100		25 to 60		0 to 10	0 to 5						
6	3/4 to 3/8.						100	90 to 100	20 to 55	0 to 15	0 to 5							
67	3/4 to No. 4						100	90 to 100		20 to 55	0 to 10	0 to 5						
68	3/4 to No. 8						100	90 to 100		30 to 65	5 to 25	0 to 10	0 to 5					
7	1/2 to No. 4							100	90 to 100	40 to 70	0 to 15	0 to 5						
78	1/2 to No. 8.							100	90 to 100	40 to 75	5 to 25	0 to 10	0 to 5					
8	3/8 to No. 8								100	85 to 100	10 to 30	0 to 10	0 to 5					
89	3/8 to No. 16.								100	90 to 100	20 to 55	5 to 30	0 to 10	0 to 5				
9	No. 4 to No. 16									100	85 to 100	10 to 40	0 to 10	0 to 5				
10	No. 4 to 0 (2)										100	85 to 100				10 to 30		

(1) In inches, except where otherwise indicated. Numbered sieves are those of the United States Standard Sieve Series.

(2) Screenings.

Where standard sizes of coarse aggregate designated by two or three digit numbers are specified, the specified gradation may be obtained by combining the appropriate single digit standard size aggregates by a suitable proportioning device which has a separate compartment for each coarse aggregate combined. The blending shall be done as directed by the Laboratory.

When Item 203 Granular Material Types A, B, C, D, E, or F is specified for embankment material, the requirements for 703.16C Type A through F described previously must be met. The embankment material, with the exception of rock and RPCC, should be spread in successive horizontal loose lift of no more than 8 in. Rock can be placed in a maximum loose lift thickness 6 in. larger than the largest diameter of the rock pieces but not to exceed 3 ft. The RPCC is mixed with natural soil or natural granular material and should be placed in maximum lifts of 18 in. The embankment material shall be compacted to a dry density greater than the percentage of maximum dry density shown in Table 3.7, or to a maximum dry density determined by the test section method.

Table 3.7. Embankment compaction requirements (ODOT, 2009)

Maximum Laboratory Dry Weight (lb/ft³)	Minimum Compaction Requirements in Percent of Laboratory
90 to 104.9	102
105 to 119.9	100
120 and more	98

3.1.4 Mechanically Stabilized Earth

MSE walls require select granular backfill (SGB) that conform to aggregate material or structural backfill Type 2. The backfill materials are described as follows:

Aggregate Material: SGB can be CCS and crushed gravel. The gradation for CCS and crushed gravel must meet the gradation requirements shown in Table 3.1 for 304 aggregate. The aggregate must also have a minimum of 90% of the pieces fractured, a maximum of 5% shale material or chart that disintegrates in five cycles of soundness test, a maximum of 50% wear for the Los Angeles test, a maximum of 15% loss for the sodium soundness test, and the portion of aggregate passing the number 40 sieve must have a maximum liquid limit of 25% and minimum plasticity index of 6.

Structural Backfill: Type 2: The Type 2 structural backfill must be of limestone, gravel, natural sand, sand manufactured from stone, and foundry sand. The gradation of the material must meet the requirements of one of the gradations shown in Table 3.8. In addition to gradation, the materials have soundness requirements. Material 1 cannot have aggregations of soil, silt, etc. by weight over 0.50%. Material 2 also has the same aggregations requirement that Material 1 has and cannot have loss over 15% for the sodium sulfate soundness test. Both Material 3 and 4 have the same sodium sulfate soundness requirements as Material 2 and cannot have more than 50% wear for the Los Angeles test.

Table 3.8. Structural backfill gradation (ODOT, 2009)

Sieve Size	Total Percent Passing			
	Material 1	Material 2	Material 3	Material 4
2 1/2 inch				100
1 inch				70 to 100
3/4 inch			100	
3/8 inch	100	100	80 to 100	
No. 4	90 to 100	95 to 100	60 to 100	25 to 100
No. 8	65 to 100	70 to 100	45 to 95	
No. 16	40 to 85	38 to 80		
No. 30	20 to 60	18 to 60		
No. 40				10 to 50
No. 50	7 to 40	5 to 30	7 to 55	
No. 100	0 to 20	0 to 10		
No. 200	0 to 10	0 to 5	0 to 15	5 to 15

Both the aggregate and structural backfill materials have additional requirements:

- Slag material and recycled Portland cement concrete cannot be used.
- The internal angle of friction of the material must equal or be greater than 43 degrees.
- The pH and resistivity must be within ODOT thresholds to limit reinforcement corrosion.

3.1.5. Approach Slab Base Material

The approach slab aggregate base is required to be CCS, crushed gravel, crushed ACBFS, GS, or OH slag. The CCS, crushed gravel, crushed ACBFS, and OH need to meet the gradation shown in Table 3.1 with OH slag having 0 to 10% passing the No. 200 sieve. GS shall be furnished such that it will compact and have the gradation shown in Table 3.9.

Table 3.9. Granulated slag gradation requirements (ODOT, 2009)

Sieve Size	Total Percent Passing
2 inch	100
1 inch	85 to 100
No. 100	0 to 15

The material can be placed by hand, with dozers, or graders, if the area of the approach slab is too small for self propelled spreader machines. The lift thickness should not exceed 6 in. when using 10 to 12 ton vibratory rollers. The lift should be no more than 4 in. thick if vibratory rollers are not used. The material must be compacted to 98% of the maximum dry density. A minimum of eight passes of the compaction equipment should be used.

3.2. Ohio Current and Past Research

3.2.1. Approach slabs

An evaluation of bridge approach design and construction based on statistical correlations was done on information gathered from 358 Ohio bridges (Timmerman, 1976). The data analysis indicated no correlation between bridge approach performance and Ohio design and construction parameters used in the study. Relating bridge condition to satisfactory or unsatisfactory behavior proved extremely difficult, while not providing any reliability. However, several general observations were noted on approach characteristics during this study. The bridge approaches in Ohio performed better when built on embankment and foundation soils of low plasticity and slight cohesiveness. Abutments with wing walls exhibited larger approach slab settlement than bridges without wing walls due to inadequate compaction of wing-wall backfill. The differential settlement between the bridge abutment and the end of the approach slab was greater for pile-supported abutments versus stub abutments. The pile-supported abutments, however, provided better bridge support. Lastly, the largest pile-supported abutment settlement occurred with cast-in-place reinforced concrete piles supported by soil friction and/or end bearing in moderately over-consolidated cohesive soils having a liquidity index near zero. According to Timmerman, the design and construction policies of ODOT appear to be satisfactory and the only way to ensure good approach performance is with active maintenance.

ODOT experienced approach slab distress shortly after applying the integral concept to continuous steel beam bridges (Burke, 1999). When such bridges were constructed adjacent to asphalt concrete approach pavements, approach slab seats at the ends of bridge superstructures were fractured and settled, hindering movement of vehicular traffic.

Approach slabs were not anchored to superstructures in the first ODOT adaptations of the integral concept to continuous steel bridges. Instead, friction between slabs and aggregate bases tended to anchor the slabs and aggregate bases together. As they should, these bridges contracted and expanded in response to daily ambient temperature changes. Because these joints were not sealed, roadway debris infiltrated them while they were open. Subsequently, during superstructure expansion, the force of the expanding superstructure compressing joint debris provided sufficient pressure to overcome friction at the approach slab-aggregate base interface, pushing the approach slabs toward the asphalt concrete pavement in small incremental movements as the joints continued to open, fill with debris, and close with each temperature cycle. Within a few years, the approach slabs were pushed to near the edges of the slab seats, diminishing the bearing area and causing fractures due to traffic weight.

Tying approach slabs to slab seats of integral bridges with reinforcing bars has prevented the slab expansion problem. ODOT places such bars diagonally through slab seats to function as longitudinal ties, as well as hinges to facilitate settlement of the far end of approach slabs. Other engineers sometimes ignored the probability of gradual and long-term consolidation of approach embankments and used straight extensions of top deck slab reinforcement to tie approach slabs to bridges. Such straight ties in approach slabs on new embankments caused slab cracking and tie steel yielding. Burke (1999) recommends effective approach slab designs should consider cyclic movement, joint infiltration, and embankment consolidation.

3.2.2 Erosion

Bridges with closed decks function to retain and transport bridge deck drainage to the approaches (Burke, 1999). Without the protection of full-width approach slabs with curbs or parapets, accumulated deck drainage will erode shoulder support, embankment surfaces, and backfill at the abutments. To eliminate this problem, closed deck-type integral and semi-integral bridges should be provided with full-width approach slabs with curbs or raised parapets. The approach slabs should also be made high enough to compensate for future overlays.

3.2.3 Cycle Control Joints

Probably the most significant unresolved problem with integral and semi-integral bridges is the availability of cost-effective, fully functional, and durable cycle control joints (Burke, 1999). Short bridges usually employ a common pavement-movement joint composed of pre-formed fillers. Longer bridges often utilize fingerplate joints with easily maintainable curb inlets and drainage troughs. Recently (2010), ODOT personnel have found that modular joint designs have shown success for long bridge application. With intermediate-length bridges, however, development of suitable cycle control joints is still in the evolutionary stages. Compression seals, strip seals, and other elastomeric devices have been used with marginal success. Recently (2010), ODOT personnel stated strip seals have been utilized with a high rate of success.

After considerable, yet unsuccessful, experimentation efforts, ODOT decided to use an easily maintainable pavement-pressure relief joint (a joint filled with asphalt concrete), until a more suitable joint is developed. This decision was made recognizing that, during cold weather, such joints will crack open and allow surface water to enter. Sleeper slabs are used not only to support adjacent slabs but also to help minimize the adverse consequences of surface drainage penetrating joints while they are open. This particular approach not only facilitates cyclic movement of approach slabs, it also is a very cost-effective design, because such joints also function to protect bridges from longitudinal pressures generated by the restrained growth of jointed rigid pavement. Lateral subsurface drainage provisions adjacent to relief joint sleeper slabs are important to avoid trapping drainage water and promoting pavement pumping.

3.2.4 Hinged Joints

An ODOT attempt to reduce integral abutment pile bending resulted in superstructure-encased stringers hinged to abutment pile caps (Burke, 1999). This hinge facilitates superstructure rotation at abutments due to deck slab placement, highway traffic movement, and abutment pile cap rotation caused by thermal expansion/contraction of the superstructure. ODOT personnel (2010) indicate that the rotation will only occur in this case if the resistance of the piles/embankment is greater than the resistance of the hinge bar. ODOT currently proposes that the resistance against rotation increases as the skew increases.

Water penetration at the hinge joints was prevented by several means, including raised approach slab curbs and longitudinal roadway under drains turned laterally to embankments upon reaching bridge approaches. Sealing to the back of hinged joints was attempted by using 2 ft of porous backfill against abutments, perforated drain pipes to drain toward embankment sides, and

elastomeric sealers. However, after several years of monitoring the new hinge design in several bridges, it appears that the long-term success of the ODOT hinged abutment design, compared to structures related to integral abutments without hinges, is marginal at best.

4. OTHER STATES' BRIDGE DESIGN AND CONSTRUCTION STATE OF PRACTICE

4.1. Colorado

4.1.1. Standards, Specifications, and Details

The Colorado DOT (CDOT) (2009) uses a single reinforced approach slab with minimum length of 20 ft for typical approach slabs. Figure 4.1 shows a plan and section of the typical CDOT approach slab. The approach slab rests on a corbel and is tied to the superstructure with a horizontal No. 5 bar as shown in Figure 4.2. The expansion joint is located at the inverted-T sleeper slab as detailed in Figure 4.3. The expansion joint material is similar to the ODOT compression seal joint material. At the slab-to-sleeper interface, is #20 gage sheet metal intended to reduce sliding friction.

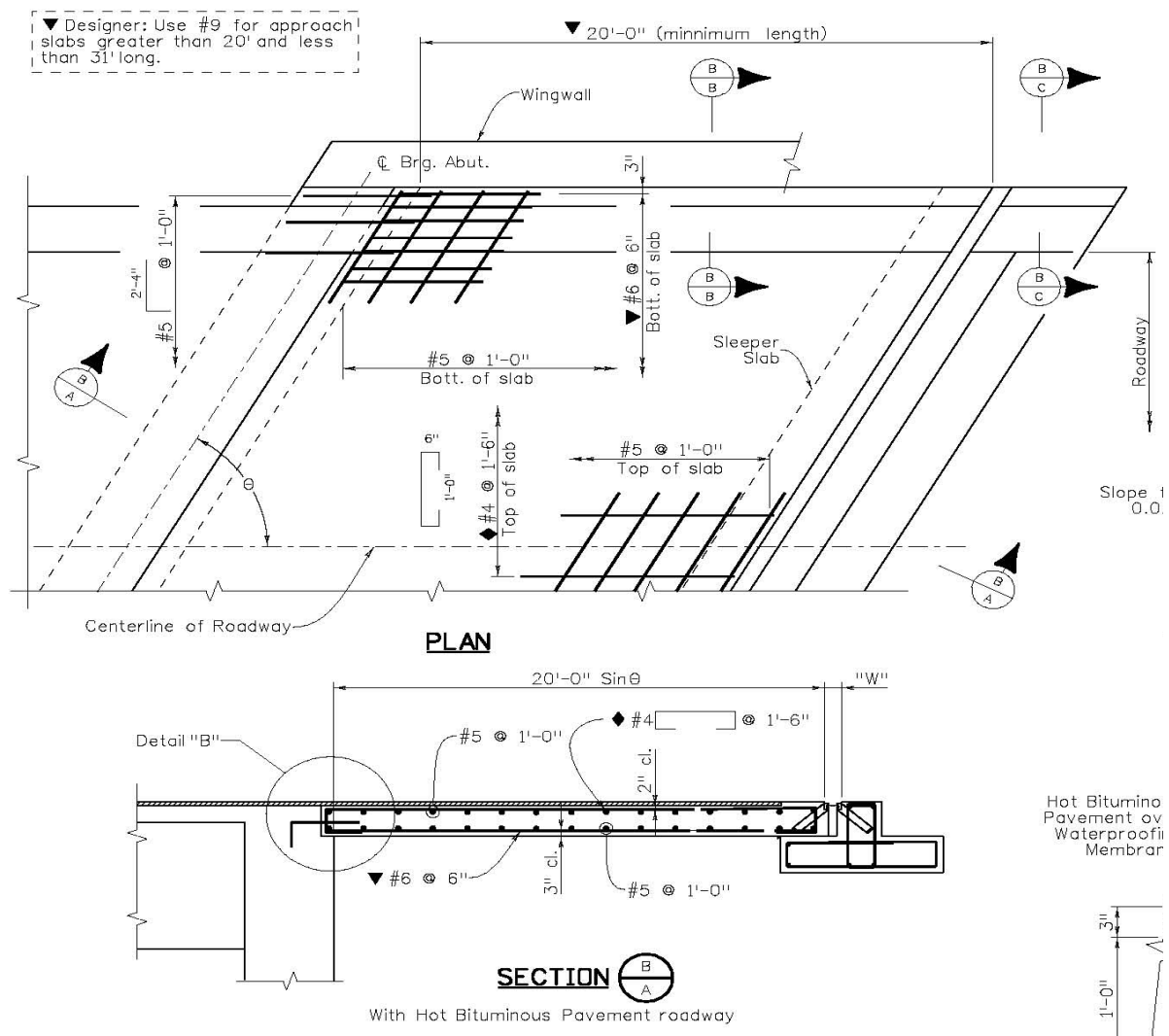


Figure 4.1. CDOT approach slab with expansion joint at the sleeper slab plan and section (2009)

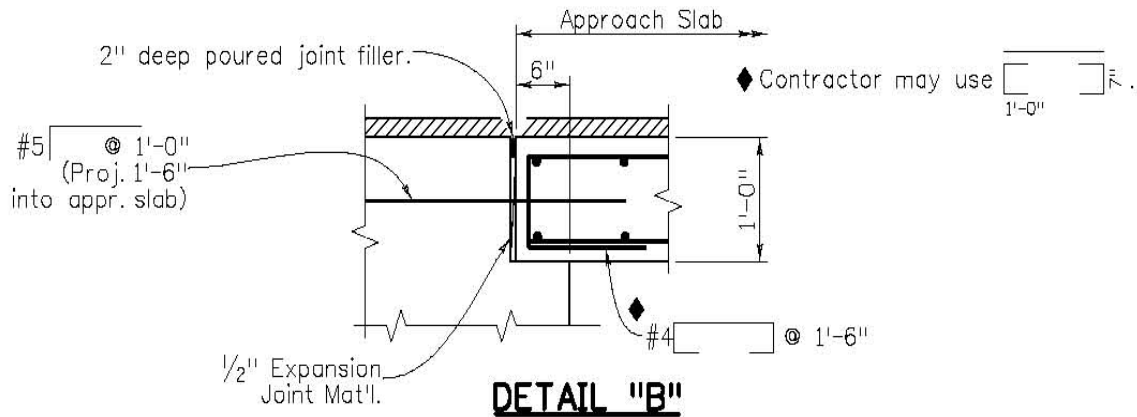


Figure 4.2. Approach slab connection and bearing at bridge (CDOT, 2009)

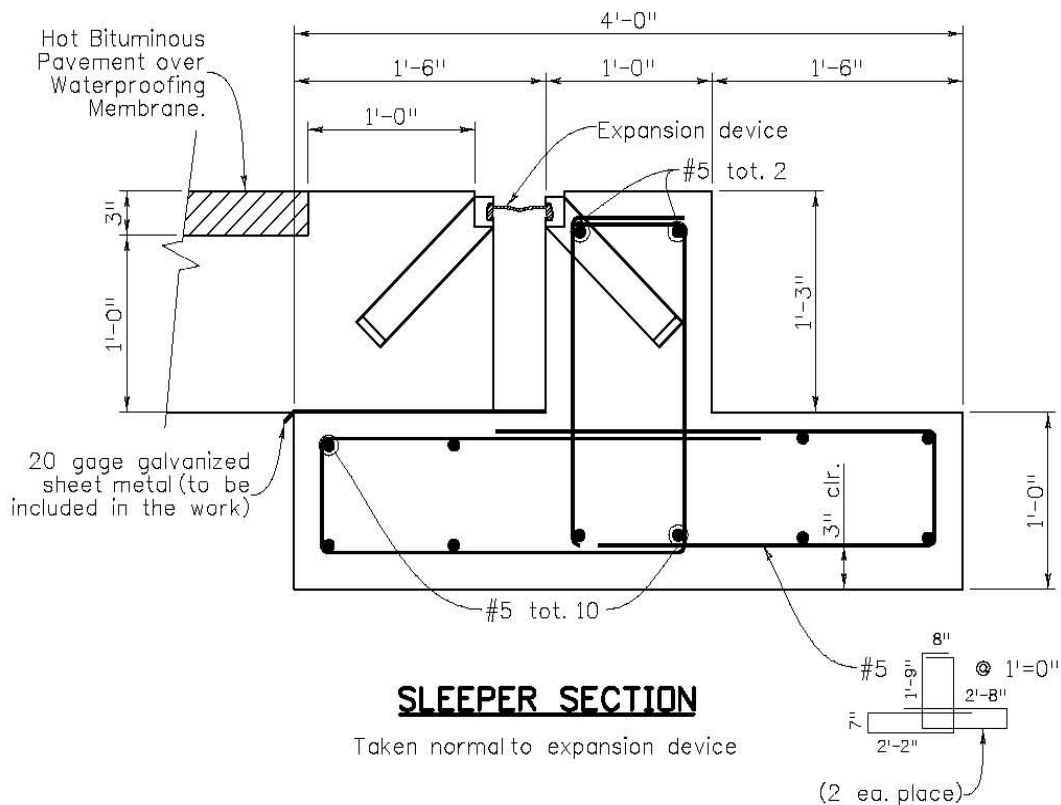


Figure 4.3. Approach slab expansion joint at sleeper slab (CDOT, 2009)

CDOT also has a typical detail for cases when the expansion joint to be placed is at the bridge joint, as shown in Figure 4.4. For this scenario, the approach slab is tied to the sleeper slab with reinforcing bars and the bridge paving notch is used at the sliding surface for the bridge expansion and contraction. The same expansion joint is used as described above. A trimmed 4 in. plastic pipe is used below the joint to drain water away from the abutment joint.

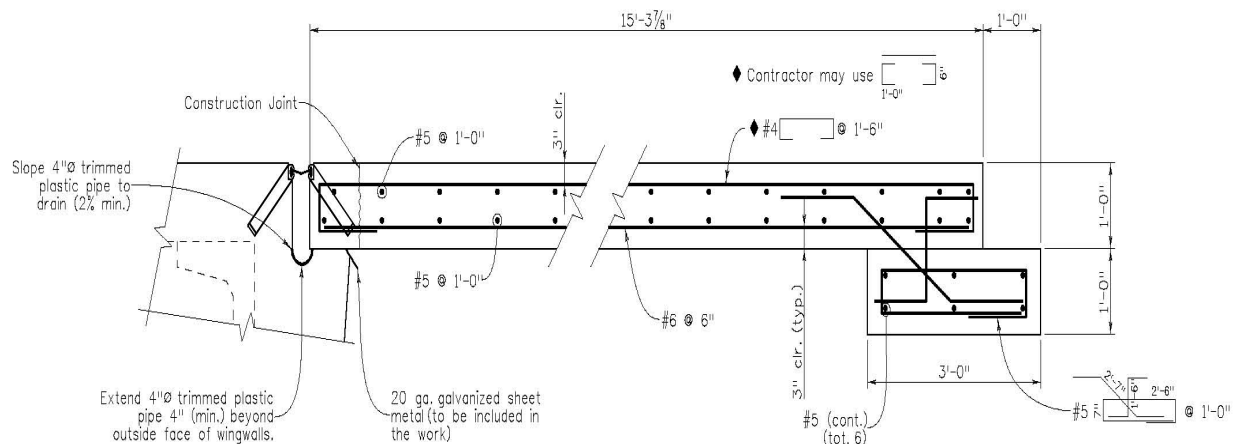


Figure 4.4. CDOT approach slab section with expansion joint located at bridge joint (2009)

CDOT has a similar standard detail as ODOT when approach slabs are used with asphalt roadways as shown in Figure 4.5. In this case, the approach slab is rigidly attached to the bridge superstructure and rests on the bridge paving notch. The other end of the approach slab rests on a sleeper slab and butts directly against the asphalt pavement. CDOT specifies a 3 in. hot bituminous layer of pavement over a waterproofing membrane to be placed over the pavement, approach slab, and bridge deck.

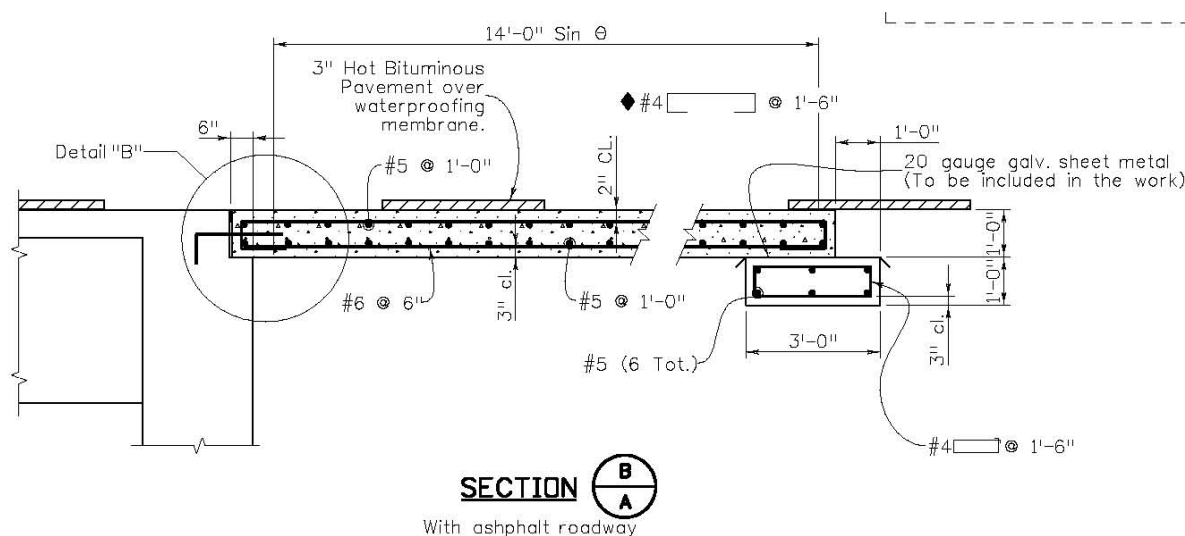


Figure 4.5. Approach slab with asphalt roadway and 3 in. asphalt overlay (CDOT, 2009)

4.1.2. Embankment Quality Assurance

Currently, CDOT uses single orientation nuclear gauge (NG) testing practices for testing the compaction of backfills at MSE walls and bridge embankments. However, single orientation NG testing has limitations on the allowable distance away from walls or structures and can give inaccurate readings up to +/-3 to 4% compaction. Due to NG limitations, Mooney et al. (2008)

investigated different quality assurance devices to determine if they can better determine the compaction for MSE wall and bridge embankments for Class 1 backfill in Colorado. A number of testing devices were evaluated by determining the pros and cons of each instrument and their applicability for backfill types and locations. From the initial evaluation, the dynamic cone penetrometer (DCP), light weight deflectometer (LWD), and the Clegg Hammer were further evaluated in the field. Field testing on MSE wall and bridge approaches revealed the LWD, DCP, and Clegg Hammer are all capable of determining the compactiveness of Class 1 backfill at a distance within 1 ft of a wall or structure. Target values were determined for each of the devices for a desired 95% field compaction. During the testing, the DCP was found to be sensitive to moisture content and needed to penetrate geogrid or geofabric for deep testing. The LWD was insensitive to moisture and the Clegg Hammer sensitively was inconclusive.

Mooney et al. suggested that to improve NG testing, the single orientation four minute reading be changed to a four orientation one minute reading at each orientation. The multiple orientations would help account for the spatial differences in density, modulus, and shear strength. The researchers also recommended a pilot study using the LWD and Clegg Hammer in conjunction with the NG to better establish target values, evaluate how target values change with soil type, moisture, and seasons, and allow them to populate a database of target values, as well as for inspectors, consultants, and contractors to evaluate the devices.

4.2. Illinois

4.2.1. Standards, Specifications, and Details

Kevin Riechers of Illinois indicated they have been building integral abutment bridges since the early 1980s and began connecting the approach slab approximately five years after that (Greimann et al., 2008). The typical detail used by Illinois is shown in Figure 4.6. This detail consists of #5 reinforcing bars spaced every 12 in. extended horizontally from the bridge deck into the approach slab with 4 ft in the bridge deck and 6 ft in the approach slab. In addition, vertical #5 reinforcing bars extend from the corbel into the approach slab every 12 in. The reason cited for connecting the slab and bridge was to keep the joint closed to keep water and debris out and to ensure that the pavement moves with the bridge. Transverse cracking of the slab was reported to be a problem. Riechers also reported that another problem is the settlement of the sleeper slab at the other end of the approach slab and that a new design is being considered. No research has been performed on approach slab to bridge connections. Also, nothing is apparently done to reduce surface friction under the approach slab except a bond breaker between the slab and wing walls of U-Back abutments. The soil is backfilled at the abutment with no compaction to avoid additional lateral earth pressures that may restrain thermal expansion of the bridge.

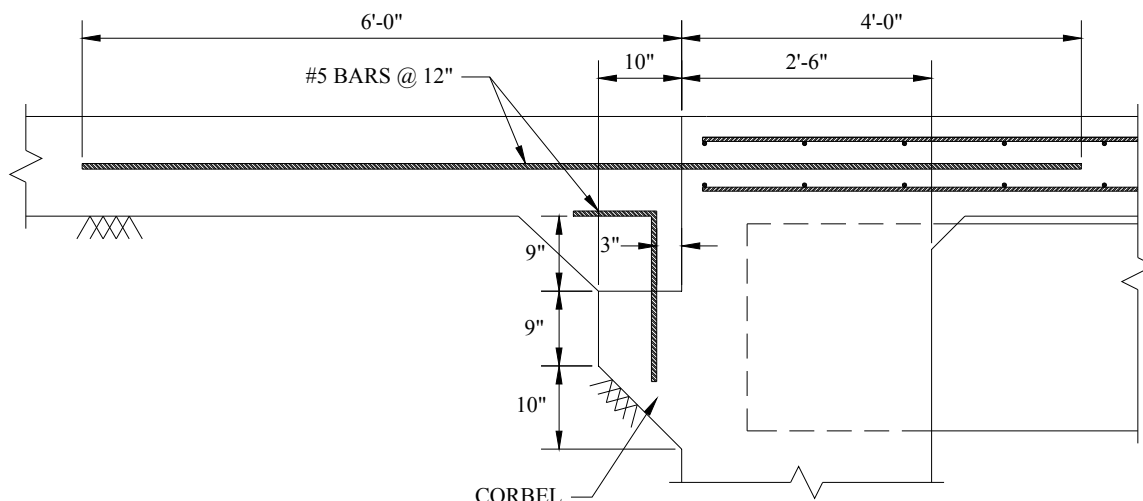


Figure 4.6. Typical Illinois detail (Greimann et al., 2008)

4.2.2. Embankment Material

Illinois DOT (IDOT) (2010) specifications contain two general sections for constructing embankments. The first section, Embankment, states that the embankment is to be constructed of materials that will compact and develop stability. The material is to be placed in 8 in. loose lifts and leveled by means of bulldozers, blade graders, or equipment approved by the engineer. For embankments greater than 3 ft deep, the first 2 ft are allowed to be compacted to 90% of standard laboratory density. The next 1 ft must be compacted to a minimum of 93% and the rest of the embankment must be compacted to 95% standard laboratory density.

The other IDOT section provides information pertaining to porous granular embankments. The coarse aggregate that can be used should be gravel, crushed gravel, crushed stone, crushed concrete, crushed slag, chats, crushed sandstone, or wet bottom boiler slag. The required gradation for the coarse aggregate is shown in Table 4.1. The fine aggregate that can be used must be sand, stone sand, wet bottom boiler slag, slag sand, or chats. The material must meet the quality deleterious count shown in Table 4.2 and have the gradation shown in Table 4.3. Both aggregates are to be placed in 6 in. loose lifts and compacted to approval by the engineer.

Table 4.1. Porous granular coarse aggregate gradation (IDOT, 2010)

Sieve Size	Total Percent Passing
3 inch	100
1 inch	90 to 100
No. 4	50 to 100
No. 16	30 to 80
No. 50	0 to 20
No. 200	0 to 4

Table 4.2. Porous granular fine aggregate quality (IDOT, 2010)

QUALITY TEST	Class B
Na ₂ SO ₄ Soundness 5 Cycle, Illinois	15
Modified AASHTO T 104, % Loss max.	
Minus No. 200 (75 µm) Sieve Material,	6
Illinois Modified AASHTO T 11, % max.	
Deleterious Materials: *,**	
Shale, % max.	3.0
Clay Lumps, % max.	3.0
Coal, Lignite, & Shells, % max.	3.0
Conglomerate, % max.	3.0
Other Deleterious, % max.	3.0
Total Deleterious, % max.	5.0

*Applies only to sand

** Test shall be run according to Illinois Test Procedure 204

Table 4.3. Porous granular fine aggregate material (IDOT, 2010)

Sieve Size	Total Percent Passing
No. 8	100
No. 40	40 to 80
No. 100	0 to 6
No. 200	0 to 4

4.2.3. Current and Past Research

A visual survey of 1,181 Illinois bridge approaches was conducted in 1994 to determine the frequency of differential approach settlement (Long et al., 1998). The research team concluded that 27% of the approaches exhibited significant differential movement that leads to discomfort of the driver. Although there are infinite sources for the cause of the bump at the bridge, Long et al. determined the six major causes of differential movement for Illinois bridges: 1.) material compression or erosion at the abutment and embankment interface, 2.) broken approach slab, 3.) compression of foundation soils 4.) internal erosion of embankment soils 5.) poor construction practices, and 6.) distortion of foundation soils caused by areal mechanisms.

In general, the differential movement was found to occur at the embankment/abutment interface, the end of the approach slab, or at a break in the approach slab (Long et al., 1998). The differential movement of these elements was on the order of 0.20 in. to 0.30 in. The approach–relative gradient was found to be a better predictor for rider discomfort and approach distress. The approach-relative gradient is determined by dividing the differential settlement by the length over which the settlement occurs. For new construction, an approach-relative gradient less than 1/200 generally provides good rider comfort.

4.3. Iowa

4.3.1. Standards, Specifications, and Details

The Iowa DOT (2009) prefers the use of integral abutments over stub abutments to eliminate the maintenance problems associated with expansion joints. Table 4.4 shows the bridge skews and lengths at which the Iowa DOT uses integral versus stub abutments.

Table 4.4. Iowa DOT criteria for using and integral or stub abutment (2009)

Skew	Bridge Lengths	Remarks
0 - 30° incl.	0 – 300 feet (0 - 91 500 mm)	Use integral abutments
0 - 30° incl.	300 – 500 feet (91 500 - 152 500 mm)	Show stub abutments on Situation Plan, and include note on plan to investigate during final design for use of integral abutments. Greater length than 500 feet use stub.
30° - 45° incl.	0 – 150 feet (0 - 46 000 mm)	Use integral abutments. Greater length than this use stub.
above 45°	Any Length	Do not design a bridge with a skew this high.

Figure 4.7 shows the typical Iowa DOT integral abutment. Note that a concrete cap beam is cast integrally with the piles. The piles have their weak axis oriented parallel to the line of travel on the bridge. The steel piles oriented in this direction increase the longitudinal bridge flexibility. The cap beam is integrally attached to the superstructure by vertical #8 reinforcing bars placed around the perimeter of the cap beam. Unlike the ODOT integral connection, which allows rotation of the superstructure independent of the substructure, the Iowa DOT connection provides a rigid attachment between the superstructure and the abutment, allowing the piles to translate and rotate and, thereby, creating stiffness continuity across the system.

Figure 4.8 shows the Iowa DOT typical approach slab detail. The approach slab consists of two general sections. The section next to the bridge is reinforced and tied to the bridge paving notch. The next segment is a non-reinforced section. A doweled contraction joint, noted 'CD' in the figure, separates the reinforced section and non-reinforced section. Within the non-reinforced section is the expansion joint for the bridge. The expansion joint is a doweled expansion joint, noted 'EF' in the figure, which connects the approach system to the roadway pavement. A detail of the 'EF' joint is shown in Figure 4.9, along with other Iowa DOT expansion joints. The 'EF' joint has dowels at 12 in. on center, greased and sleeved on one end to allow them to slide in and out of the concrete as the bridge and pavement expand and contract. The joint is about 3.5 in. wide and is filled with flexible foam to keep debris out of the joint. Although this is the typical Iowa DOT approach slab, several other approach slabs and expansion joints have been used and researched and are discussed herein.

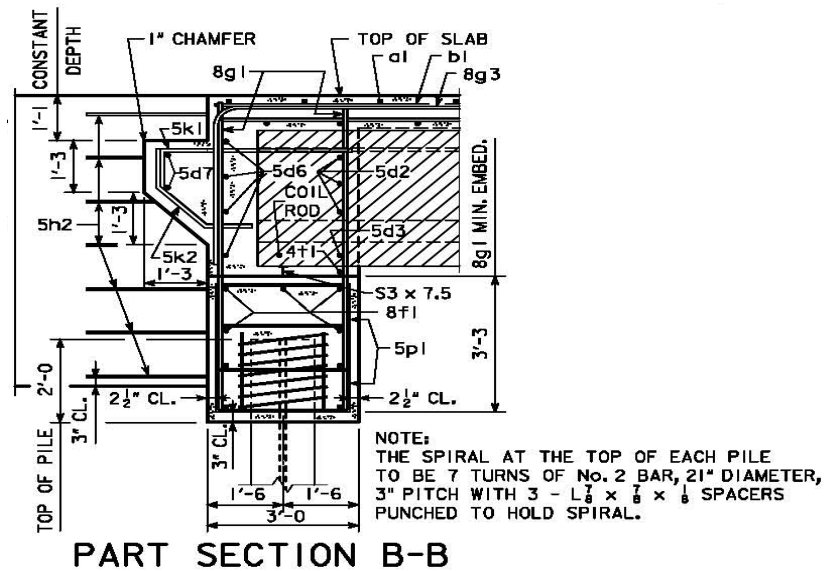
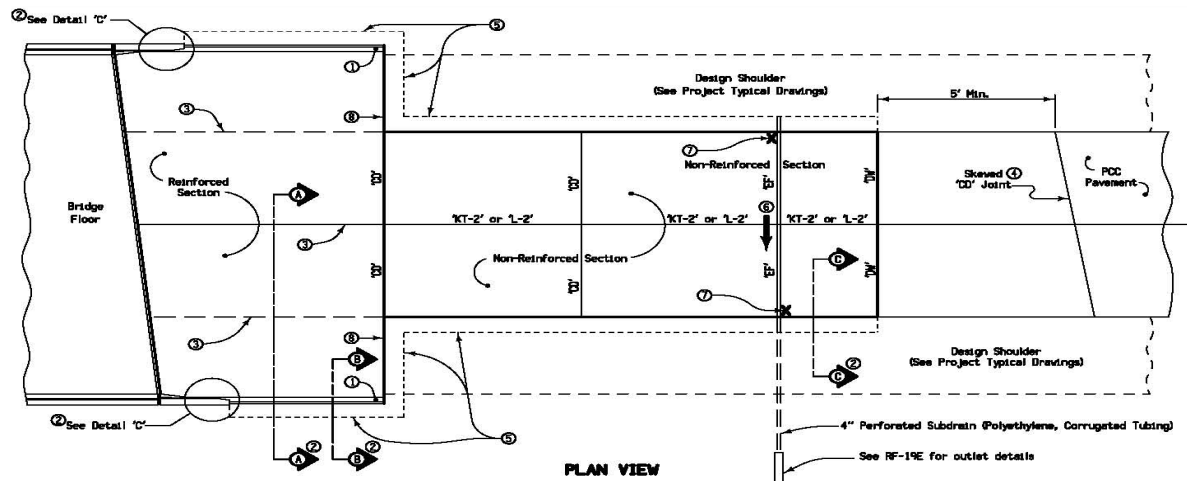
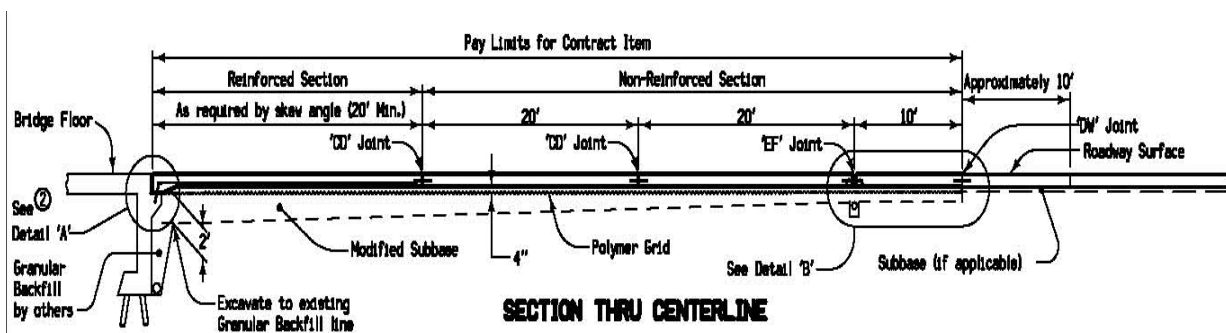


Figure 4.7. Iowa DOT typical integral abutment design (2009)



a. Bridge approach plan



b. Bridge approach profile

Figure 4.8. Iowa DOT typical approach slab plan and profile (2009)

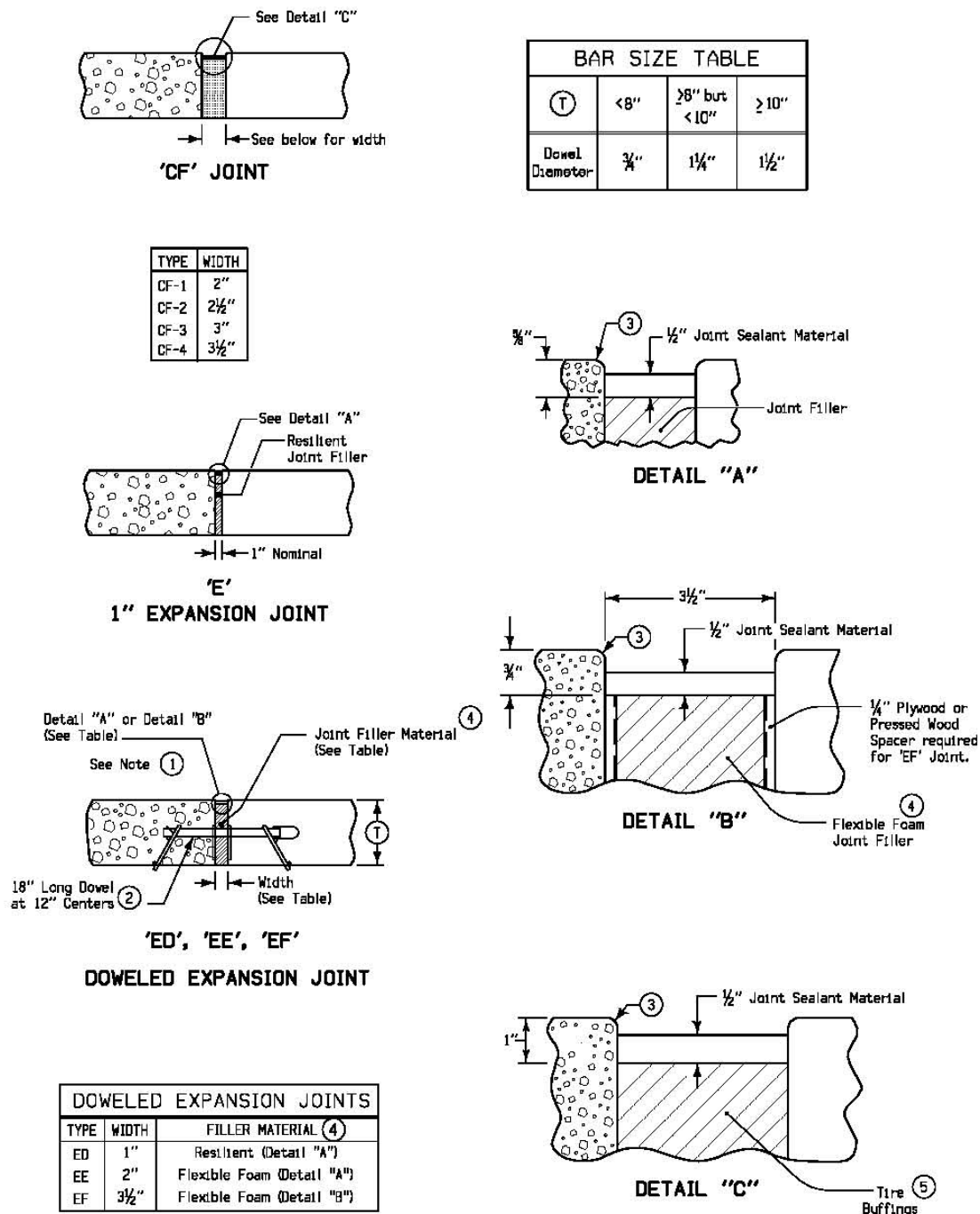


Figure 4.9. Iowa DOT typical expansion joint details (2009)

4.3.2 Embankment Material

Appropriate embankment material shall be specified in the contract documents. With the exception of rock fills and granular blankets, the material shall be deposited in horizontal layers not exceeding 8 in. (loose). Soils containing roots, sod, or other vegetation shall be placed in the

outermost 3 ft of the embankment. Layers of drier and wetter soils should be alternated when practical. For rock fill material, lift thicknesses will be allowed up to 4 ft. Granular blankets, consisting of crushed stone or natural sand and gravel, are to be spread in widths and thicknesses as shown in the contract documents. Quality of the granular backfill material should meet the specifications in Table 4.5.

Table 4.5. Aggregate quality specifications (granular backfill materials) (Iowa DOT, 2009)

Coarse Aggregate Quality	Maximum Percent Allowed	Test Method
Abrasion	55	AASHTO T 96
C Freeze	20	Office of Materials Test Method No. Iowa 211, Method C
Total of Abrasion & C Freeze	65	---
Clay Lumps and Friable Particles	4	Materials I.M. 368

Unsuitable materials may be used according to Iowa standard road plan RL-1B, unless the engineer directs otherwise. Unless otherwise specified, unsuitable material in uniform layers is to be no more than 8 in. (loose thickness). The contractor must cover each unsuitable layer with at least one layer of suitable material.

With the exception of rock fills and granular blankets, material shall be deposited in horizontal layers not exceeding 8 in. (loose). Two types of compaction methods are used for embankment construction.

Type A: A minimum of one rolling per in. depth of each lift is required. Additionally, a roller must penetrate no more than 3 in. into an 8 in. (33%) lift. The lead engineer shall then determine if the moisture content is suitable for satisfactory compaction. Aerating or adding moisture may be need as described in Section 2107 of the Iowa DOT standard specifications.

Type B: This compaction method requires a specified number of disking and roller coverages or the equivalent. Before applying the next lift, the surface is to be smoothed and compacted such that penetration of the roller is no more than that of Type A compaction. Aeration and moisture limitations are also to be complied with as referenced in Type A compaction.

Compaction with specific moisture and density control is to be followed as described in Section 2107 of the Iowa DOT standard specifications.

4.3.3. Current and Past Research

4.3.3.1. Approach Practices

To determine bridge approach problems and provide recommendations for improving bridge approaches, White et al. (2005) studied 74 existing or under-construction bridges in Iowa. The field investigation of the existing bridges revealed the following deficiencies (pictorially shown in Figure 4.10):

1. Voids were found under the approach slabs. The voids indicated the backfill had insufficient moisture control and/or compaction.
2. The expansion joint was not properly sealed when flexible foam or recycled tire joint fillers were used.
3. Erosion of the backfill material was seen to cause voids under the approach slab, erosion around H-pile support, failure of the slope projection, and faulting of the approach slab. Most of the bridges inspected had poor water management leading to erosion.
4. Bridges that had surface drains also had less erosion than bridges without surface drains. The Iowa DOT drain shown in Figure 4.11 appeared to be the most effective drain detail seen.
5. Several of the bridge subdrains were found to be blocked or collapsed.
6. Most of the bridges investigated with approach slab problems had slopes greater than 1/200, which is above the maximum gradient as discussed by previous authors.
7. Grouting below the approach slab did not significantly reduce further settlement of the backfill.

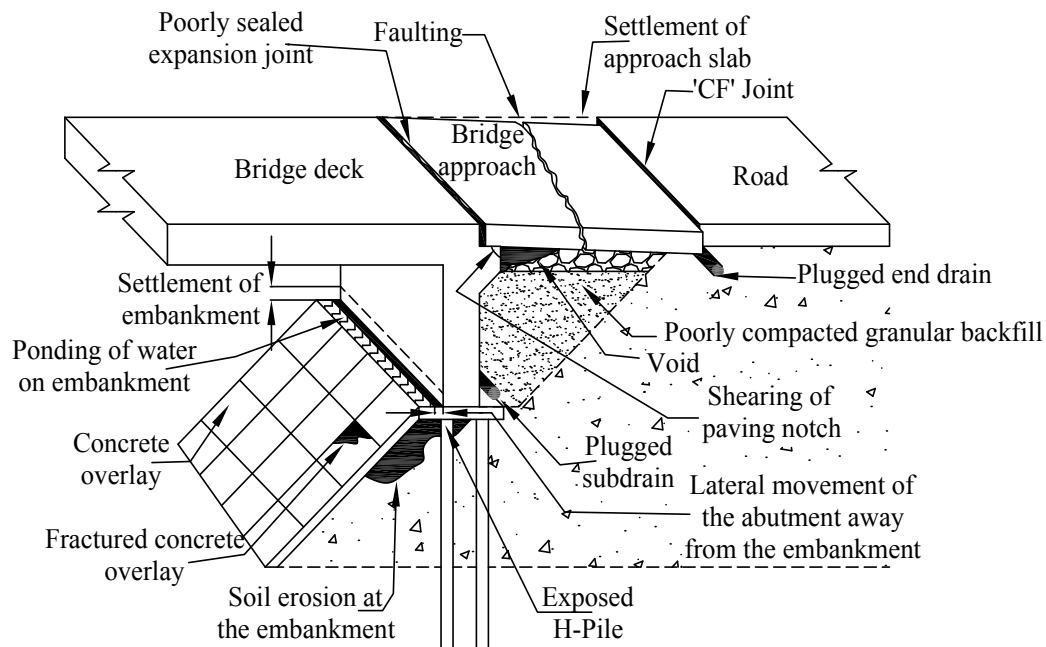


Figure 4.10. Common problems seen at bridge sites in Iowa (White et al., 2005)



Figure 4.11. Effective surface drain detail (White et al., 2005).

Investigation of the new bridge approach construction practices revealed that most of the granular backfill that was being used as abutment fill at new bridge sites was not being sufficiently compacted. In addition, when the backfill material was being compacted, the moisture content was near the bulking moisture content, leaving the backfill susceptible to collapse at saturation. Lastly, several subdrains were observed to be plugged with soil during or shortly after construction. Porous backfill was not used around the subdrains at most bridge sites.

The backfill materials used by the Iowa DOT were also characterized by looking at grain size distribution and conducting collapse index tests. The grain size distribution of porous backfill (classified as SP according to the Unified Soil Classification System/USCS) and granular backfill (classified as GP according to USCS) was compared to the average opening of the drainage pipe perforations. The porous backfill had 1% of the particles finer than the average pipe perforations; however, the granular backfill had about 70% of the particles smaller than the perforations. The grain size distribution was also used to categorize the backfill as erodible. The porous backfill was out of the erosion range for grain size, while the granular backfill used at most Iowa DOT bridges had common grain sizes with erodible soils, leaving it more susceptible to erosion. The collapse test concluded that the granular backfill settles about 6% of the original height due to saturation. The porous backfill did not settle due to saturation.

White et al. (2005) also constructed a laboratory bridge water management model to evaluate the current Iowa DOT backfill specifications and practices and to look at various backfill alternatives for recommendation and future use. A schematic of the model is shown in Figure 4.12. The actual model is pictured in Figure 4.13.

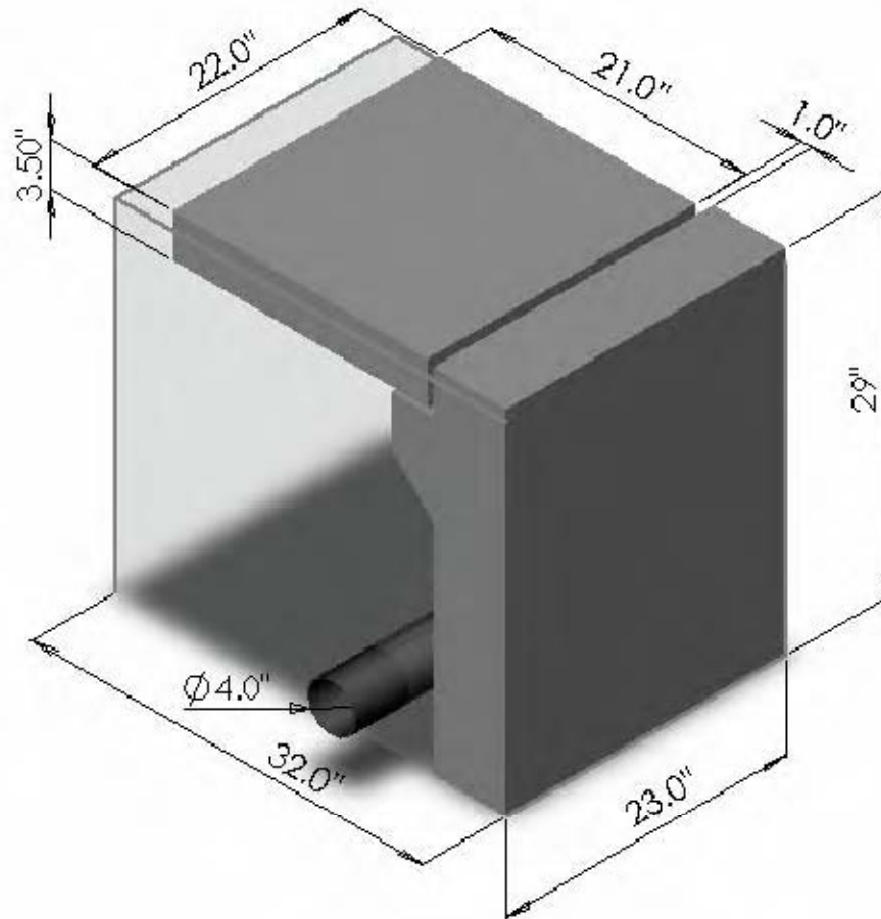


Figure 4.12. ISU water management bridge approach model schematic (White et al., 2005)



Figure 4.13. ISU water management bridge approach model (White et al., 2005).

After loading the model, water is forced to flow through the expansion joint under the approach slab through the drainage system or material and out the subdrain. The water was continuously re-circulated through the system at a steady state for four hours. Settlement at the end of the approach slab was measured, along with the void under the slab, at maximum steady state water flow conditions.

The scaled laboratory testing found that using porous backfill helped minimize slab settlement and void formation relative to granular backfill. Other backfill alternatives, including using various geocomposite drainage material/systems at the abutment, resulted in 7 to 12 times the increase in flow over that of granular fill. Recycled tire chips were also tested in the model resulting in reduced settlement, low void formation, and an increase in drainage of 17 times that of granular material. Overall, the study suggested use of a combination of porous backfill and geocomposite drainage systems behind newly-constructed abutments, improved embankment compaction practices, connecting the approach slab to the abutment, and supporting the far end of the approach slab on a sleeper slab with a 2 in. construction joint.

4.3.3.2. Paving Notch

The Iowa State University (ISU) Bridge Engineering Center (BEC) (2008) designed an alternative to conventional paving notch construction. This new system consists of a rectangular, precast concrete element that is connected to the rear of the abutment using high-strength threaded steel rods and an epoxy adhesive that is similar to that used in segmental bridge construction. Full-scale laboratory testing of the proposed paving notch replacement system was performed and consisted of a series of static and dynamic load tests to investigate the system abilities to sustain repeated cyclic and ultimate loads. The first phase of testing, post-tensioning without epoxy adhesive, was intended to investigate the post-tensioning (PT) force needed to prevent slip of the paving notch without using an adhesive. Phase 2 included the use of a drilled and epoxy grouted anchor (one row of stainless steel rods). Phase 3 comprised the Iowa DOT desire to compare the strength of the proposed system to their current cast-in-place (CIP) repair system. The final phase of the testing program (drilled and epoxy grouted anchoring with *two* rows of stainless steel rods) consisted of the application of a fatigue load to the precast paving notch specimen to simulate a finite number of wheel load applications.

Based on the results of the testing and the post-test visual inspections, it was concluded that: 1.) When epoxy adhesives are used, the connection of the precast paving notch to the abutment can be adequately achieved by hand-tightening 3/4 in. diameter stainless steel threaded rods that are drilled and grouted about 10 in. into the abutment. 2.) The use of an additional set (row) of stainless threaded rods improved the ultimate load-carrying capacity of the precast paving notch system. 3.) In comparison to the ultimate strength of the current Iowa DOT CIP paving notch repair system, the proposed precast paving notch system showed larger ultimate load carrying capacity. 4.) No significant slippage was observed during cyclic testing. 5.) The use of different materials and reinforcing steel for the precast paving notch specimen had little influence on the overall performance of the system; none of the tested precast paving notch specimens failed during the testing. In all cases, failures occurred at the connection of the system. The final design for the field implementation, which was modified from the original design based on the findings and lessons learned from the laboratory testing, is shown in Figure 4.14.

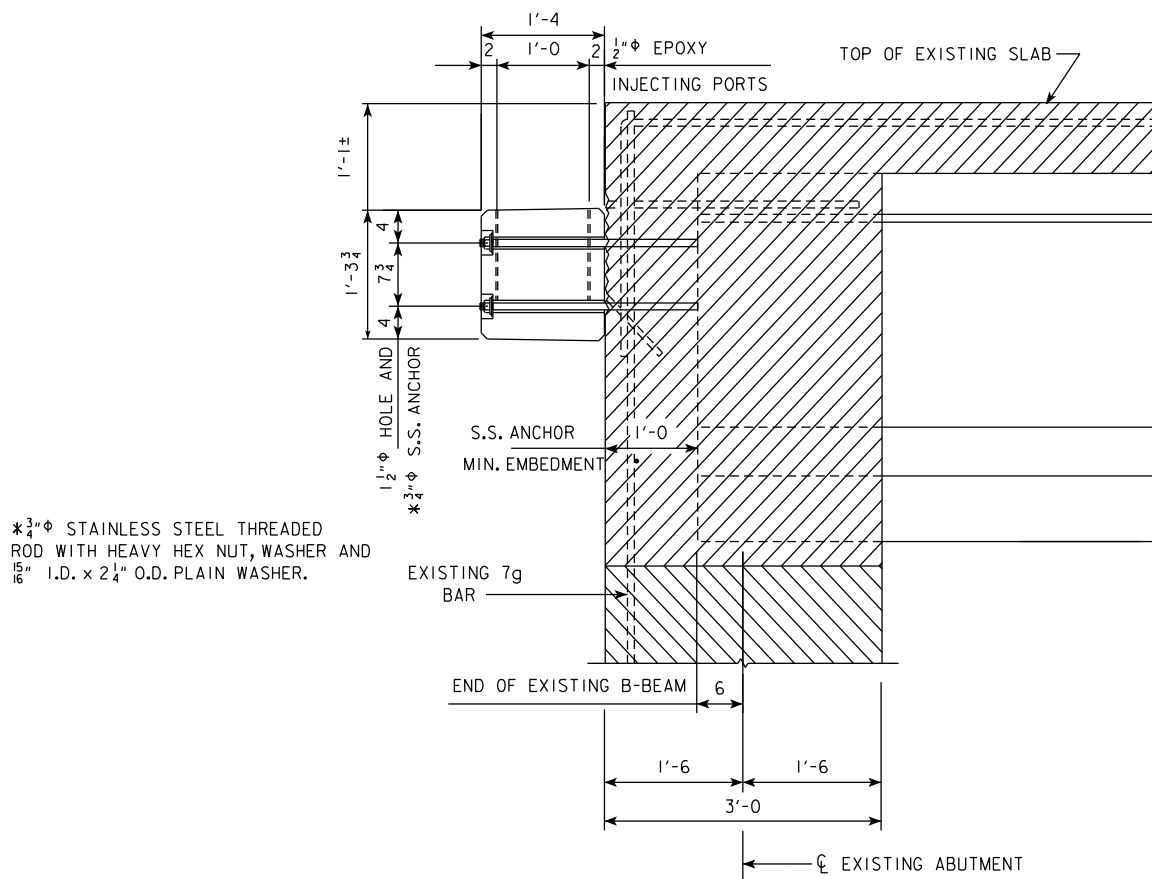


Figure 4.14. Precast paving notch system selected for field implementation (BEC, 2008)

4.3.3.3. Precast Approach Slab and Connections

The Iowa DOT has long recognized that approach slab pavements of integral abutment bridges are prone to settlement and cracking (Greimann et al., 2008), which manifests itself as the “bump at the end of the bridge.” The bump is generally not a significant safety problem; rather, it is an expensive maintenance issue. A commonly recommended solution is to integrally attach the approach slab to the bridge abutment, which moves the expansion joint to a location further from the bridge where soil settlement is less of a concern and maintenance is easier. Two different approach slabs, one being precast concrete and the other being CIP concrete, were integrally connected to side-by-side bridges on Iowa Highway 60. The primary objective in studying the bridges was to evaluate approach slab performance and the impacts the approach slabs had on the bridge.

Greimann et al. installed a health monitoring system on both bridges and the two different approach slab systems. To encompass all aspects of the system and to obtain meaningful conclusions, several behaviors were studied and monitored during the evaluation period, including abutment movement, bridge girder strain changes, approach slab strain changes, approach slab joint displacements, post-tensioning strain, and abutment pile strain changes.

Based on the information obtained from the 12 month monitoring period, the following general conclusions were made in regards to the integral approach slab system: 1.) The integral connection between the approach slabs and the bridges appear to function well with no observed distress at the connection and no relative longitudinal movement measured between the two components. 2.) Tying the approach slab to the bridge appears to impact the bridge abutment displacements and girder forces. 3.) The source of the impact may, however, be the manner in which the approach slab is attached to the main line pavement. 4.) The two different approach slabs, the longer precast slab and the shorter CIP slab, appear to impact the bridge differently. This impact was clear in the differences in the midspan moments and the slab strain patterns over time. It is not clear, however, whether it was the type of approach slab or the size of the approach slab that has the greatest impact. 5.) The measured strains in the approach slabs indicate that a force exists at the expansion joint and should be taken into consideration when designing both the approach slab and the bridge. The observed responses generally followed an annual cyclic and/or short-term cyclic pattern over time. The annual cyclic pattern had summer responses at one extreme, a transition through the fall to the other extreme response in the winter, followed by a transition in the spring back to the summer responses. A linear relationship of the transitions between the extreme responses was typically observed. Seasonal and short-term cycles were also evident in most data, probably caused by friction ratcheting.

4.3.3.4 Abutment Reinforcement

White et al. (2005) also performed an analytical investigation of the potential of the approach slab settlement due to failure of the pavement notch or the slab itself at the bridge end. A finite element and computer-aided strut-and-tie model were used to investigate the paving notch and abutment. The abutment/paving notch investigated was used on non-integral bridges in Iowa and had a 10 in. paving notch connected to a 15 in. wide back wall. The back wall was 49 in. tall. The notch and abutment wall reinforcing all consisted of # 5 bars.

White et al. found reinforcing used by the Iowa DOT in the paving notch was sufficient for the demands estimated from the strut and tie forces under the worst possible static and dynamic load cases. However, the analysis revealed that the vertical reinforcement in the abutment walls of non-integral bridges may not be adequate. It is suggested that these #5 reinforcing bars be replaced with #7 reinforcing bars. The abutment wall for integral bridges was also investigated and found that, for a 36 in. abutment width, the #8 vertical bars are generally satisfactory.

Although most findings of the analytical study suggested that the current reinforcement details for the pavement notch and the approach slab are adequate, it is emphasized that poor workmanship and/or use of poor quality concrete can lead to premature failure of the pavement notch and the approach slab. Hence, good inspection and quality control procedures should be followed during construction of the bridge abutments and approach slabs.

4.4. Kansas

4.4.1. Standards, Specifications, and Details

From Kansas, John Jones reported that approach slabs have been connected to the bridge for the last 12 years (Greimann et al., 2008). The connection is made by extending #5 reinforcing bars horizontally from the bridge deck into the approach slab and ending in a standard hook, seen in Figure 4.15. The approach slab rests on a corbel at the bridge end and a sleeper slab at the other end, typically 13 ft away. The reason behind the connection was to remove the bump that formed at the end of the bridge. Though the bump was removed from the bridge end, it now appears between the slab and pavement. Jones reported that the connection has performed reasonably well and that public perception has been positive. Problems may arise if the sleeper slab settles, causing negative moments in the slab at the abutment. A solution to this is carefully mud-jacking the slab being mindful to avoid clogging the drain behind the abutment. No research has been performed and nothing is used to reduce friction. The backfill criteria used is the same as the road criteria (18 in. lifts at 90% compaction) with a strip drain installed behind the abutment.

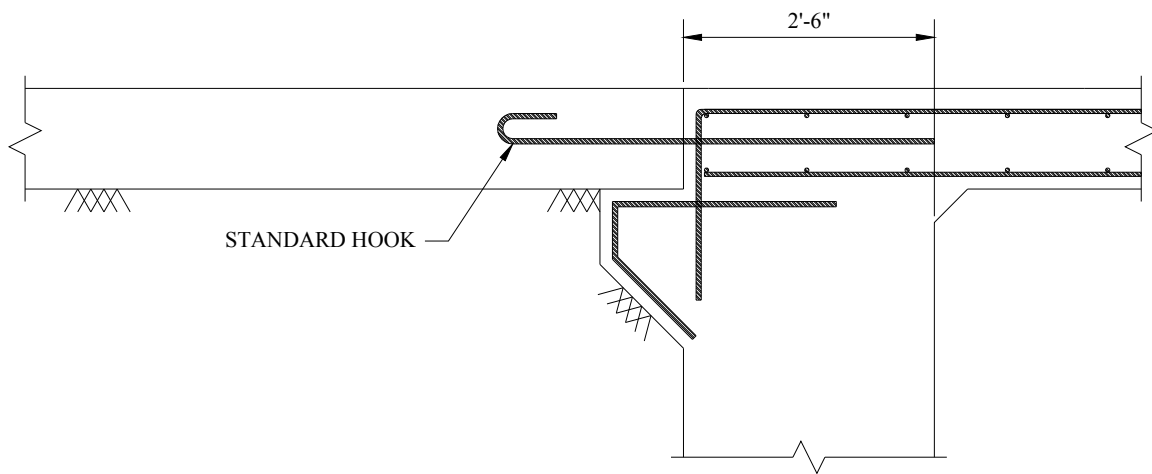


Figure 4.15. Typical Kansas detail (Greimann et al., 2008)

4.4.2. Embankment Material

According to the Kansas DOT (KDOT) (2009) embankments can be constructed from material classified as soil, rock/soil, or rock. Table 4.6 gives gradation criteria for the materials.

Table 4.6. Embankment gradation classification (KDOT, 2009)

Classification	Gradation Criteria
Soil	Less than 20% retained on 3/4 in. sieve
Rock/Soil	Greater than 20% less than 80% retained on 3/4 in. sieve
Rock*	Greater than 80% retained on the 3/4 in. sieve

Four various types of compaction used by KDOT are shown in Table 4.7. If the contract documents do not specify compaction, Type B compactions are the default to be used. When soil embankment material is to be used, the material can be placed in horizontal lifts of approximately 8 in. loose thickness and compacted as specified on the contract documents and at the proper moisture content. When rock/soil is utilized, the material can be placed on 10 in. loose thickness and compacted with a vibratory roller to the proper density. Rock embankment materials can be placed in loose lift thickness of approximately the average size of the larger rocks but not to exceed 2 ft.

Table 4.7. Various types of soil compaction requirements by KDOT (2009)

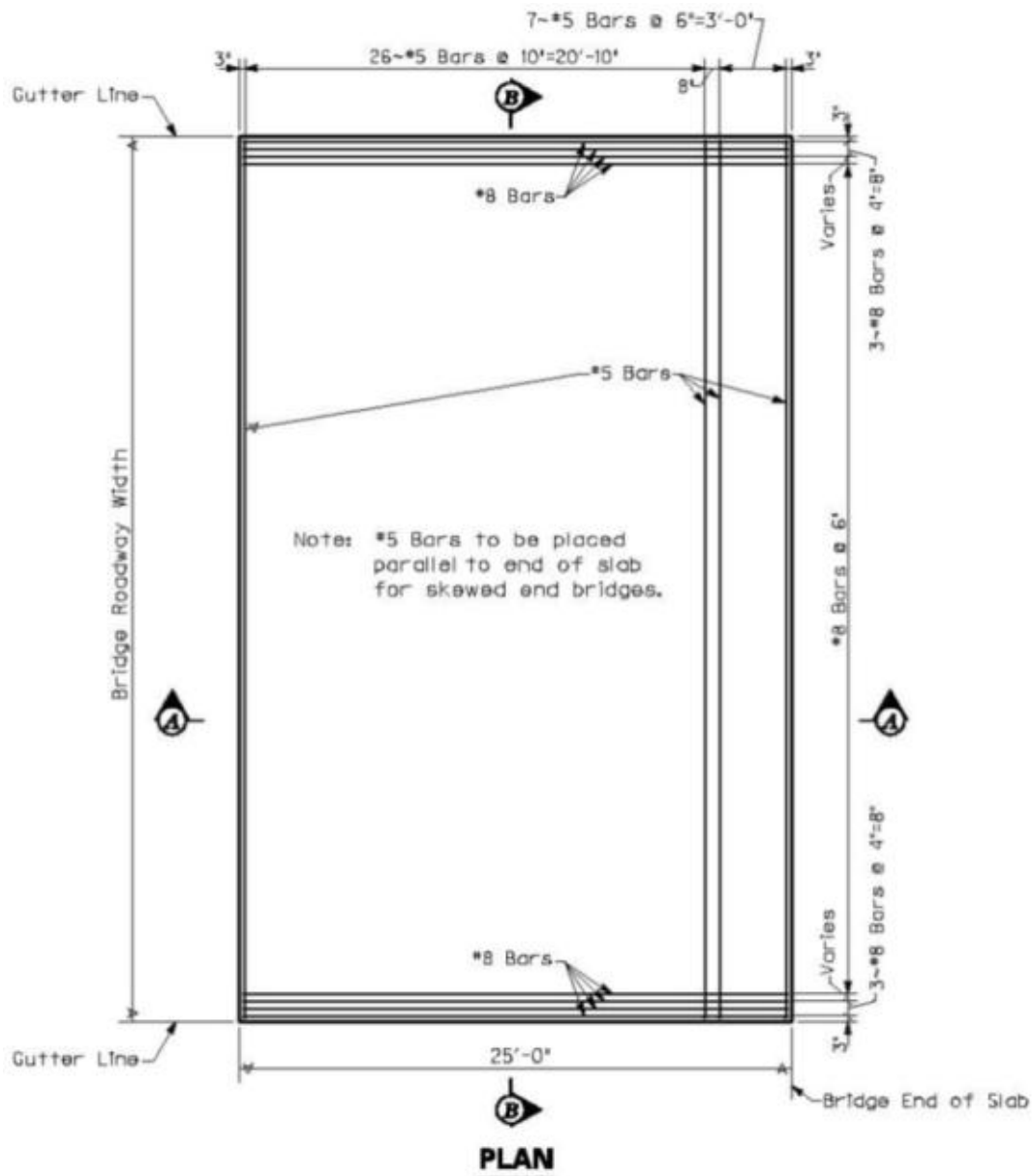
Compaction Type	Minimum Compacted Soil Density
Type AAA	100% of Standard Density
Type AA	95% of Standard Density
Type A	90% of Standard Density
Type B	<p>Such that no further consolidation is gained by additional rolling. The Engineer will visually determine acceptable Type B compaction based on the following:</p> <ul style="list-style-type: none"> • Acceptable Type B compaction is demonstrated if the tamping feet of a tamping (sheepsfoot) roller “walks out” of the soil and rides on top of the lift being compacted. • In soil with low plasticity or nonplastic fine-grained materials, the tamping feet may not “walk out” of the material being compacted. With these materials, acceptable Type B compaction is demonstrated if the tamping feet support the weight of the roller (without the drum of the roller contacting the lift being compacted). • In sand and gravel, where the use of a tamping roller produces unacceptable results, use other types of rollers (such as a pneumatic-tired) to compact this type of material. With these materials, acceptable Type B compaction is demonstrated if no further consolidation is evident after additional passes of the roller. • In small irregular areas where the use of conventional compaction equipment is impracticable, use other equipment and methods to obtain compaction. The Engineer will determine by visual inspection if Type B compaction is obtained. • If the Engineer is unable to visually determine that Type B compaction is obtained, the Engineer may conduct density tests on the compacted soil. If tested, the compacted soil density shall be at least 90% of the standard density.

4.5. Kentucky

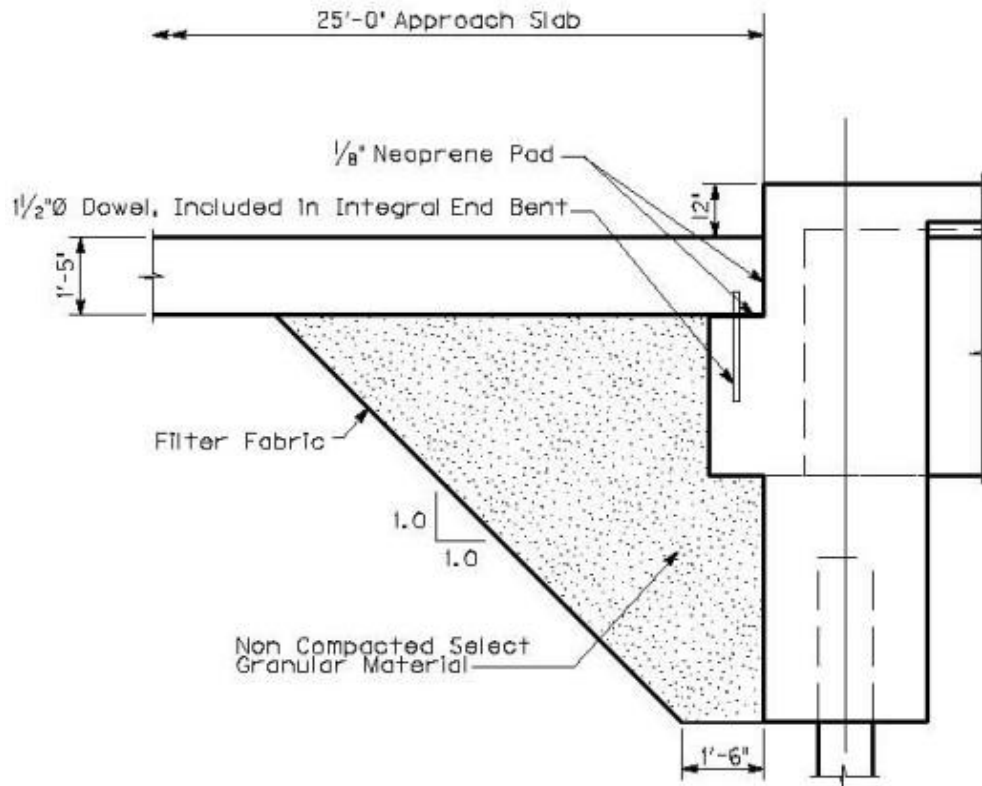
4.5.1. Standards, Specifications, and Details

The Kentucky Transportation Cabinet (KYTC) (2009) typical approach slab is illustrated in Figure 4.16. The approach slab is 25 ft long with a 17 in. thickness. What is different about Kentucky’s approach slab is that the finish elevation is 12 in. lower than the bridge’s finish elevation. The lowered approach slab surface allows pavement to be placed on top of the

approach slab, ensuring a high-quality alignment between the two. Further, this system builds in a known fix for tide quality issues.



a. Plan view of approach slab



TYPICAL SECTION @ BRIDGE END

b. Section of approach slab

Figure 4.16. Kentucky typical approach slab details and abutment connection (KYTC, 2009)

Figure 4.17 shows the typical KYTC expansion joint detail. The Kentucky expansion joint is very similar to the Iowa DOT 'EF' joint. The expansion joint uses greased dowel bars with sleeves on one end to allow the joint to move horizontally, but restricts vertical movement across the joint.

4.5.2. Embankment Material

KYTC does not list specific embankment materials within their standard specifications, but appears to provide the information on the contract documents. The specifications do, however, state that all embankment materials must be compacted to a density of at least 95% maximum density. The specifications also provided information for general embankment materials as follows: earth, friable sandstone, weathered rock, waste crushed aggregate, bank gravel, creek gravel, or similar materials should be constructed in lifts not exceeding a loose depth of 12 in. thickness prior to compacting.

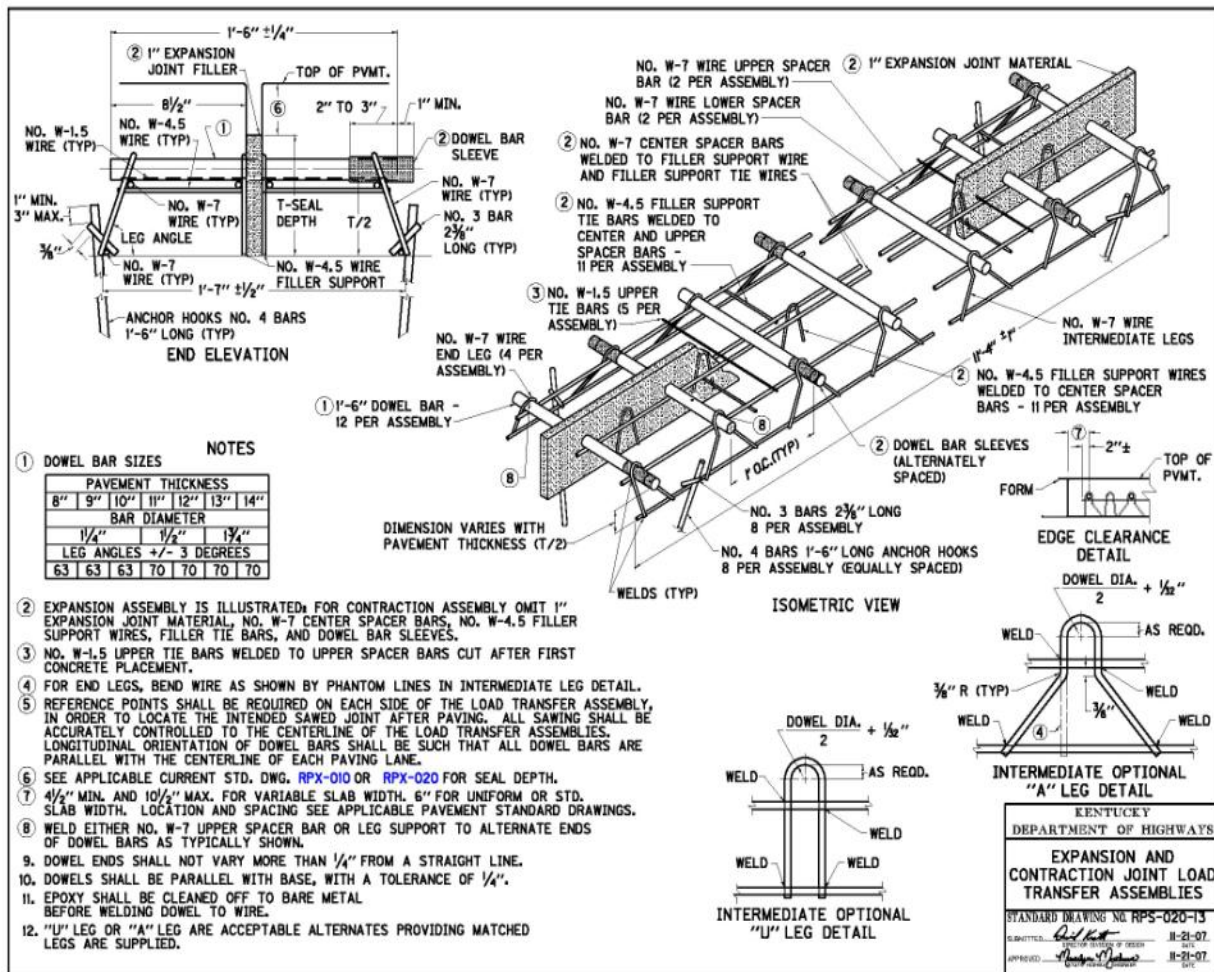


Figure 4.17. Kentucky typical expansion joint detail (KYTC, 2009)

Un-weathered limestone, Durable Shale, or Durable Sandstone shall be constructed in lifts not exceeding 3 ft. The maximum size of boulders or large rocks cannot exceed 3 ft vertically or 4.5 ft horizontally. In addition, rocks should be distributed to minimize voids, pockets, and bridging. Non-durable shale must have rock fragments removed or broken down if they have a thickness greater than 4 in. or any dimension greater than 1.5 ft. The material should be placed in loose lifts not exceeding 8 in. Water needs to be applied; then, material needs to be disked to accelerate slaking.

4.6. Louisiana

4.6.1. Current and Past Research

Das et al. (1999) studied pile-supported approach slabs in Louisiana. The use of pile-supported approach slabs is to provide a transition between the bridge and roadway over soft and organic subsoils that are mainly found in southern Louisiana. Although several of the pile-supported

approach slabs were performing well, there were several that exhibited rideability issues due to differential settlement between the highway and bridge abutment. Figure 4.18 shows the typical pile support configuration that LA DOTD uses for approach slabs.

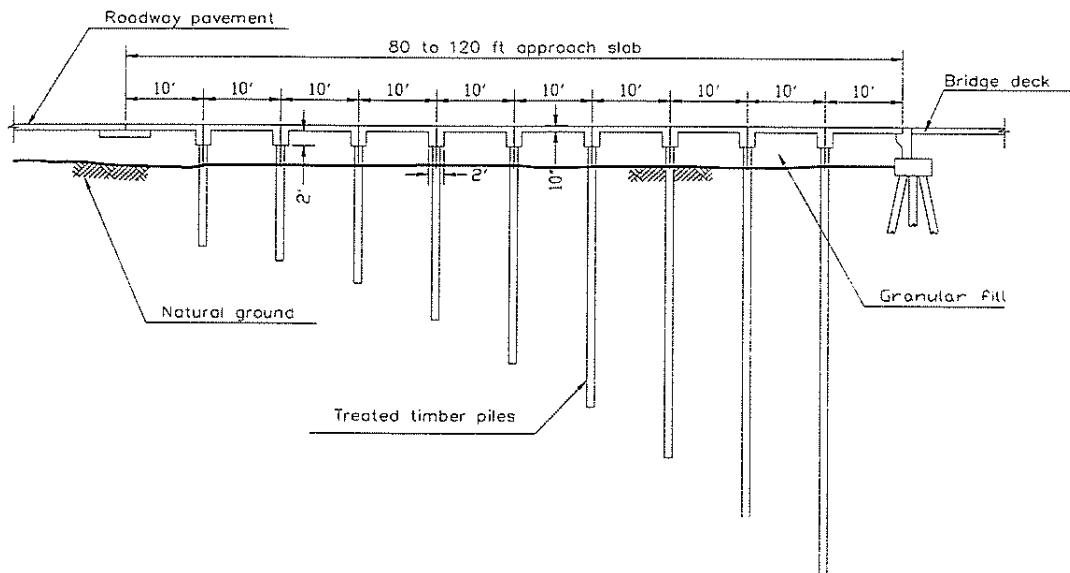


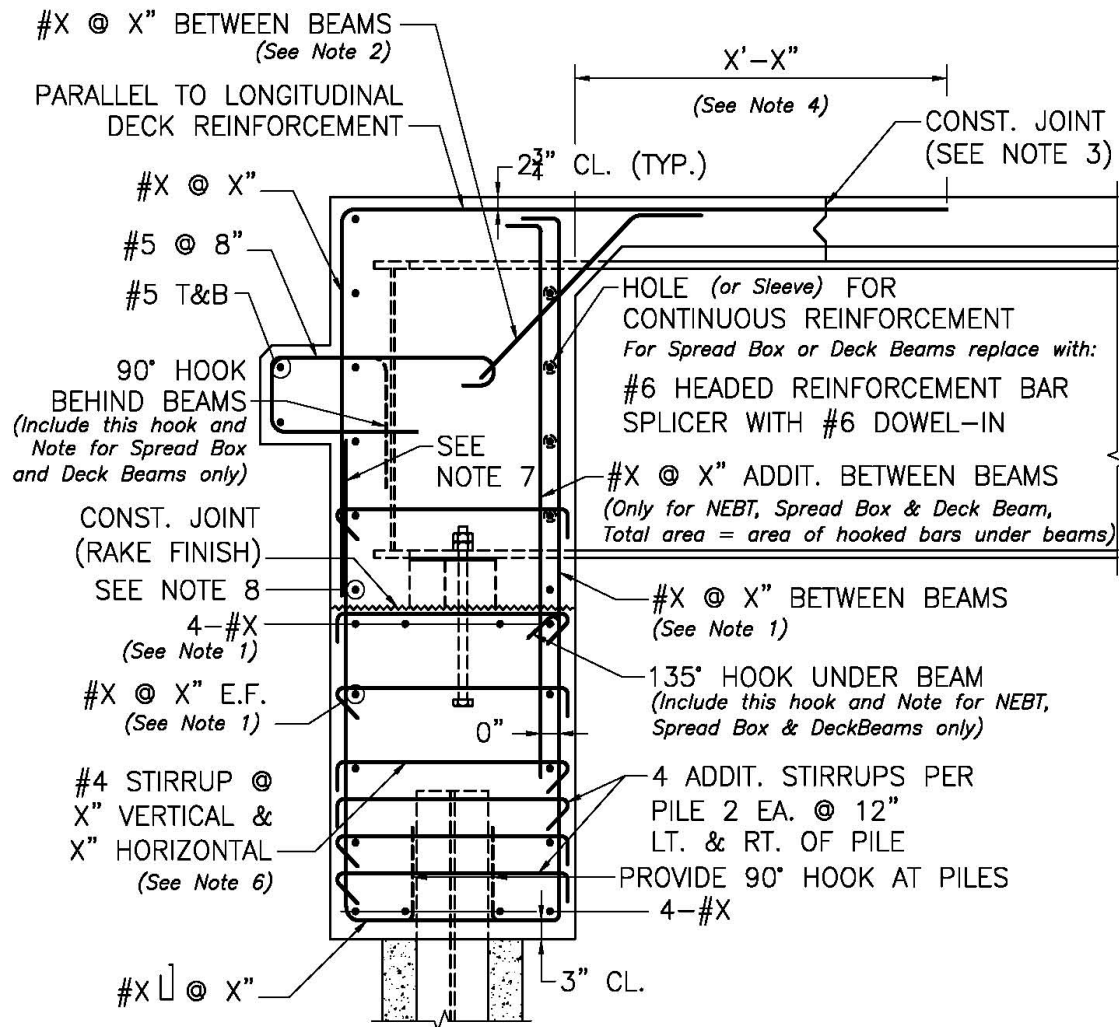
Figure 4.18. Typical LA DOTD pile approach slab support (DAS et al., 1999)

Das et al. concluded that the current LA DOTD design for pile supported approach slabs was not necessarily adequate to produce acceptable field performance, because of varying site conditions from bridge to bridge. The most influential variable that controlled the performance was found to be drag forces (e.g., negative skin friction) on the pile. To reduce the drag force on the piles, Das et al. recommend that the piles be placed after sufficient embankment consolidation had taken place, longer piles to be used in some cases, and/or increase the surcharge height or period. Das et al. developed a spreadsheet program to predict pile settlement based on site conditions. Design parameters for approach slab support include pile length, pile spacing, embankment height, and approach slab dimensions.

4.7. Massachusetts

4.7.1. Standards, Specifications, and Details

Figure 4.19 shows the typical integral abutment for the Massachusetts DOT (MassDOT) (2009). The abutment cap is connected to the superstructure with vertical reinforcing bars around the perimeter similar to the Iowa DOT integral abutment.

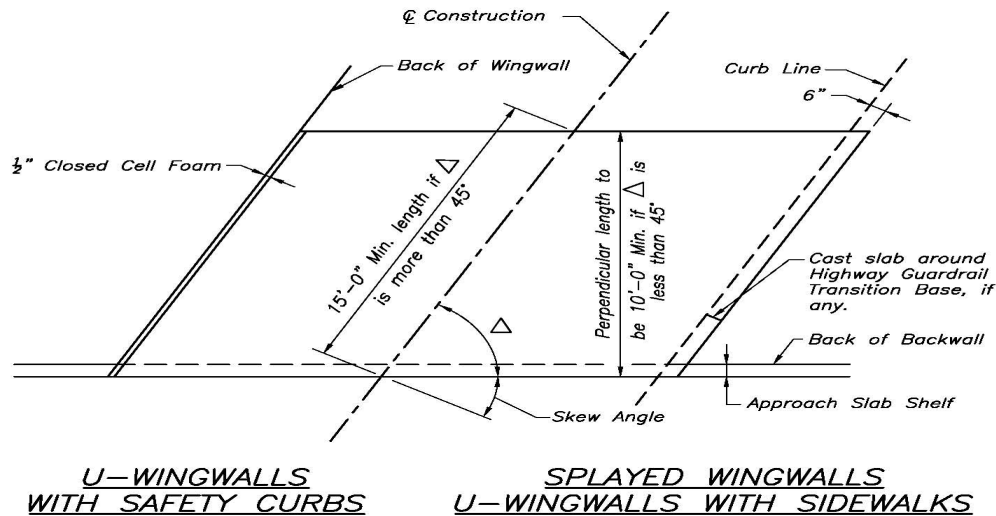


TYPICAL INTEGRAL ABUTMENT REINFORCEMENT

SCALE: 1/2" = 1'-0"

Figure 4.19. MassDOT integral abutment reinforcement (2009)

The typical MassDOT approach slab plan is shown in Figure 4.20. The MassDOT has three typical details for the approach slab configuration. Figure 4.21 shows the Type 1 approach slab. The approach slab is inset for placement of a thin layer of asphalt over the approach slab. In this case the other two approach slabs, Type 2 and Type 3, are shown in Figure 4.22 and 4.23, respectively. Both of these approach slabs have a 14 in. inset for a full-depth layer of pavement to be placed above the approach slab. The difference in the approach is that the Type 2 approach is not integrally connected to the paving notch. The end of the paving notch located next to the pavement is keyed into the sub base, therefore "locking" the approach slab in place. The expansion joint for the Type 2 approach slab is located at the bridge approach slab interface.

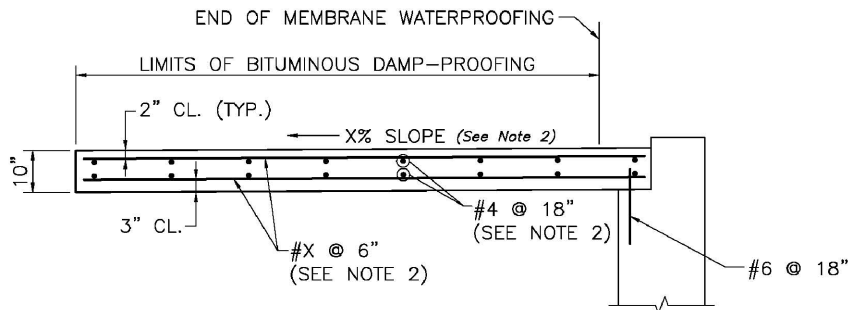


PLAN
NOT TO SCALE

NOTES:

1. Where angle Δ is greater or equal to 45° : length of 10" thick slab along center line of construction is 15'-0" and main reinforcement is #7 @ 6" top and bottom parallel to centerline of construction.
2. Where angle Δ is less than 45° : length of 10" thick slab perpendicular to abutment is 10'-0" minimum and main reinforcement is #6 @ 6" top and bottom perpendicular to abutment.
3. If removable panel is used, place steel parallel to centerline of construction and design approach slab accordingly.
4. Place #4 @ 18" top and bottom parallel to abutment and between main reinforcement.
5. Width of approach slab may have to be reduced if it interferes with utilities.

Figure 4.20. MassDOT approach slab plan (2009)



NOTES:

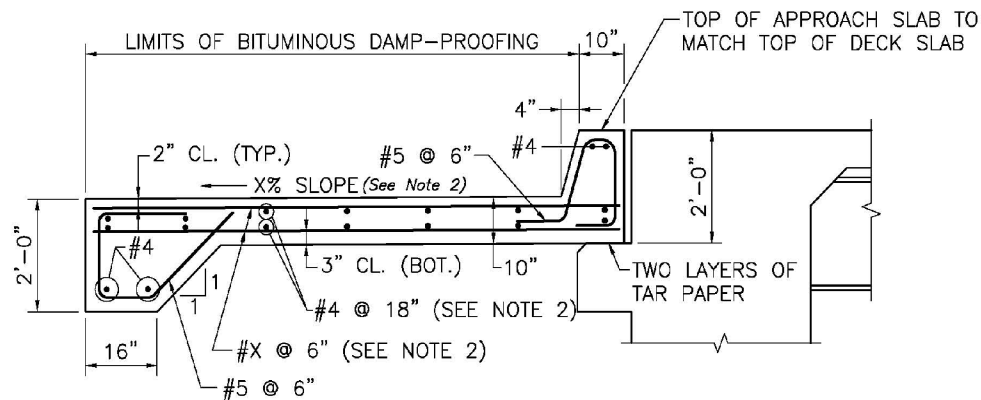
1. APPROACH SLAB TO BE 4000 PSI, $1\frac{1}{2}$ IN, 565 CEMENT CONCRETE.
2. PLACE LONGITUDINAL REINFORCEMENT (Specify orientation per Dwg. No. 3.1.12). PLACE TRANSVERSE REINFORCEMENT PARALLEL TO ABUTMENT.

APPROACH SLAB DETAILS
SCALE: $\frac{1}{2}" = 1'-0"$

NOTES:

1. See Roadway Section drawings for abutment details and dimensions not shown here.
2. If approach roadway slopes down and away from the abutment at greater than 2%, change slope of approach slab to approach roadway grade plus 1%, rounded up to the nearest 1%. Otherwise set slope at 2%.

Figure 4.21. MassDOT typical approach slab details: approach slab Type 1 detail (2009)



NOTES:

1. APPROACH SLAB TO BE 4000 PSI, 1½ IN, 565 CEMENT CONCRETE.
2. PLACE LONGITUDINAL REINFORCEMENT (Specify orientation per Dwg. No. 3.1.12).
PLACE TRANSVERSE REINFORCEMENT PARALLEL TO ABUTMENT.

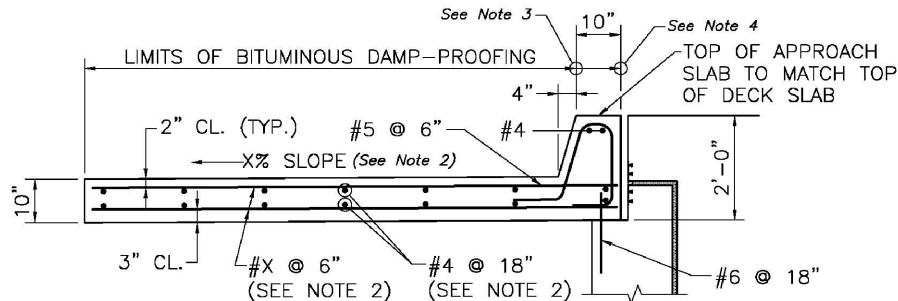
APPROACH SLAB DETAILS

SCALE: ½" = 1'-0"

NOTES:

1. See Roadway Section drawings for abutment details and dimensions not shown here.
2. If approach roadway slopes down and away from the abutment at greater than 2%, change slope of approach slab to approach roadway grade plus 1%, rounded up to the nearest 1%. Otherwise set slope at 2%.

Figure 4.22. MassDOT typical approach slab details: approach slab Type 2 detail (2009)



NOTES:

1. APPROACH SLAB TO BE 4000 PSI, 1½ IN, 565 CEMENT CONCRETE.
2. PLACE LONGITUDINAL REINFORCEMENT (Specify orientation per Dwg. No. 3.1.12).
PLACE TRANSVERSE REINFORCEMENT PARALLEL TO ABUTMENT.

APPROACH SLAB DETAILS

SCALE: ½" = 1'-0"

NOTES:

1. See Roadway Section drawings for abutment details and dimensions not shown here.
2. If approach roadway slopes down and away from the abutment at greater than 2%, change the slope of approach slab to approach roadway grade plus 1%, rounded up to the nearest 1%. Otherwise set slope at 2%.
3. End Bituminous damp-proofing here when using Asphaltic Bridge Joints.
4. End Bituminous damp-proofing here when using armored joints.

Figure 4.23. MassDOT typical approach slab details: approach slab Type 3 detail (2009)

Figure 4.24 shows different paving notch configurations for the lowered approach slabs. The location of the paving notch in Figure 4.24 is either located above the beam seat construction joint or below the beam seat construction joint.

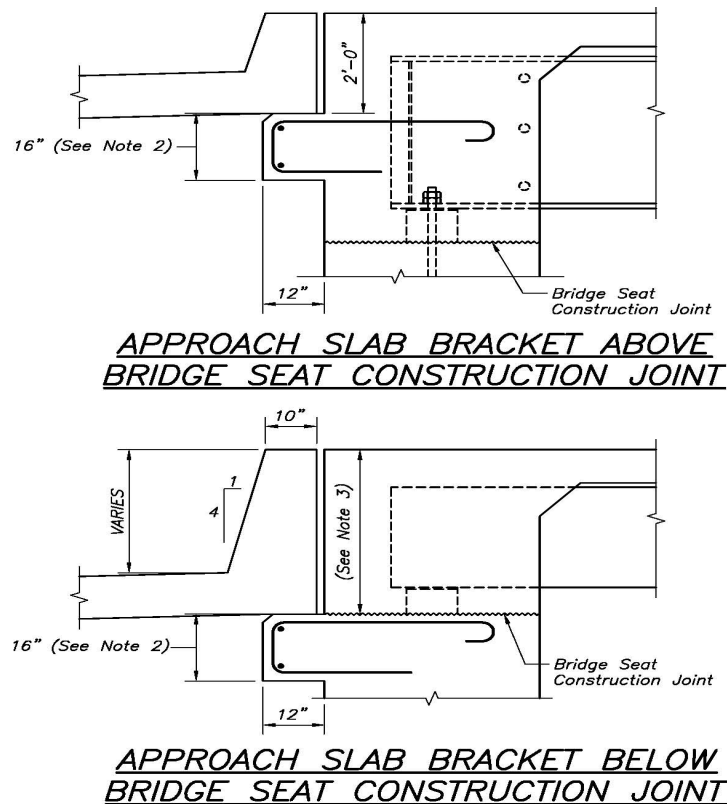


Figure 4.24. MassDOT paving notch details for lowered approach slabs (2009)

4.7.2. Embankment Material

The material used for embankments in Massachusetts shall consist of solid, sound mineral aggregate that is free of deleterious, organic, elastic or foreign matter and shall be graded for satisfactory compaction (MassDOT, 2008). The material shall meet the requirements of one of seven possible material types: ordinary borrow, gravel borrow, sand borrow, gravel borrow for bridge foundations, special borrow, impervious soil borrow, and crushed stone for bridge foundations. The seven material types are described as follows:

Ordinary Borrow: Ordinary borrow is material that is not specified as gravel borrow, sand borrow, special borrow, or a particular kind of borrow that is designated at A-1, A-2, A-3 by AASHTO-M145. The material must be able to be spread and compacted into embankments.

Special Borrow: Special borrow can either consist of a native in situ soil or crushed rock. The native soil is classified under AASHTO-M145 as A-3 or that portion of A-1 with less than 12% passing the No. 200 sieve. The crushed rock must have 50% maximum wear for the LA abrasion test, a plasticity index of 6% maximum, and a gradation shown in Table 4.8.

Table 4.8. Special borrow crushed rock gradation (MassDOT, 2008)

Sieve Size	Total Percent Passing
6 inch	100
2 inch	90 to 100
No. 4	20 to 65
No. 200	0 to 12

Gravel Borrow: Gravel borrow is inert material that is hard, durable stone and coarse sand, free from loam and clay, surface coatings, and deleterious materials. The gradation requirements are shown in Table 4.9. The maximum size of stone can be specified by the engineer at 6 in., 3 in., or 2 in. Gravel for bridge foundations has the same requirements for gradation, however, the largest particle size allowed is 6 in.

Table 4.9. Gravel borrow gradation requirements (MassDOT, 2008)

Sieve Size	Total Percent Passing
1/2 inch	50 to 85
No. 4	40 to 75
No. 50	8 to 28
No. 200	0 to 10

Sand Borrow: Sand borrow needs to consist of clean inert, hard, durable grains of quartz or other hard durable rock, free from loam or clay, surface coatings and deleterious materials. The allowable amount to material passing the No. 200 sieve shall not exceed 10% and the maximum particle size is 3/8 in.

Impervious Soil: An impervious soil must conform to one of the following AASHTO-M145 A-4, A-5, A-6, A-7, A-2 soils containing more than 20% passing the No. 200 sieve. All material shall be free of stumps, brush, and stones larger than 3 in. in diameter.

Crushed Stone: Crushed stone can consist of durable crushed rock or crushed gravel stone. The crushed rock shall be made of angular fragments obtained by breaching and crushing solid or shattered natural rock. The material must be free from (less than 15% by weight) thin, flat, elongated or other objectionable pieces. The crushed gravel must be made from boulders or fieldstone with a minimum diameter of 8 in. before crushing. Both types of crushed stone shall be freed of clay, loam, and have the gradation as shown in Table 4.10.

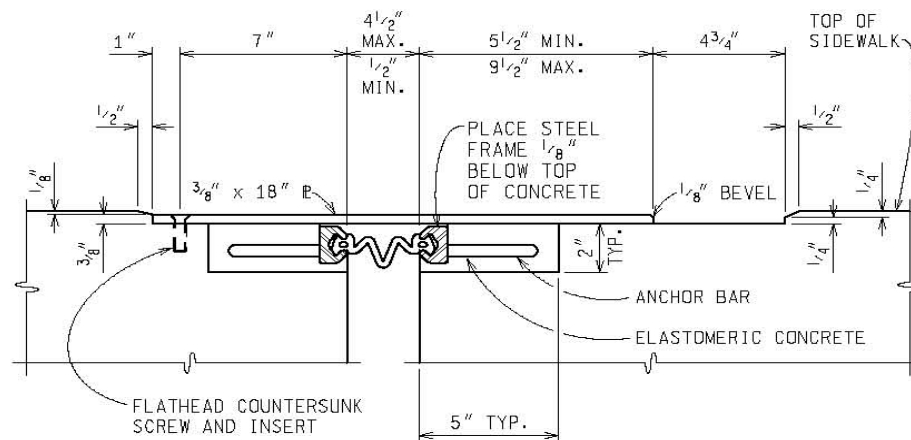
Table 4.10. Crushed stone gradation (MassDOT, 2008)

Sieve Size	Total Percent Passing
3 inch	100
1 1/2 inch	95 to 100
1 inch	35 to 70
3/4 inch	0 to 25

4.8. Michigan

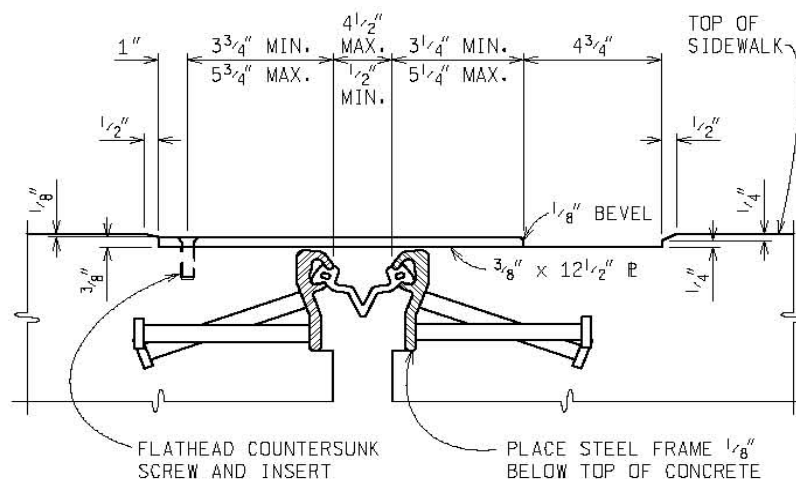
4.8.1. Standards, Specifications, and Details

Typical expansion joint details used by the Michigan DOT (MDOT) (2009) require a continuous neoprene seal across the deck as shown in Figure 4.25. MDOT expansion joint devices are similar to the ODOT strip seal shown in Figure 3.9. MDOT permits the use of proprietary products and lists D.S. Brown Co., Watson-Bowman & Acme, Inc., and Structural Rubber Products Co. as acceptable makers of expansion joint devices.



SECTION THROUGH EXPANSION JOINT
AND COVER PLATE

a. Expansion joint with block-outs



SECTION THROUGH EXPANSION JOINT
AND COVER PLATE

b. Expansion joint anchored into deck

Figure 4.25. MDOT strip seal expansion joints (2009)

4.8.2. Embankment Material

The materials used for embankments specified by the MDOT can be Granular Material Class II or Class III. The different embankment materials are described as follows:

Granular Material: Granular material consist of sand, gravel, crushed stone, iron blast furnace slag, reverberatory furnace slag or a blend of aggregates conforming to the grading requirements of Table 4.11. When Class II material is specified MDOT allows Class I material to substituted. Similarly, if Class III is specified then Class I, Class II, Class IIA, or Class IIIA can be substituted.

Table 4.11. Granular Material Gradation

Sieve Size (a)	Percent Passing for Material Class				
	Class I	Class II	Class IIA	Class III	Class IIIA
6 inch				100	
3 inch		100	100	95 to 100	
2 inch	100				
1 inch		60 to 100	60 to 100		
1/2 inch	45 to 85				
3/8 inch					100
No. 4	20 to 85				
No. 30	5 to 30				
No. 100		0 to 30	0 to 35		0 to 30
No. 200 (b)	0 to 5	0 to 7	0 to 10	0 to 15	0 to 15

a. Test results based on dry weight

b. Use test method MTM 108 for Loss by Washing

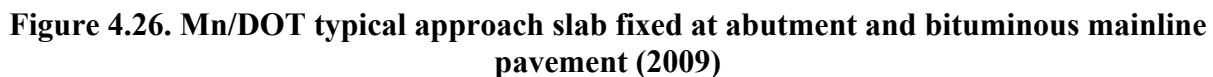
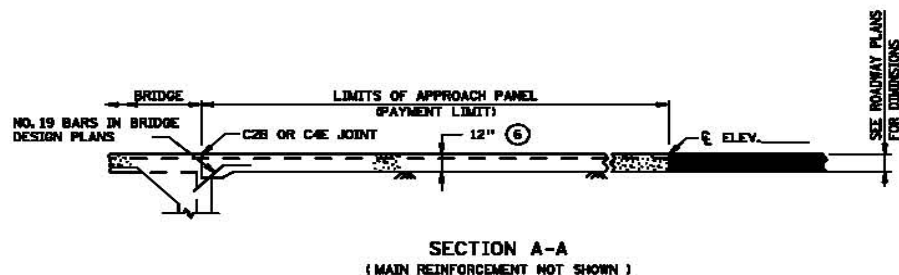
4.9. Minnesota

4.9.1. Standards, Specifications, and Plans

The Minnesota DOT (Mn/DOT) (2009) has two typical details for approach slabs used on integral abutments. The approach detail shown in Figure 4.26 is used when the mainline pavement is bituminous. The mainline bituminous pavement is placed directly against the end of the approach slab with no expansion joint. The second detail for the approach slab is used when the mainline pavement approach the bridge is made of concrete as shown in Figure 4.27. The approach slab details have the same connection to the abutment. The approach slab with concrete pavement has a sleeper slab located below the approach slab and pavement interface. The interface between the slab and pavement has an expansion joint as shown in Figure 4.28b. The joint is 4 in. wide and is filled with joint sealer and is not doweled. The pavement is anchored to the sleeper slab with vertical hook bars shown in Figure 4.28. The mainline pavement also has two rows of 1 ft by 1 ft keys located approximate 20 ft away from the approach slab expansion joint. The keys extend into the base material, as show in Figure 4.28b, and are used to anchor the pavement from horizontal movement.

Figure 4.28a shows the approach slab tied to the sleeper slab with vertical reinforcement. The E&S expansion joint is located to account for horizontal movement of the concrete pavement.

DIVIDED-URBAN ROADWAY PLAN
PARALLEL & NONPARALLEL WINGWALLS



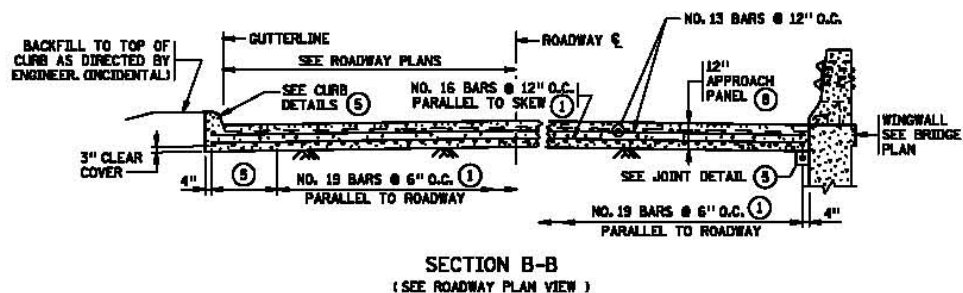
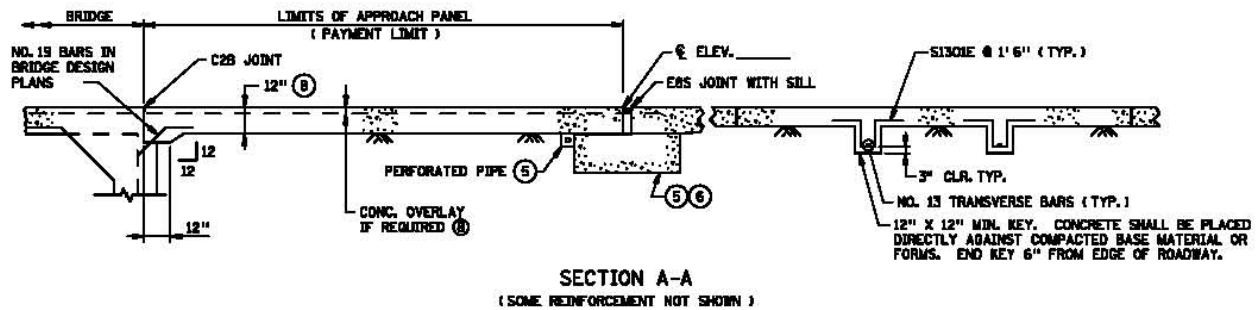
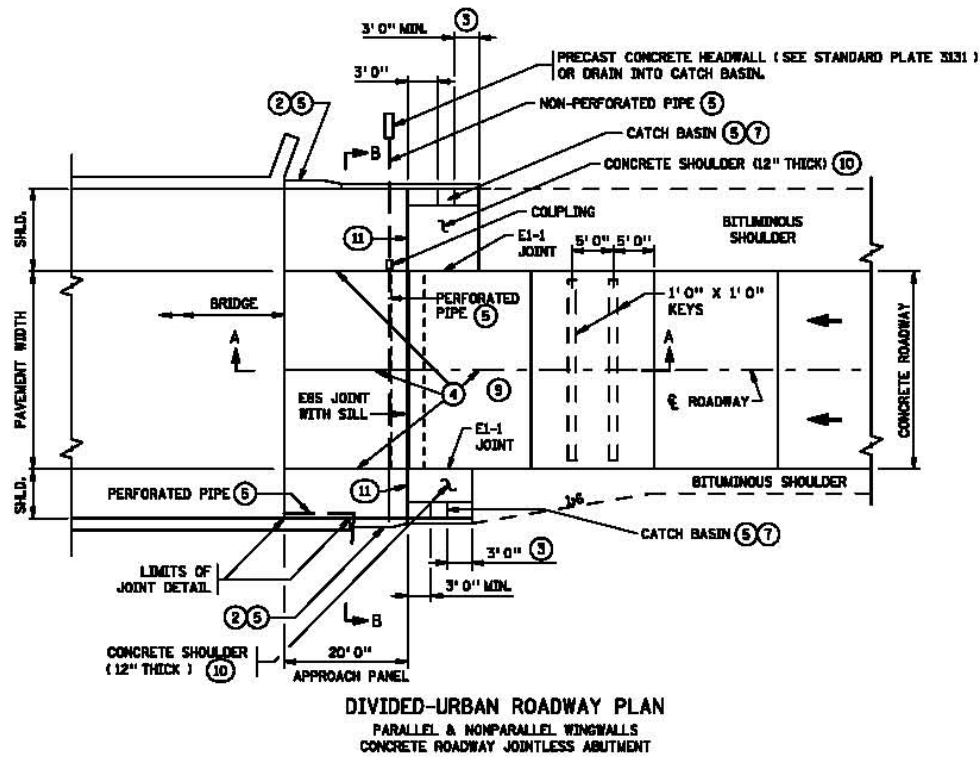
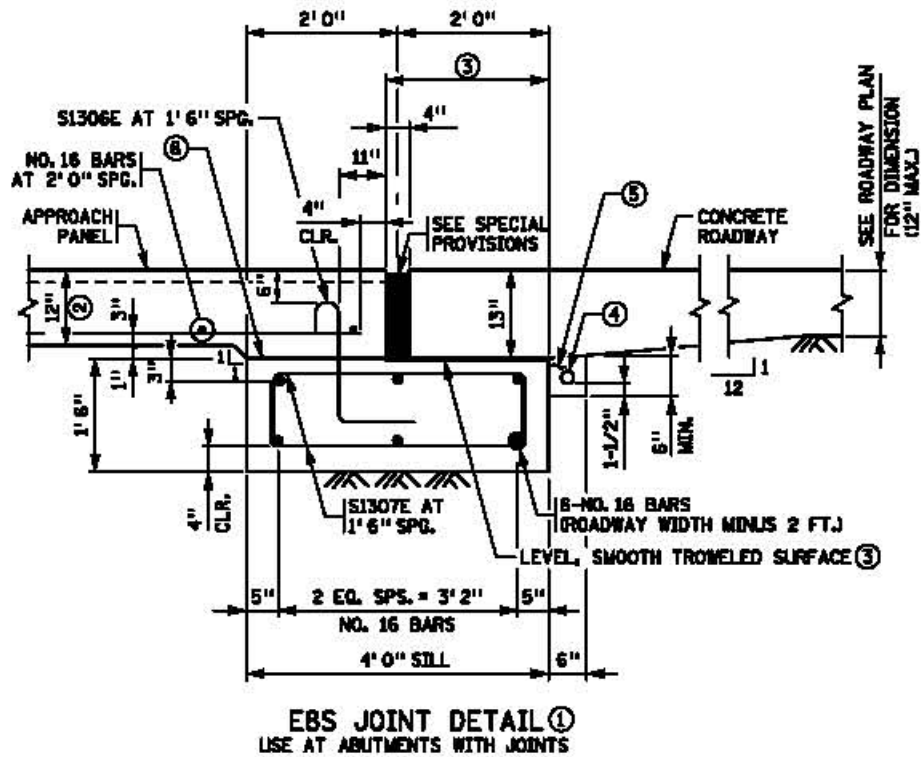
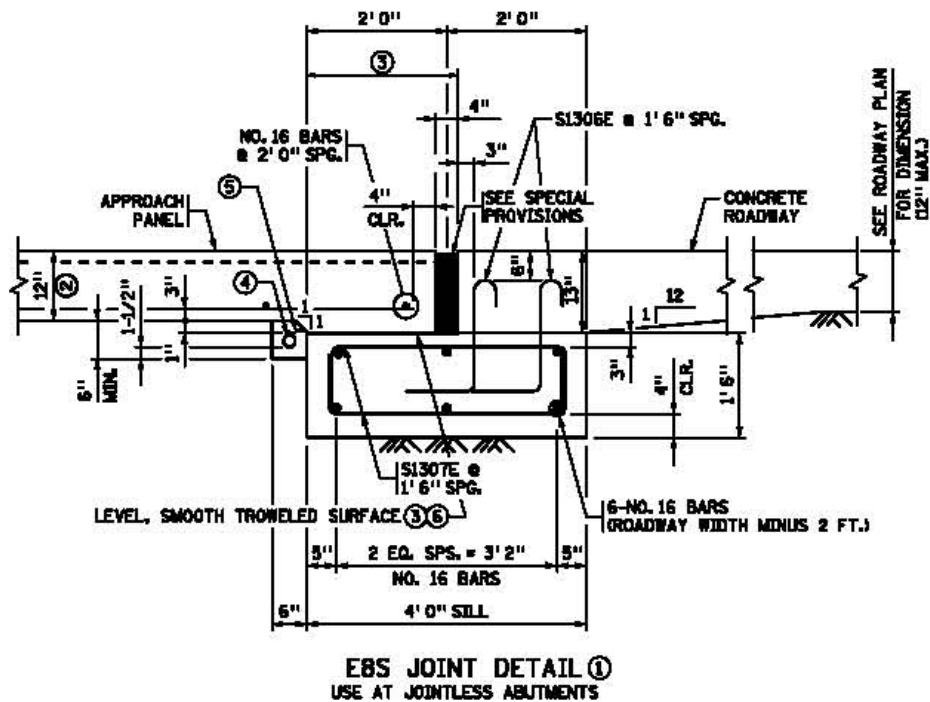


Figure 4.27. Mn/DOT typical approach slab fixed at abutment and concrete mainline pavement (2009)



a. Sleeper slab with bridge expansion joint at abutment



b. Sleeper slab with bridge expansion joint at end of approach slab

Figure 4.28. Mn/DOT typical sleeper slab details with EBS expansion joint (2009)

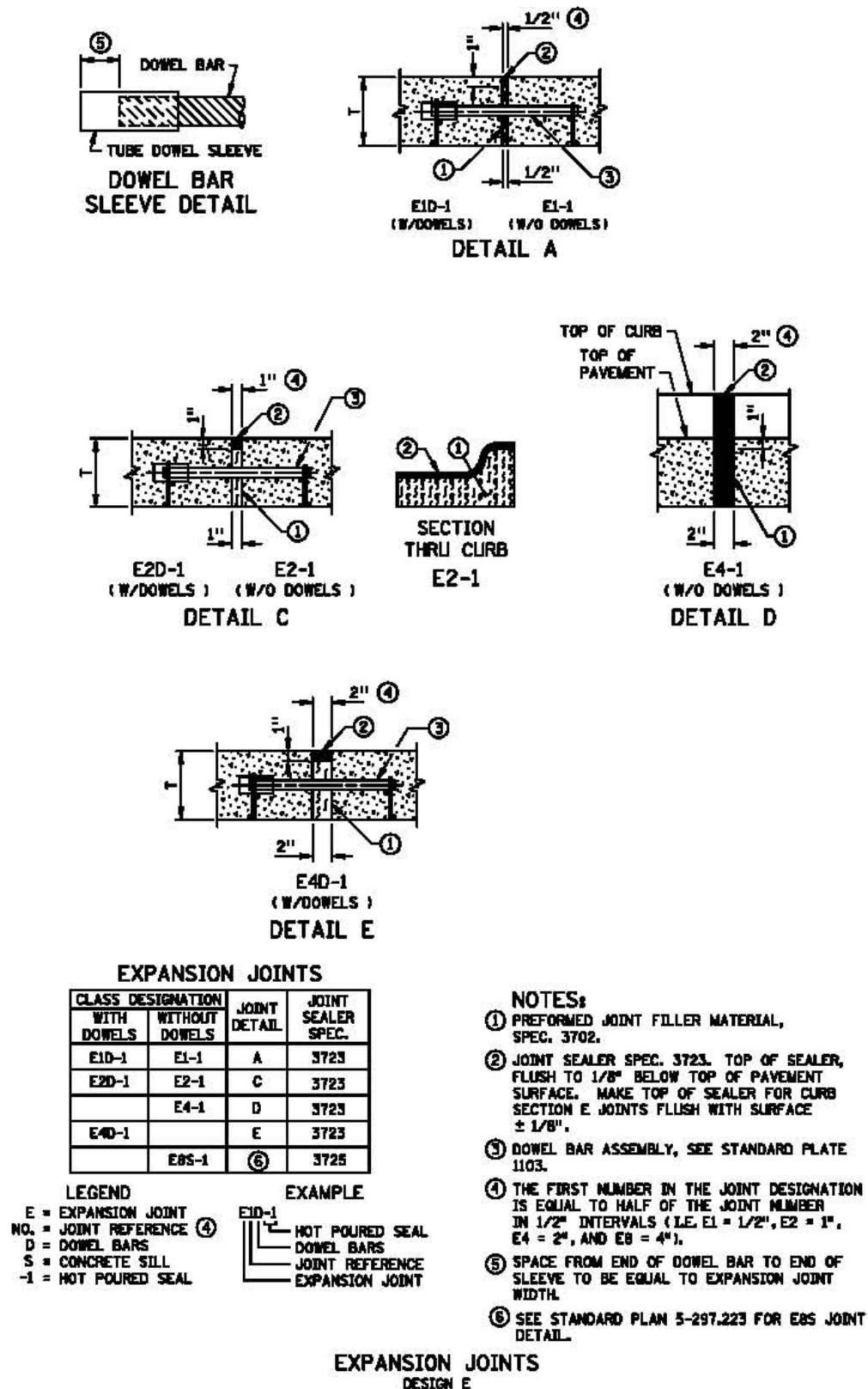


Figure 4.29. Mn/DOT typical expansion joint details (2009)

Paul Rowekamp provided information on the practices in Minnesota (Greimann et al., 2008). He reported that Minnesota has been building integral abutment bridges for approximately five to six years and connecting the approach slabs to the bridge for the last three years. The standard detail, shown in Figure 4.30, is to extend a reinforcing bar diagonally from the abutment into the approach slab. This connection was implemented because of maintenance concerns pertaining to the opening of the joint between the slab and bridge. He explained that after the bridge has expanded to its limits and begins to contract, the slab may not move with the bridge immediately, because of friction with soil and lack of friction between the slab and the paving notch. Thus the joint opens slightly, filling with debris. The next season the same thing happens, filling the joint with more debris. The slab now has less to rest on, and water can now flow in and beneath the slab. As the slab approaches the edge of the paving seat, it may eventually fall completely off. Rowekamp reported that the initial connection design used an 8 ft horizontal bar extending 4 ft each way into the slab and bridge deck. Transverse cracking across the entire approach slab appeared approximately where the horizontal bar ended, possibly caused by rotation of the slab being restrained. Two years ago a change was made to the current detail, and no problems have been reported thus far. Minnesota standard details do not call for any friction-reducing material below the approach slab. Backfill of the abutment is specified as modified select granular material (having no fines) and is installed in typical lifts and compacted.

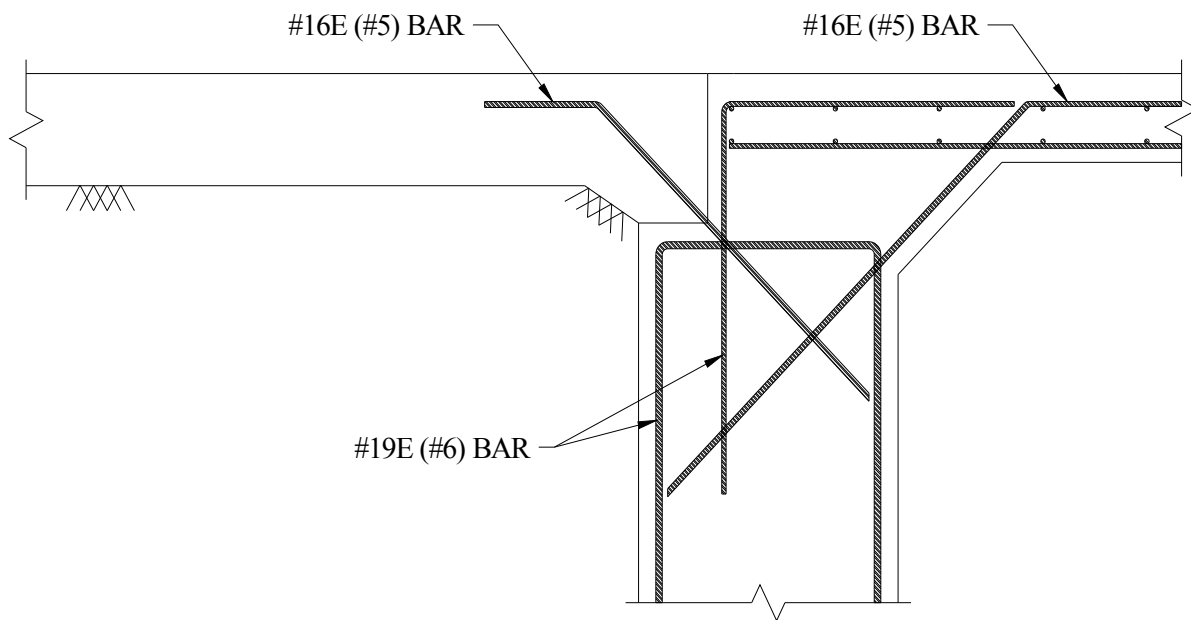
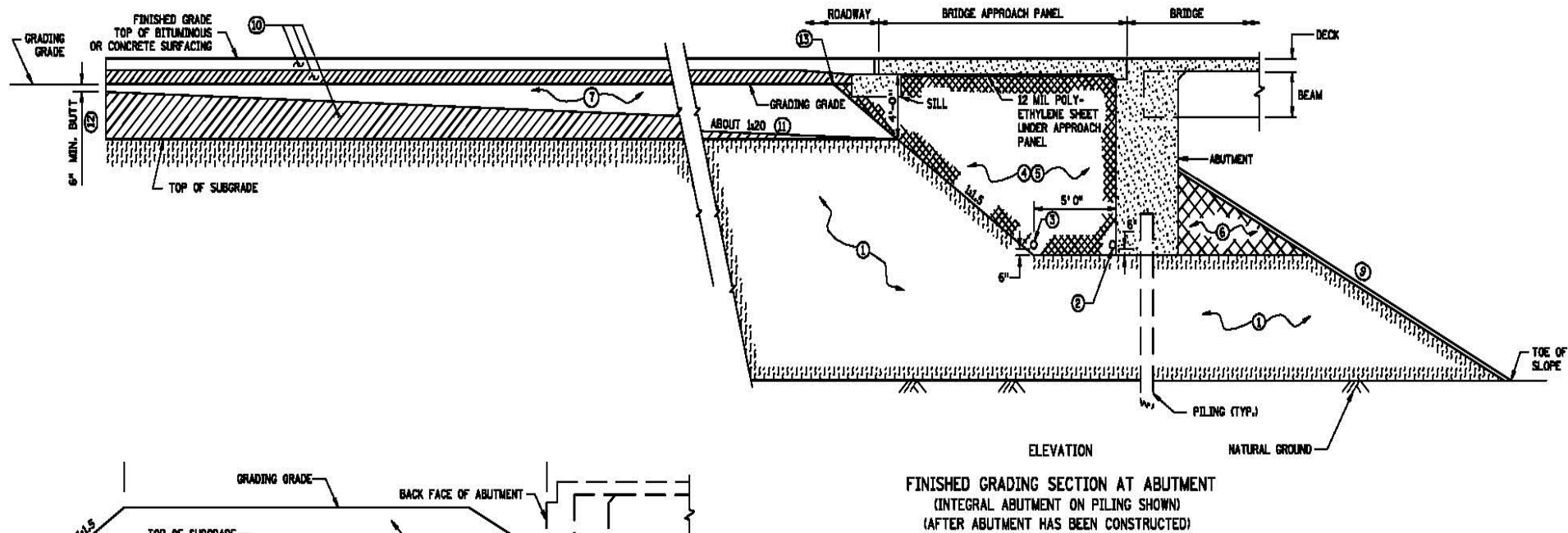


Figure 4.30. Typical Minnesota detail (Greimann et al., 2008)

Figure 4.31 shows the typical finish grading section for Mn/DOT integral abutment bridges. The bottom layer of soil for the embankment material is natural soil or a suitable graded material. The next layer starts at the bottom of the pile caps and is a select granular material. Two subsurface pipe drains are placed at the interface between the base material and the select granular material. Under the approach slab, a 12 mil polyethylene sheet of plastic is placed as a barrier between the approach slab and the select granular material.



NOTES:

- ① NATURAL GROUND OR SUITABLE GRADING MATERIAL.
- ② SUBSURFACE PIPE DRAIN. SEE BRIDGE PLAN FOR STANDARD DETAIL B910 FOR DETAILS.
- ③ SUBSURFACE PIPE DRAIN. SEE GRADING PLAN FOR DETAILS. FURNISH AND INSTALL IF SHOWN IN GRADING PLAN.
- ④ SELECT GRANULAR MATERIAL MODIFIED 10% SHALL COMPLY WITH SPEC. 3149.282, MODIFIED TO 10% OR LESS PASSING THE NUMBER 200 SIEVE.
- ⑤ QUANTITY OF SELECT GRANULAR MATERIAL MODIFIED 10% IS BASED ON DIMENSIONS SHOWN AND PAYMENT IS BASED ON THIS QUANTITY. SEE GRADING PLAN FOR QUANTITY. Mn/DOT SPEC. 1903 SHALL NOT APPLY IF THE CONTRACTOR CHOOSES TO INCREASE DIMENSIONS IN ORDER TO FACILITATE CONSTRUCTION OPERATIONS AND ANY QUANTITY INCREASES SHALL BE CONSIDERED INCIDENTAL.
- ⑥ SUITABLE GRADING MATERIAL.
- ⑦ BACKFILL MATERIAL SHALL COMPLY WITH SPEC. 3149.282 (SELECT GRANULAR BORROW).
- ⑧ SURCHARGE MATERIAL SHALL BE PLACED DURING ROADWAY EMBANKMENT CONSTRUCTION. THIS SAME MATERIAL TO BE REMOVED AS STRUCTURE EXCAVATION JUST PRIOR TO THE ABUTMENT CONSTRUCTION (SEE BRIDGE PLAN FOR METHOD OF PAYMENT AND QUANTITIES). EXCAVATION LIMITS ARE SHOWN.
- ⑨ SEE BRIDGE PLANS FOR SLOPE AND SLOPE PROTECTION.
- ⑩ SEE GRADING PLANS FOR TYPE OF MATERIAL.
- ⑪ START 1:20 TAPER AT END OF APPROACH PANEL. 1:20 VARIES WHEN APPROACH PANEL IS SKEWED.
- ⑫ GRADING TO BE SQUARED OFF ON SKEWED APPROACHES.
- ⑬ TOP OF 1:1.5 SLOPE FORMS A LINE PARALLEL TO END OF BRIDGE.

Figure 4.31. Mn/DOT integral abutment finished grading section (2009)

4.10. Missouri

4.10.1. Standards, Specifications, and Details

The typical approach slab plan and section used by the Missouri DOT (MoDOT) (2009) is shown in Figure 4.32 and 4.33 respectively. The approach slab shown in Figure 4.33 is connected to the abutment with horizontal #5 reinforcing bars at 12 in. on center. The end of the approach slab rests on a sleeper slab. Between the sleeper slab and the approach slab are two layers of building felt. According to the plan, a 3/4 in. joint is placed between the approach slab and pavement and filled with joint filler.

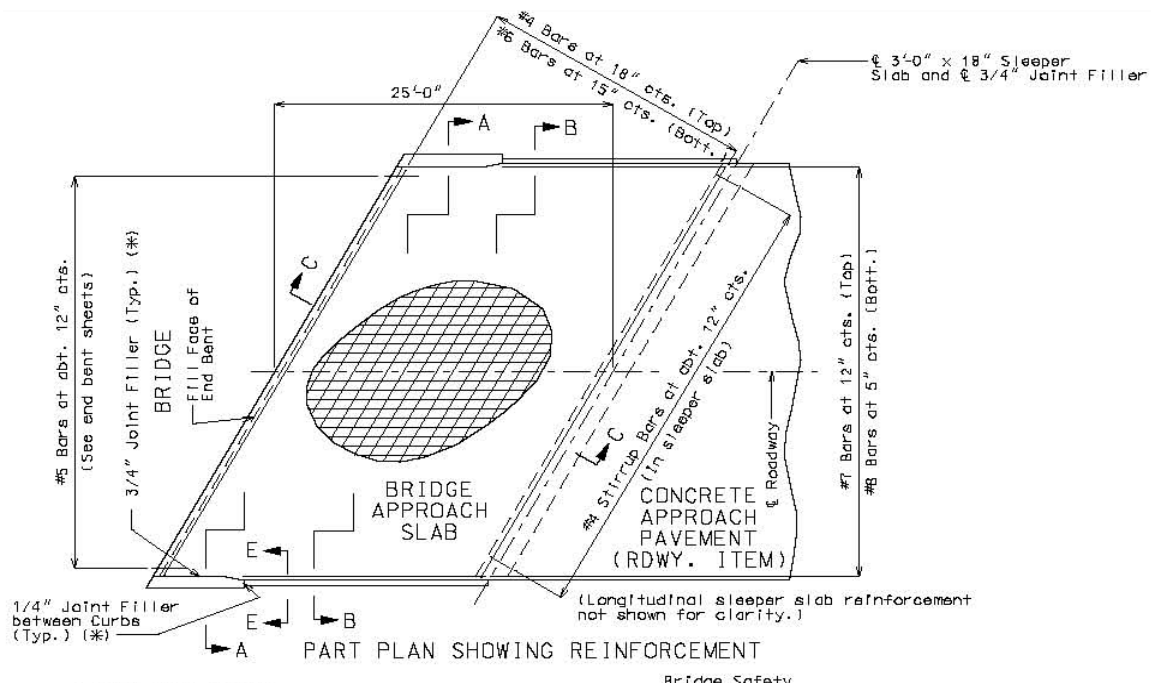


Figure 4.32. MoDOT typical approach slab plan (2009)

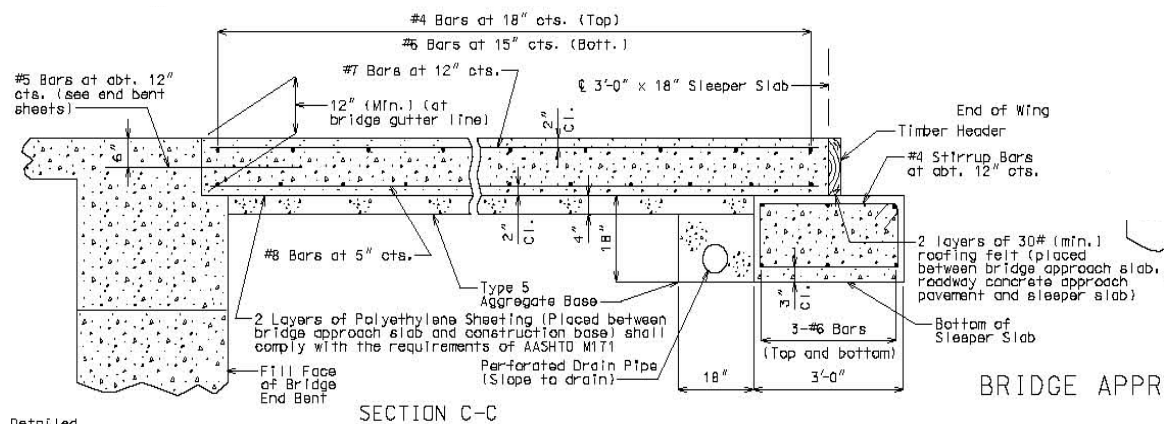


Figure 4.33. MoDOT typical approach slab section (2009)

David Straatmann with MoDOT indicated that connecting the approach slab to the bridge has been standard practice for some time (Greimann et al., 2008). The standard connection method, shown in Figure 4.34., is made by extending #5 reinforcing bars, spaced at 12 in. horizontally, between the bridge deck and approach slab. Two layers of polyethylene sheeting are used between the approach slab and the construction base.

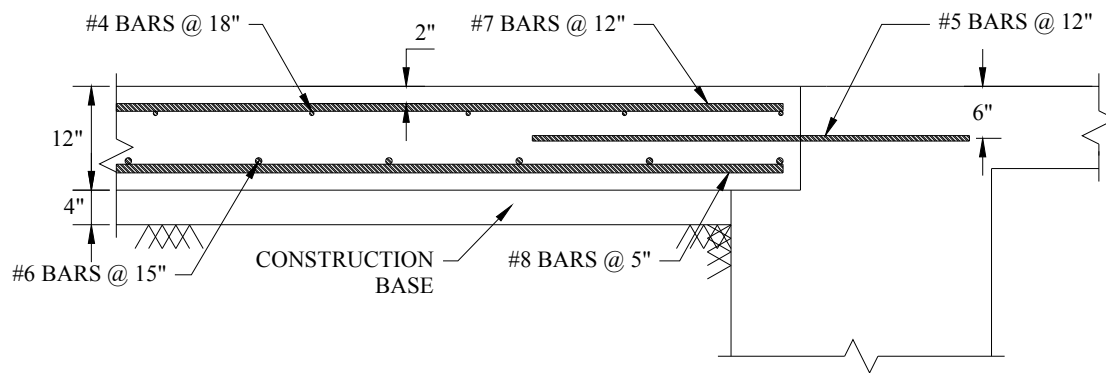


Figure 4.34. Typical Missouri detail (Greimann et al., 2008)

4.11. Nebraska

4.11.1. Standards, Specifications, and Details

In Nebraska, according to Scott Milliken, approach slabs have been used for the last 15 years, with connecting the slab to the bridge being the standard practice for at least the last 10 years (Greimann et al., 2008). The standard connection method, shown in Figure 4.35, is made by #6 reinforcing bars that extend vertically from the abutment, then bend 45 degrees into the approach slab. Nebraska refers to the approach slab as an approach section, which rests on a grade beam supported by piles at the end opposite the bridge. From the grade beam to the pavement, another transition section, called the pavement section, is used. According to Milliken, the reason for the connection was to move the bump from the end of the bridge to a location that is more easily maintained. This methodology also eliminated water from infiltrating the bearing of the bridge. A problem arising from the approach slabs was settlement of the sleeper slabs in the original design, leading to the use of grade beams as described above. Recently, hairline cracks, perpendicular to the grade beams on bridges with severe skews, were discovered. A top mat of steel was added in the approach slab, but no feedback was yet available. Overall, management is pleased with the performance thus far. There is nothing done to reduce the friction between the slab and the ground. Fill behind the abutment is considered only necessary until the concrete in the approach section reaches strength, at which time it acts like a bridge between the abutment and grade beam. Granular backfill is used, with drainage provided by drainage fabric. The material is installed in lifts and compacted with smaller equipment to avoid damaging the wing-walls.

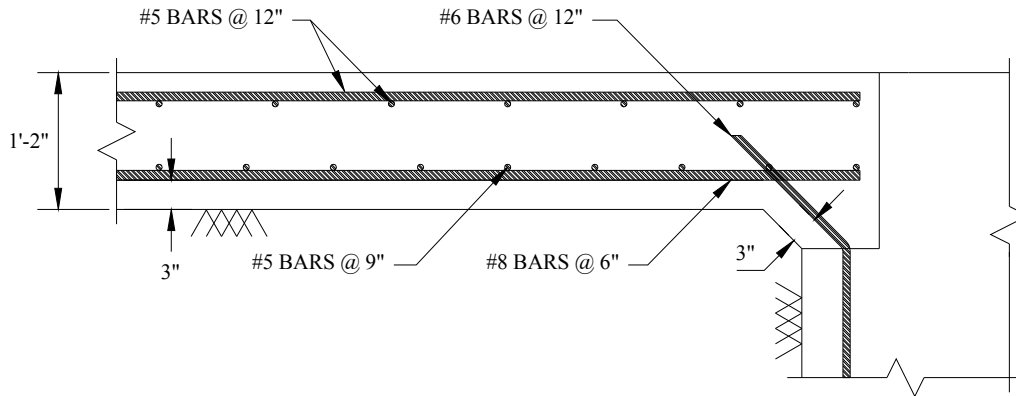


Figure 4.35. Typical Nebraska detail (Greimann et al., 2008)

4.12. New Hampshire

4.12.1. Standards, Specifications, and Details

The strip seal used by the New Hampshire DOT (NHDOT) (2009) is shown in Figure 4.36. NHDOT uses PVC drain pipes in the bridge deck to drain water from the bridge surface.

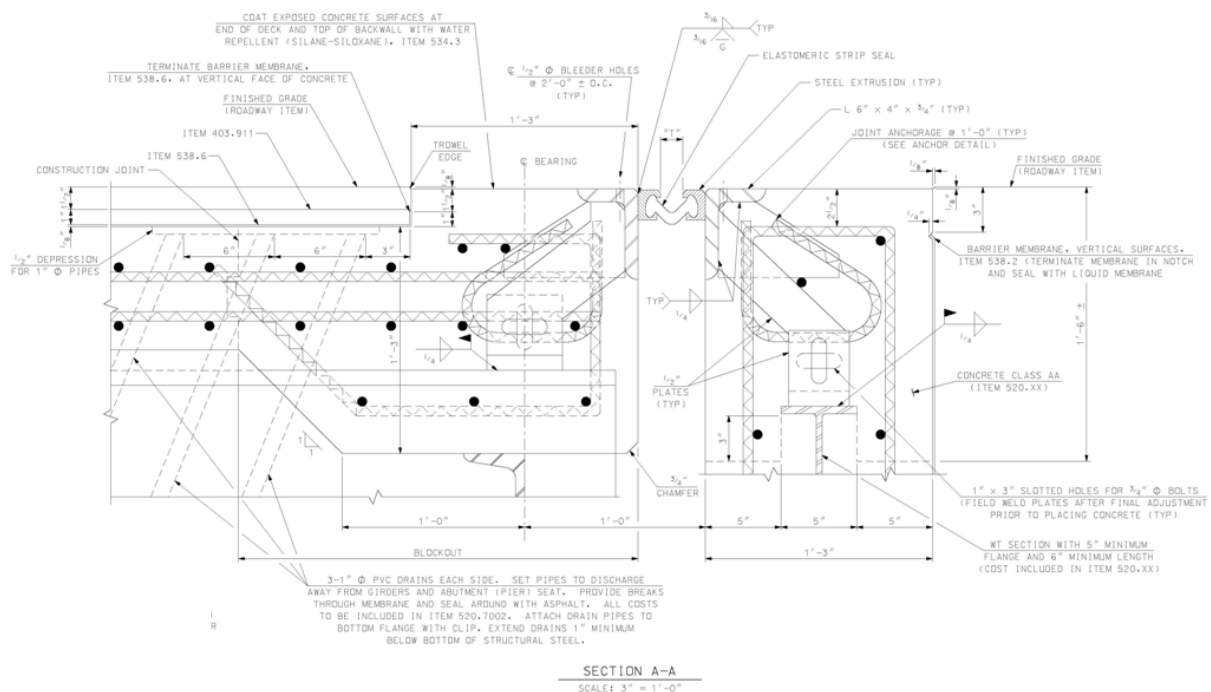


Figure 4.36. NHDOT typical strip seal detail (2009)

4.12.2. Embankment Material

NHDOT states that embankment material must conform to AASHTO M 57 using the definitions given in AASHTO M 146. The material should be clean of any saturated or unsaturated natural or man-made material. The density requirements for the material located under approach slabs and for material within 10 ft of the back of a structure not having an approach slab shall be compacted to at least 98% of the maximum density. Other materials not located in those areas are required to have 95% of maximum density compaction.

4.13. New Mexico

4.13.1. Current and Past Research

New Mexico also implemented a study to evaluate bridge approach settlement issues (Lenke, 2006). Nineteen bridges were identified in the state for the study. Observations of the 19 bridges revealed several detailing and construction strategies that can be implemented to improve performance. Listed are some of the bridge observations:

Good observations/recommendations:

1. The preventive measures for potential bridge, approach, and pavement settlement generally increase schedule time and cost but prevent future problems.
2. Provide good drainage and erosion control on the embankment underneath the bridge.
3. Maintain the joint between the bridge deck and approach slab. Cleaning and replacement is necessary to prevent stress buildup in the bridge, slab, and pavement.
4. Extend the approach and departure slabs the full width of the bridge. If the approach slab is only placed at the driving lanes, differential settlement can occur at the shoulder causing maintenance and safety issues.
5. Drainage gutters at the top of MSE walls and down embankments should be included in the design. Figure 4.37 shows a drainage gutter at the top of an MSE wall directing the water away from the bridge.

Poor observations:

1. Drop inlet drainage structures need to be positioned away from approach slabs and so they are not in the driving lanes. Drainage structures were found to settle less than the approach slab and pavement. Figure 4.38 shows a drainage structure next to the departure slab and in the driving lane.
2. Evidence of improper compaction of embankment material or construction on compressible foundation soils resulted in approach slab settlement, as well as longitudinal cracking, caused by fatigue deformation, in the asphalt wheel paths.
3. Poor drainage of water from the pavement or bridge was evident. Bad joints channel water below the slab and can cause significant erosion and undermining. Similar erosion patterns were seen below concrete projected slopes. Unmaintained joints and cracks channel water under the concrete slabs.

4. Poor construction quality assurance and quality control practices can be detrimental to long-term bridge rideability. Simple things, such as excessive lift thickness and low relative density, were found to highly affect the performance of bridges.



Figure 4.37. Drainage gutter used in New Mexico for moving water away from the bridge embankment (Lenke, 2006)

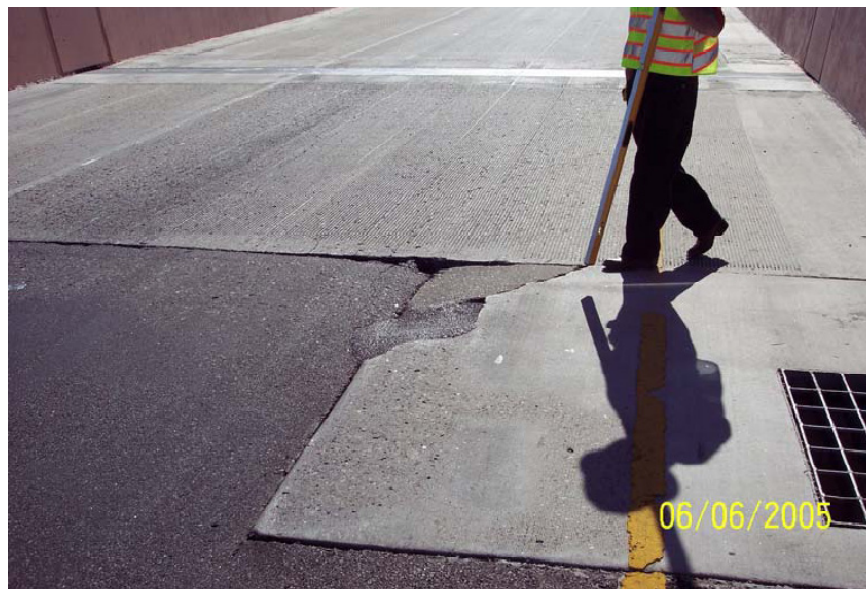


Figure 4.38. Settlement around drainage structure that is next to departure slab and in travel lane (Lenke, 2006)

4.14. New York

4.14.1. Current and Past Research

A report by Yannotti, Alampalli, and White (2006) discussed the New York State DOT (NYSDOT) experience with integral abutment bridges and presented specific practices. Of particular interest was the modification made to the approach slab to abutment connection after a 1996 study. The older detail involved the extension of bridge deck steel horizontally into the approach slab. This detail was found to be unsatisfactory because the approach slab was unable to accommodate any settlement. This settlement typically caused transverse cracking in the bridge deck and transverse and longitudinal cracking of the approach slab. A new detail, shown in Figure 4.39, was developed using reinforcing bars at 45 degrees into the bridge deck and the approach slab. This connection allows rotation of the slab by minimizing the rotational resistance at the slab-to-bridge connection.

Harry White of NYSDOT was contacted for further information. He added that the horizontal bar detail mentioned above provided negative moment capacity, so that when the fill and slab settled, rotation was restrained leading to the cracking discussed above. He also indicated that the new detail, seen in Figure 4.39, is performing adequately and no notable problems have arisen. A requirement of NYSDOT and other states is the use of a polyethylene sheet under the full width of the slab to reduce some friction.

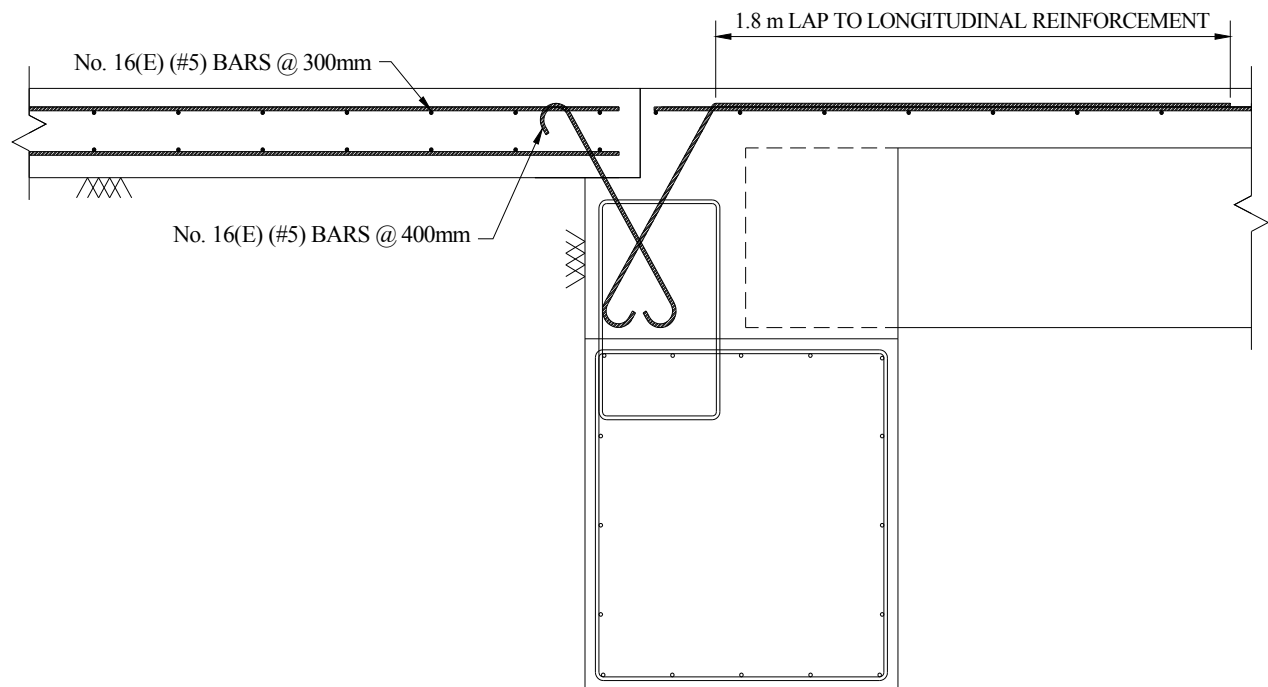


Figure 4.39. Typical New York detail (2009)

4.15. North Dakota

4.15.1. Standards, Specifications, and Details

According to Tim Schwagler of the North Dakota DOT (NDDOT), for approximately the last five years, the practice in North Dakota has been to connect the approach slab to the bridge (Greimann et al., 2008). This is accomplished by mechanically splicing a horizontal extension of #5 reinforcement from the bridge deck to the approach slab every 12 in. with joint filler (polystyrene) as shown in Figure 4.40. Two different types of approach slabs are used. On newer sites and newer embankment, the far end of the approach slab is supported on piles. When approach slabs are used on older sites where settlement is assumed to have already occurred in the embankment soil, the far end of the approach slab rests on the base course. This connection was implemented to improve performance of the joint between the approach slab and bridge. One inch joints were installed with filler and joint sealant. NDDOT found that the joints were opening and tearing the sealant. The connected joints have performed very well and no adjustments have been made. When the abutments are backfilled, a trench at the bottom 2 ft 6 in. deep is filled with rock wrapped in fabric with a drain pipe. Granular material ND Class 3 or 5 is then placed in 6 in. lifts and compacted.

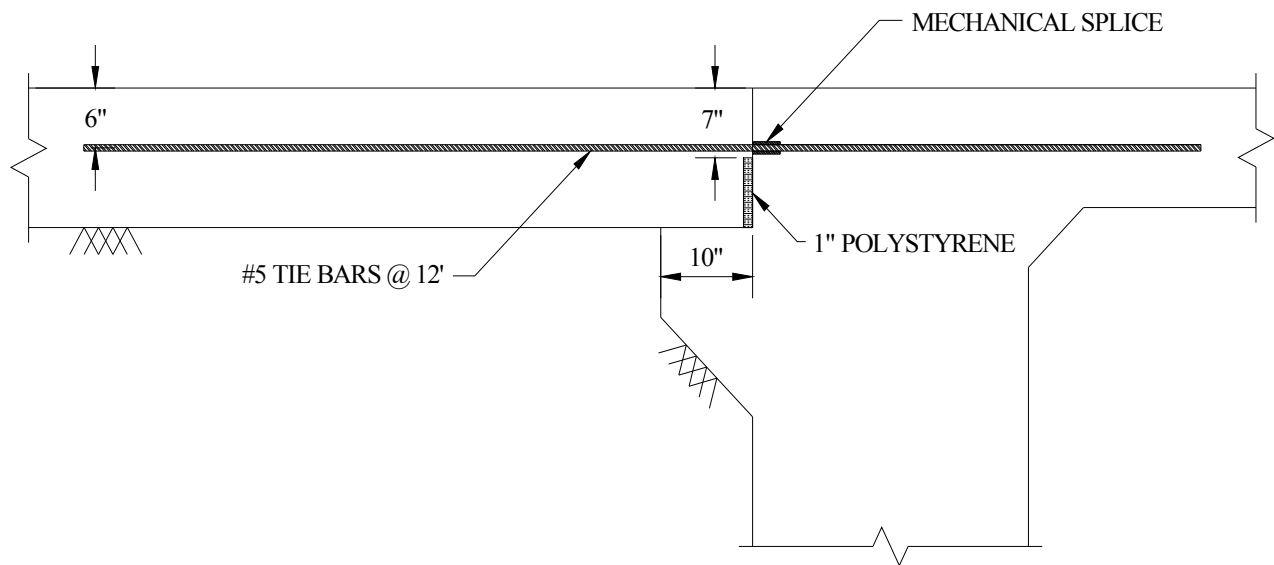


Figure 4.40. Typical North Dakota detail (Greimann et al., 2008)

Figure 4.41 show the typical approach slab drainage plan provided by NDDOT (2009). The drainage plan consists of providing surface water channels at each corner of the approach slabs. The water channels are constructed of 11 ft wide Turf Reinforcement Mats and extend down each side of the embankments to prevent the embankment from eroding.

Bridge Approach Slab Drainage Detail

D-708-4

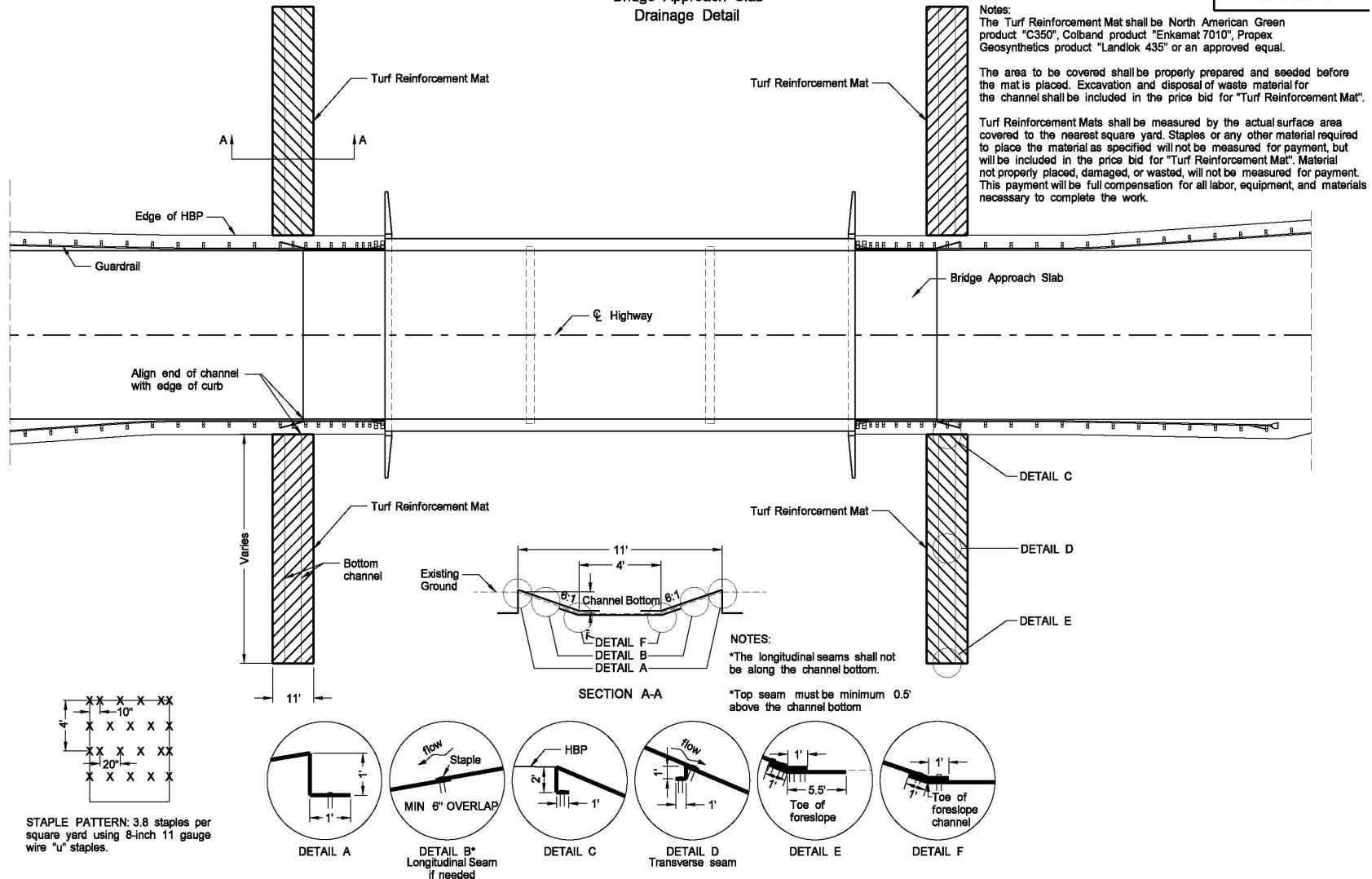


Figure 4.41. NDDOT approach slab drainage (2009)

4.16. Pennsylvania

4.16.1. Standards, Specifications, and Details

The Pennsylvania DOT (PennDOT) (2009) typical integral abutment is shown in Figure 4.42. Note that the girder is not shown for clarity. The connection between the pile cap and superstructure is provided by #5 vertical reinforcing bars at 9 in. on center in each face of the abutment. The #5 bars extend out of the pile cap a minimum of 2 ft 1 in. and are overlapped with #5 reinforcing bars in the superstructure.

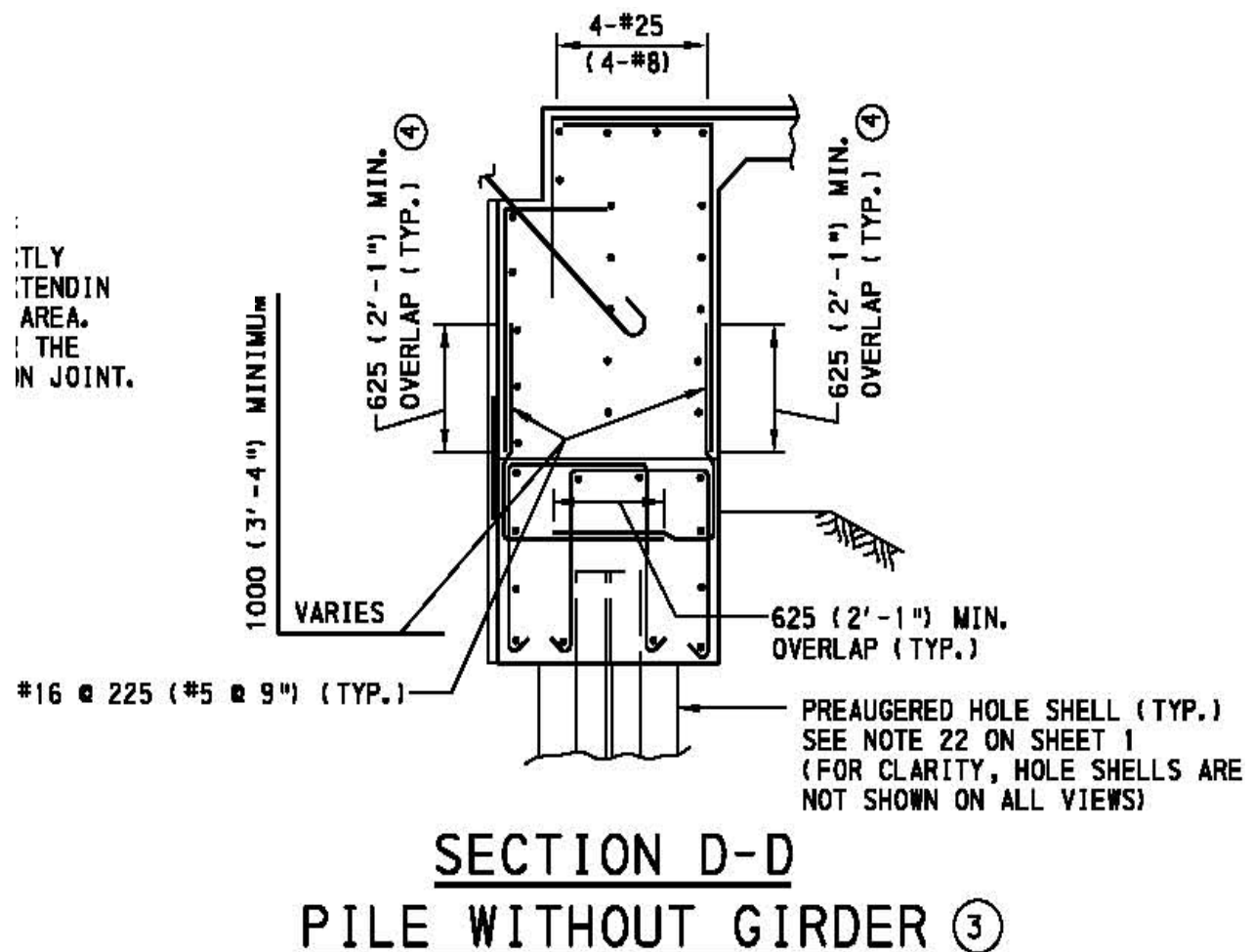


Figure 4.42. PennDOT typical integral abutment detail (2009)

The approach slab is connected to the abutment with a diagonal # 6 reinforcing bar at 9 in. on center as shown in Figure 4.43. The diagonal bar extends out of the corner edge of the 6 in. paving notch allowing rotation of the approach slab.

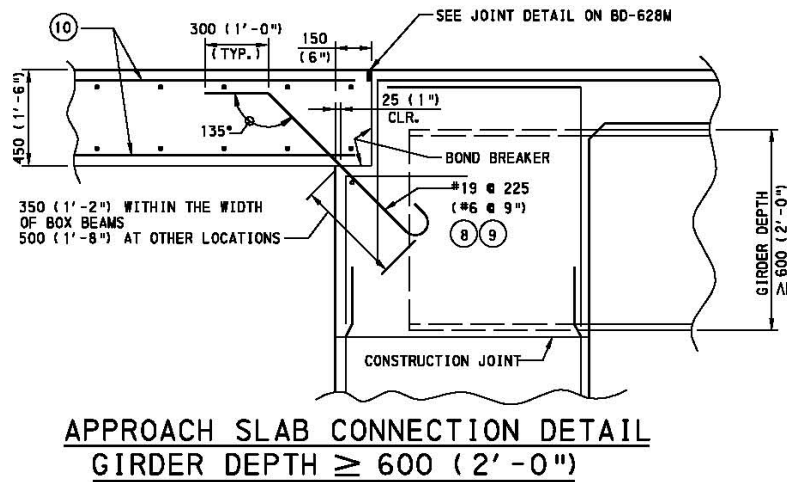


Figure 4.43. PennDOT approach slab connection to integral abutment detail (2009)

PennDOT has five types of approach slabs. Type 1, shown in Figure 4.44, is utilized when simply supported girders are used for the bridge superstructure. The bridge expansion joint placed between the girders and the abutment back wall are shown in Figure 4.45. PennDOT has two details for the expansion joint. Figure 4.45a shows the strip seal detail similar to that used by many other DOTs discussed herein. Figure 4.45b shows a tooth expansion joint with a water gutter that directs the water from the bridge deck away from the bridge abutment. In both scenarios, the approach slab is not attached to the abutment backwall with reinforcing.

The Type 1 approach slab can be used when either rigid or flexible pavement is present. The end of the approach slab rests on a sleeper slab as shown in Figure 4.46. Figure 4.46 shows the three different joint configurations that can be used at the approach slab pavement interface. Detail 1, shown in Figure 4.46a, is used with flexible pavement, while details 2 and 3, shown in Figure 4.46b and 4.46c, respectively, are utilized when concrete pavements are present. Detail 3 is similar to the ODOT relief joint with a narrow segment of asphalt placed between the approach slab and concrete pavement.

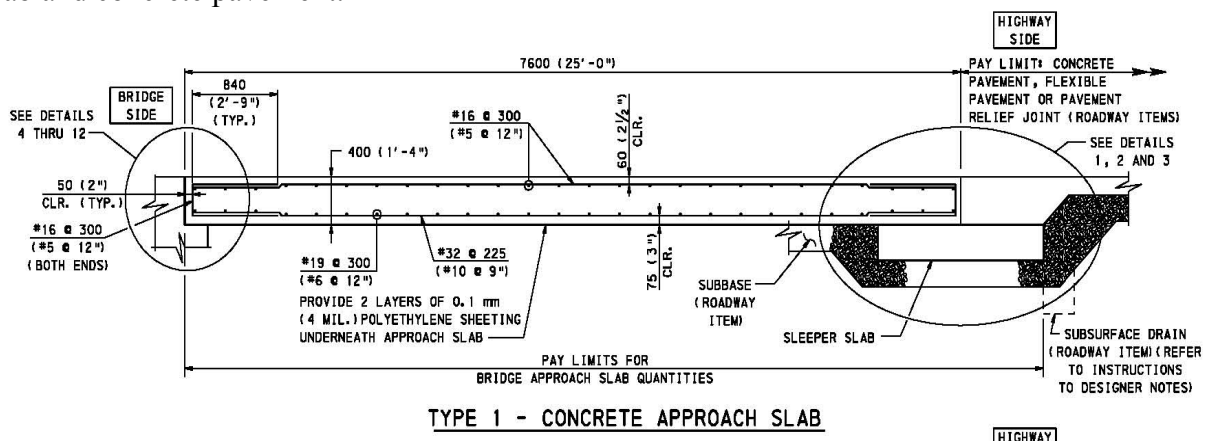
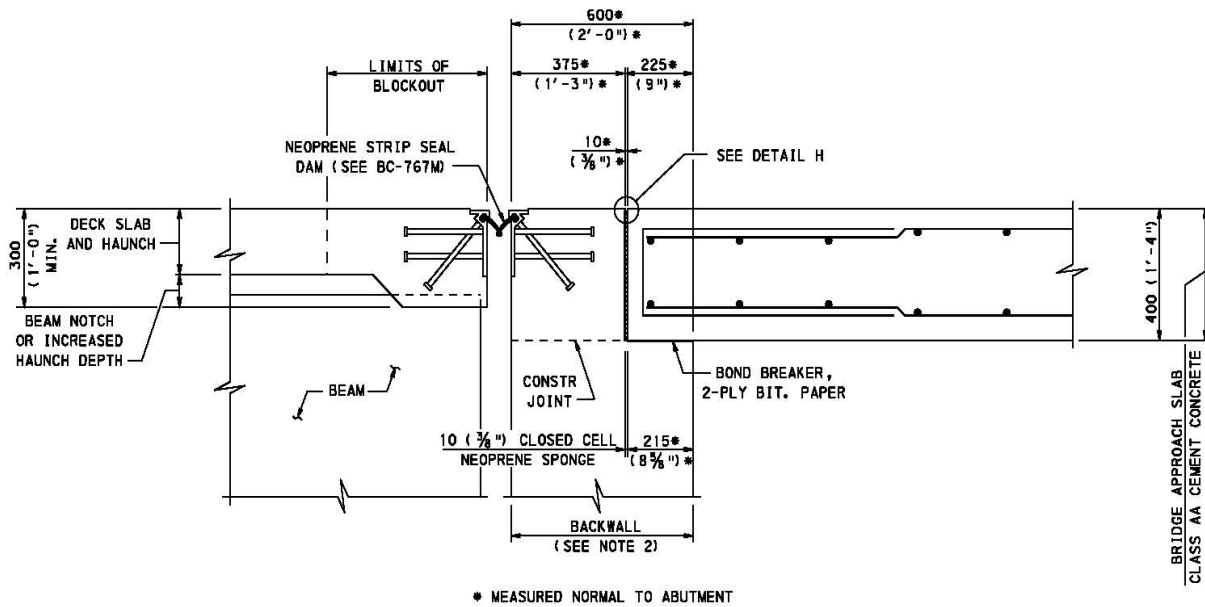


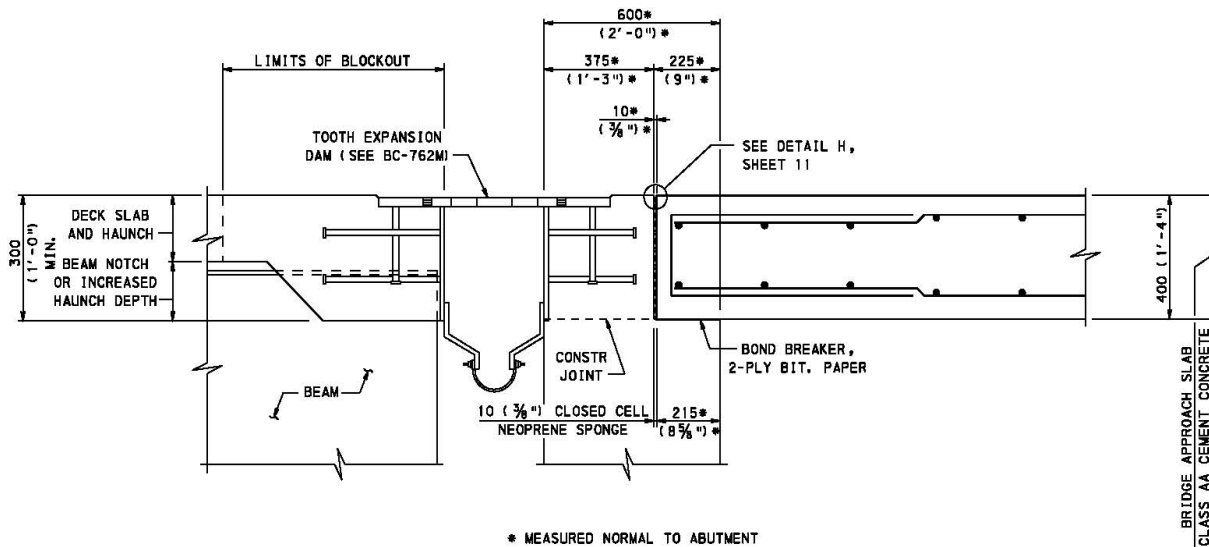
Figure 4.44. PennDOT Type 1 typical approach slab configuration for simply supported bridge girders (2009)



TYPE 1 APPROACH SLAB (WITHOUT OVERLAY) - DETAIL 4

APPROACH SLAB SUPPORTED ON ABUTMENT
BACKWALL WITH NEOPRENE STRIP SEAL DAM

a. Simply supported girder with neoprene strip seal

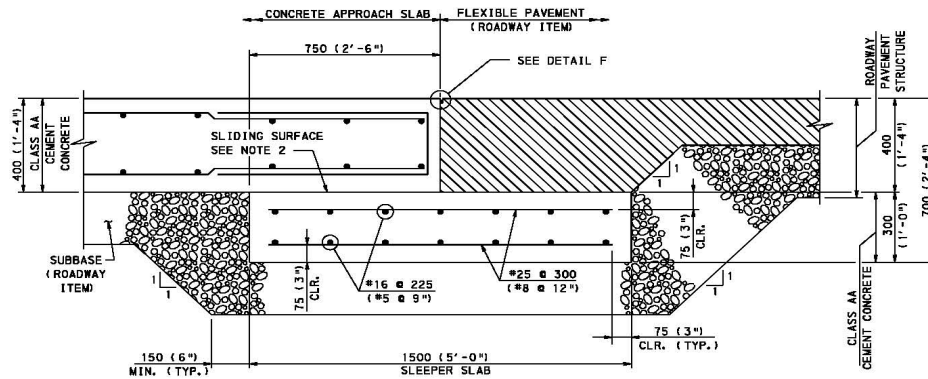


TYPE 1 APPROACH SLAB (WITHOUT OVERLAY) - DETAIL 5

APPROACH SLAB SUPPORTED ON ABUTMENT
BACKWALL WITH TOOTH EXPANSION DAM

b. Simply supported girder with tooth expansion joint at abutment backwall

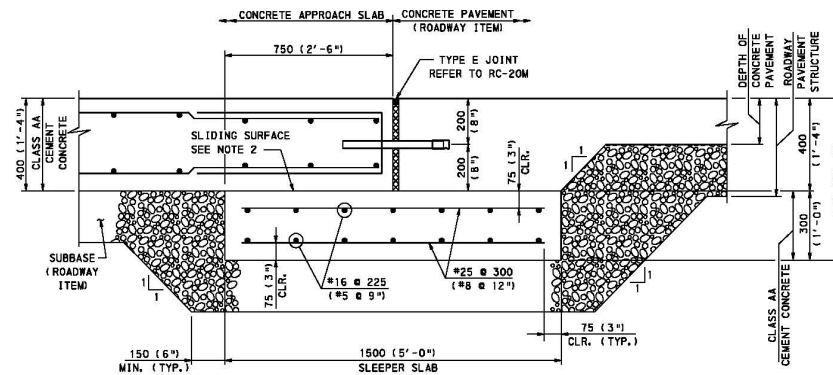
Figure 4.45. PennDOT typical expansion joints for simply supported girders (2009)



TYPE 1 APPROACH SLAB (WITHOUT OVERLAY) - DETAIL 1

END OF APPROACH SLAB ADJACENT TO FLEXIBLE PAVEMENT

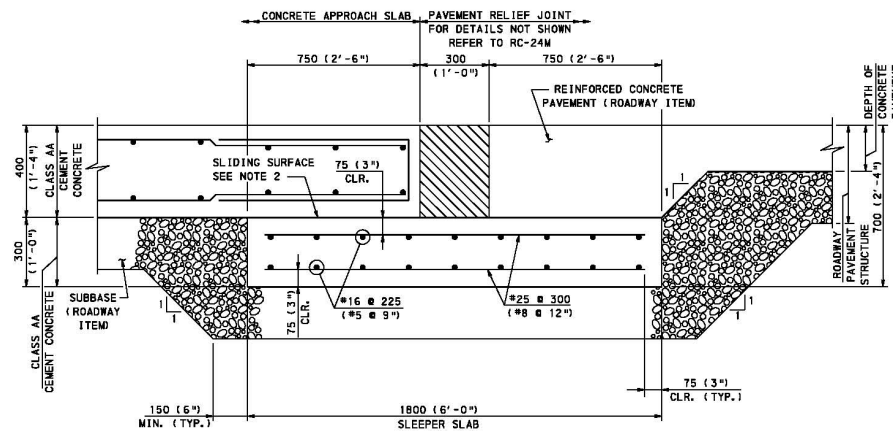
a. Approach slab with flexible pavement



TYPE 1 APPROACH SLAB - DETAIL 2

(SEE NOTE 3)
END OF APPROACH SLAB ADJACENT TO CONCRETE PAVEMENT

b. Approach slab doweled to concrete pavement



TYPE 1 APPROACH SLAB - DETAIL 3

END OF APPROACH SLAB ADJACENT TO PAVEMENT RELIEF JOINT

c. Approach slab with asphalt relief joint between concrete pavements

Figure 4.46. PennDOT typical sleeper slab joints for approach slab (2009)

The Type 2 approach slab is similar to Type 1; however the approach slab is recessed 5 in. to allow for a 5 in. layer of asphalt to be placed over the approach slab. The Type 2 approach can only be used when flexible mainline pavement is utilized. The end of the approach slab has a 5 in. overlay of asphalt on the approach slab.

Approach slabs Type 3 and 4 are used when the expansion joint for the bridge is needed to be moved away from the bridge to the end of the approach slab. Figures 4.47 and 4.48 show the Type 3 and 4 approach slabs, respectively. As seen, simply supported girders and an abutment back wall are still utilized for both types of approach slabs. The approach slab is connected to the bridge deck as shown in Figure 4.49. A diagonal # 6 reinforcing bar at 9 in. on center is used to connect the approach slab to the bridge. A 2 in. minimum gap is provided between the bottom of the approach slab and top of the backwall.

The Type 3 approach slab rests on a sleeper slab and utilizes a strip seal expansion joint. Three different configurations for the approach slab end are detailed in Figure 4.50. In each of the details, 1.5 in. PVC pipe is spaced at 10 ft on center in the sleeper slab at the expansion joint to allow drainage of any trapped water. The detail shown in figure 4.50a can only be used with flexible pavements and has a rotated L-shaped sleeper slab that the flexible pavement butts up against. Figures 4.50b and 4.50c are used with concrete pavement and have an inverted T-shaped sleeper slab. The difference between the two concrete details is the way the concrete pavement is terminated at the stem of the T. Figure 4.50b has a sleeved dowel that connects the pavement and sleeper slab. One inch of expansion joint filler is placed in between the pavement and sleeper slab. The approach end shown in Figure 4.50c uses a 12 in. asphalt relief joint to account for the pavement movement.

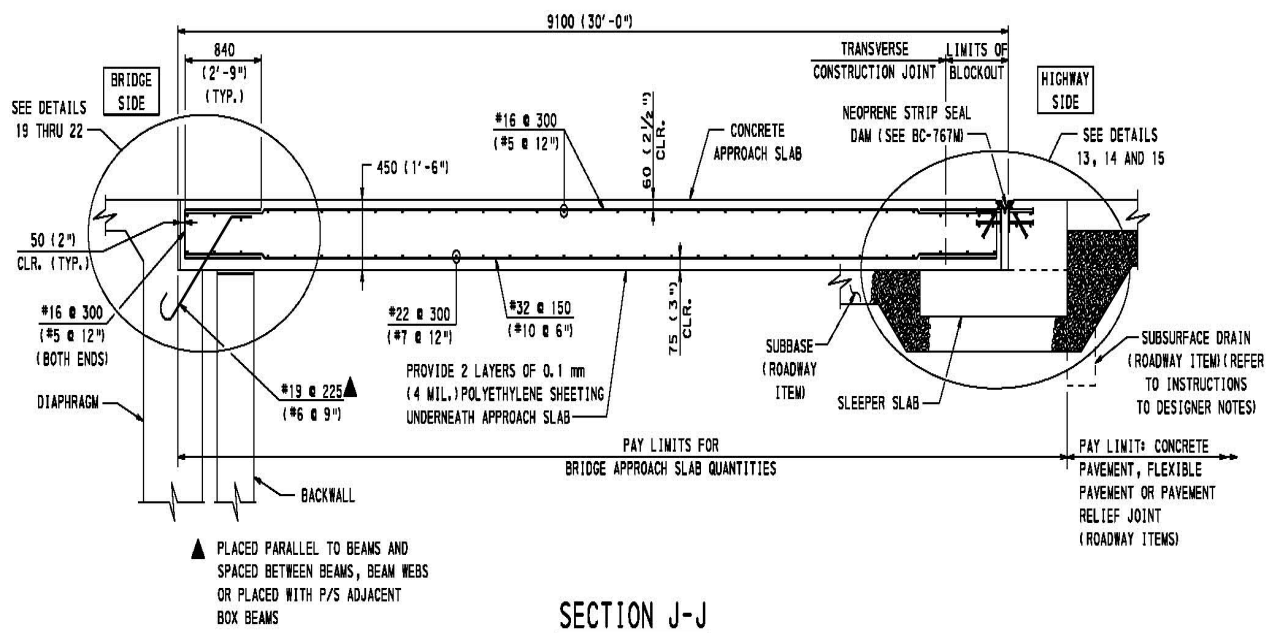
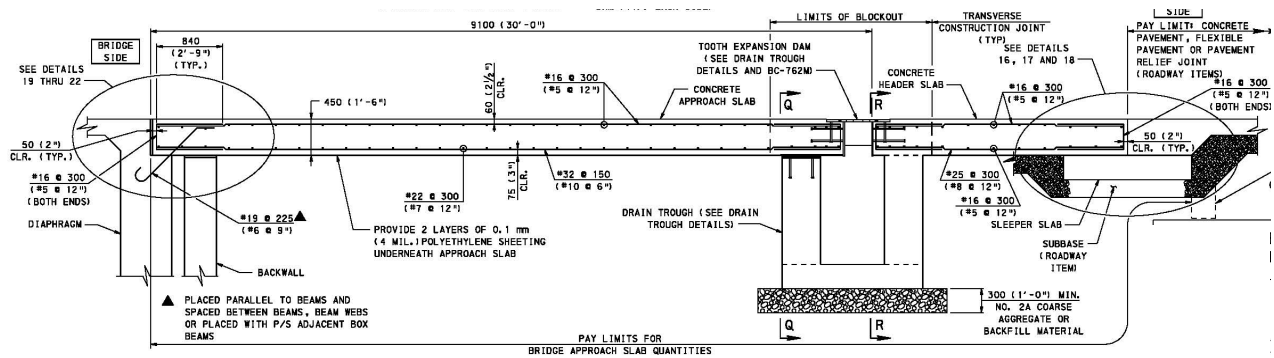
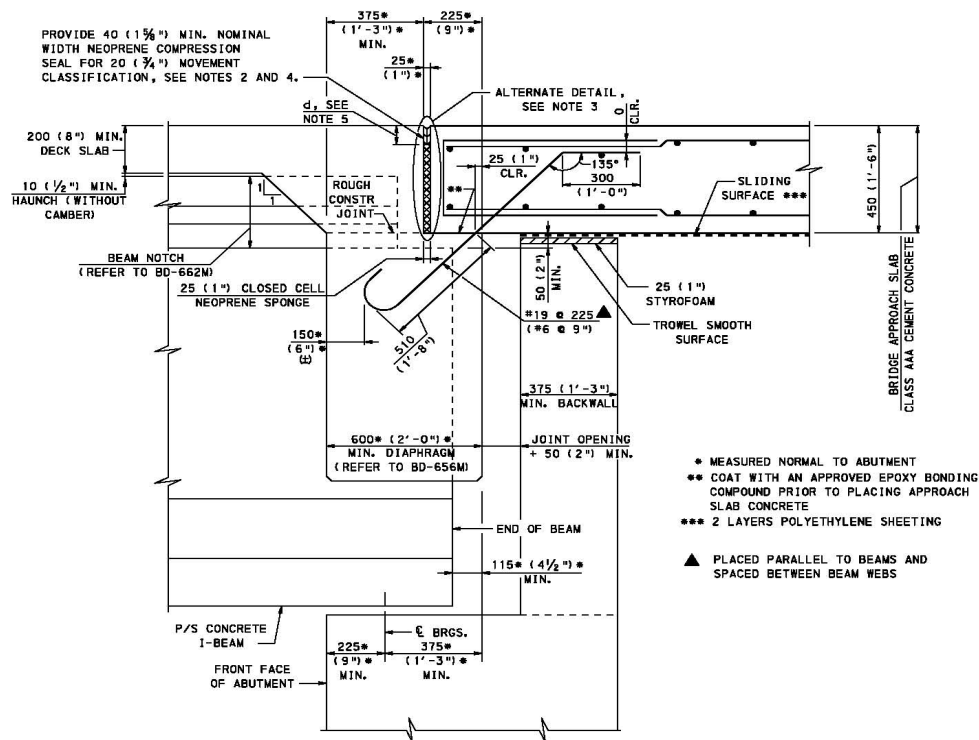


Figure 4.47. PennDOT Type 3 approach slab section for connection at girder with a backwall (2009)



SECTION 0-0

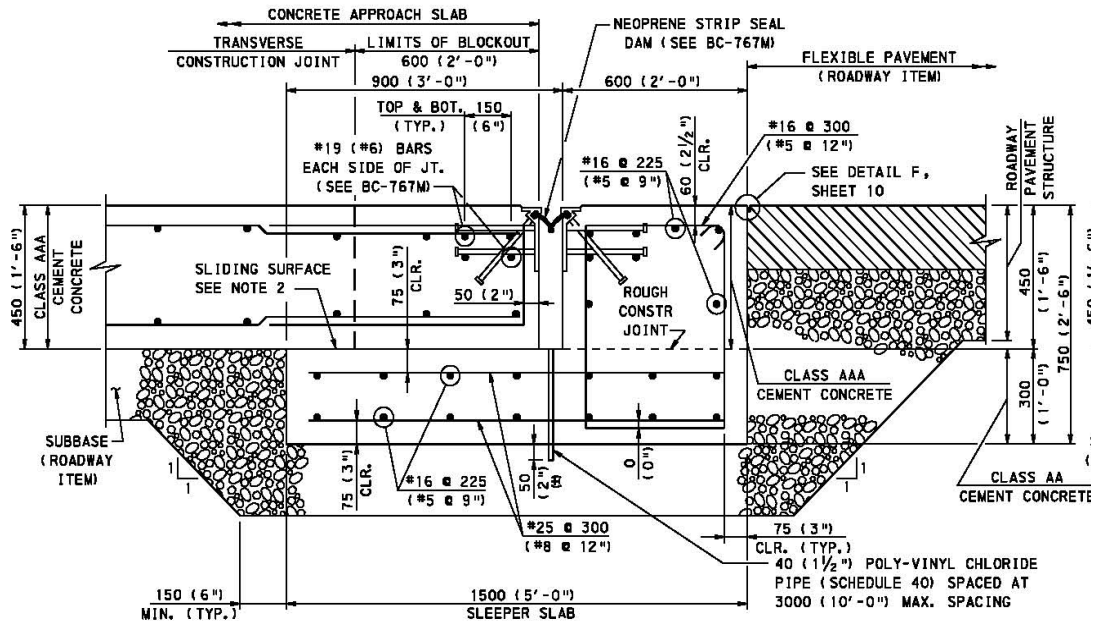
Figure 4.48. PennDOT Type 4 approach slab section for connection at girder with a backwall and a drain trough at end of approach slab (2009)



TYPE 3 AND 4 APPROACH SLAB - DETAIL 21

APPROACH SLAB CONNECTED TO THE SUPERSTRUCTURE
ADJACENT TO P/S CONCRETE I-BEAMS AND
CONCRETE END DIAPHRAGMS WITH BACKWALL
(SEE TABLE A FOR MINIMUM BEAM DEPTHS)

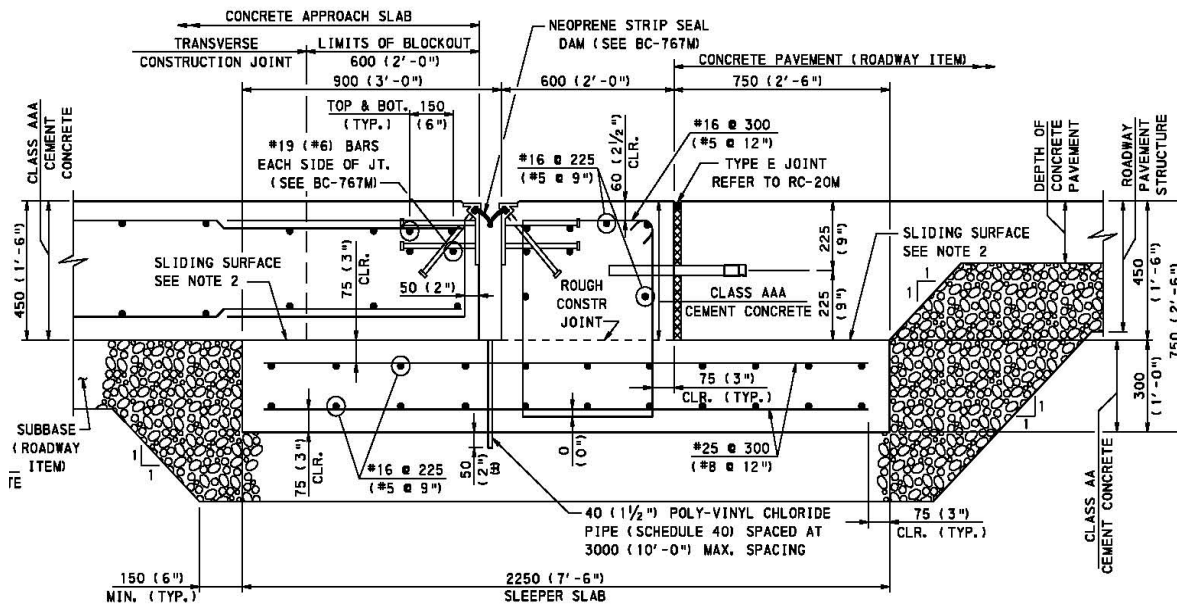
Figure 4.49. PennDOT Approach slab connection to girder when a abutment backwall is present (2009)



TYPE 3 AND TYPE 5 APPROACH SLAB - DETAIL 13

END OF APPROACH SLAB WITH NEOPRENE STRIP SEAL DAM ADJACENT TO FLEXIBLE PAVEMENT

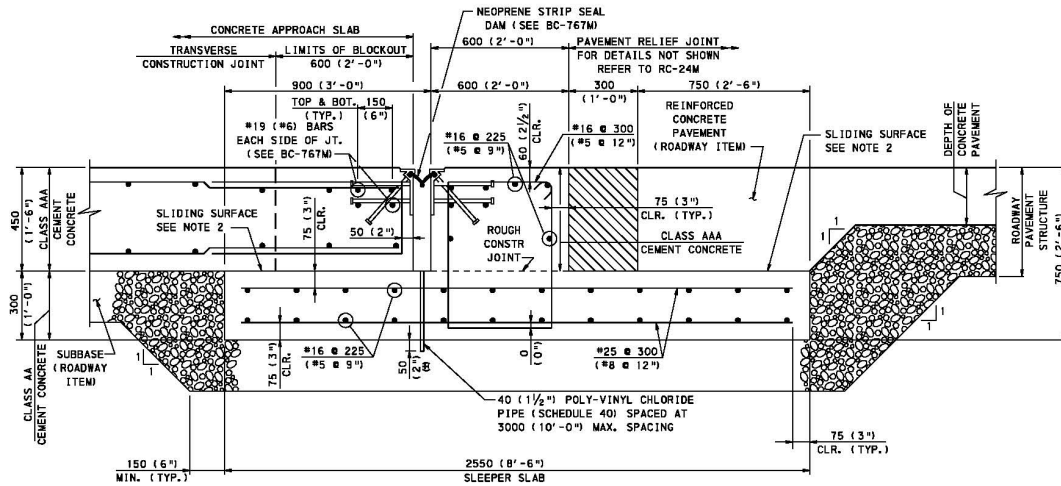
a. Strip seal expansion joint and flexible pavement



TYPE 3 AND TYPE 5 APPROACH SLAB - DETAIL 14

(SEE NOTE 3)
END OF APPROACH SLAB WITH NEOPRENE STRIP SEAL DAM ADJACENT TO CONCRETE PAVEMENT

b. Strip seal expansion joint and doweled concrete pavement



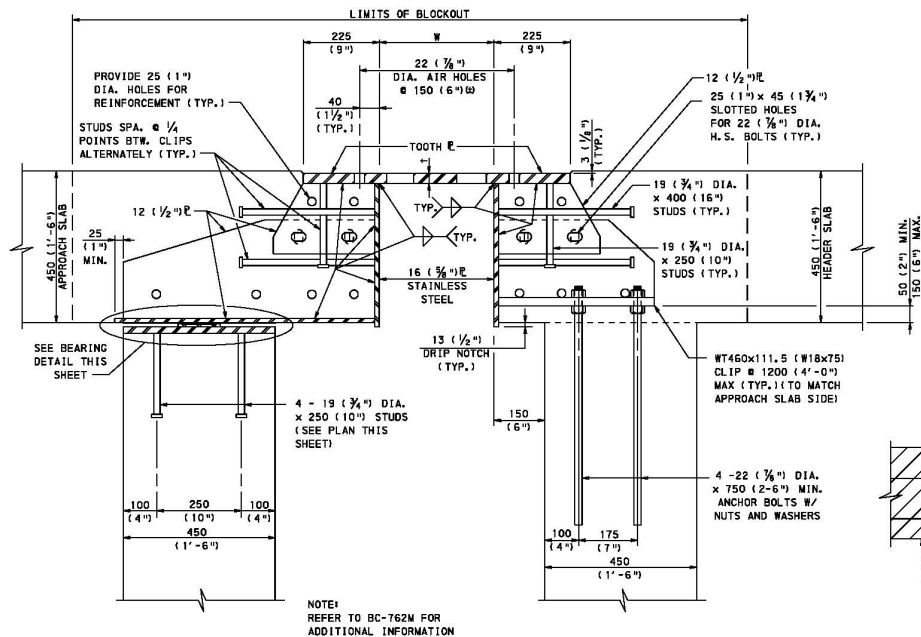
TYPE 3 AND TYPE 5 APPROACH SLAB - DETAIL 15

END OF APPROACH SLAB WITH NEOPRENE STRIP SEAL DAM ADJACENT TO PAVEMENT RELIEF JOINT

c. Strip seal expansion joint with asphalt pressure relief joint against concrete pavement

Figure 4.50. PennDOT typical expansion joint details at end of approach slab (2009)

The Type 4 approach slab uses a tooth expansion dam at the end of the approach slab. To use the tooth expansion joint, a U-shaped secondary footing drain trough is used to support the end of the approach and what is essentially a secondary approach slab. The tooth dam expansion joint is shown in Figure 4.51. The end of the secondary approach slab rests on a sleeper slab and is detailed similar to Figure 4.46 and discussed previously.



TOOTH EXPANSION DAM DETAIL

Figure 4.51. PennDOT tooth expansion joint at approach slab drain trough (2009)

The typical PennDOT approach slab for an integral abutment is shown in Figure 4.52. The detail of the connection to the abutment is similar to the connection shown in Figure 4.49 with #6 reinforcing bars at 9 in. on center. The end of the approach slab rests on a sleeper slab. The approach slab joint at the sleeper slab can be detailed with a strip seal and sleeper slab shape, as shown in Figure 4.50, or PennDOT shows the joint can be detailed similar to the joints detailed in Figure 4.46.

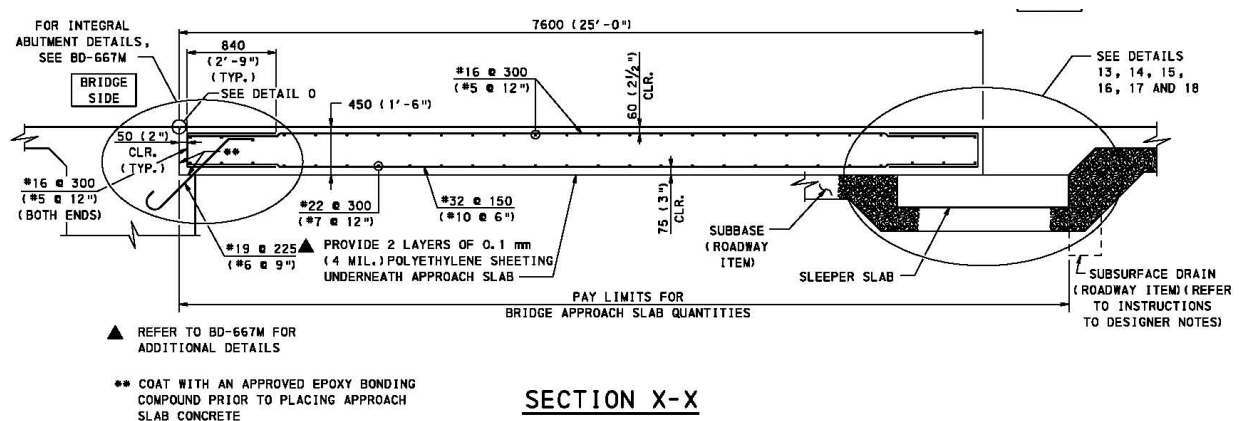


Figure 4.52. PennDOT Type 5 approach slab section used at integral abutment bridges (2009)

4.16.2 Embankment Material

The material used by PennDOT for embankments must be free of organic matter, coal, or other objectionable mater and have a maximum size that can be readily placed in loose lifts of 8 in.

Soil: Material with gradation that has more than 35% passing the No. 200 sieve, a minimum dry mass density of 95 pounds per cubic foot (pcf), a maximum liquid limit of 65, and a plasticity index not less than the liquid limit minus 30.

Granular Material: Includes natural or synthetic mineral aggregates having 35% or less passing the No. 200 sieve.

Shale: Includes rock-like material from natural consolidation of mud, clay, silt, and fine sand; usually thinly laminated, comparatively soft, and easily split.

Rock: Material that cannot be excavated without blasting or using rippers and boulders or stones that cannot be placed in lifts of 8 in. with insufficient soil to fill the voids.

Random Material: Includes concrete, brick, stone, or masonry units from demolition or a combination of four classifications previously described.

The material shall be compacted to not less than 97% of the required dry mass density as determined according to Pennsylvania Test Method (PTM) No. 106 method B. The top 3 ft of the embankment must be compacted to 100% of the required dry mass density. If the material has

more than 20% retained on the 3/4 in. sieve and less than 35% passing the No. 200 sieve or more than 30% retained on the 3/4 in. sieve and cannot be satisfactory compacted, the material shall have its compaction determined based on the non-movement of the material under compaction equipment. The material must be compacted until it no longer ruts under a loaded triaxle.

Embankments can also be constructed of select granular material when specified. Select granular material consists of durable bank or crushed gravel, stone, or slag mixed or blended with suitable filler materials. The material must also be free from organic matter, lumps, or excessive amounts of clay, and have no more than 10% deleterious shale by weight. Select granular material for PennDOT shall have the gradation shown in Table 4.12.

Table 4.12. PennDOT select granular material

Sieve Size	Total Percent Passing
2 inch	100
No. 4	15 to 60
No. 100	0 to 30

4.17. South Dakota

4.17.1. Standards, Specifications, and Details

According to Steve Johnson of SDDOT, the standard practice is almost always to connect the approach slab to the bridge deck on integral abutment bridges (Greimann et al., 2008). This has been the practice for approximately the last 25 years. The connection is made by extending #7 reinforcing bars that are embedded horizontally 2 ft into the bridge deck into the approach slab for 2 ft every 9 in. as shown in Figure 4.53. A mechanical splice is used to make construction easier. After backfilling of the abutment is complete, the horizontal reinforcement is spliced. The connection is used to keep water from flowing into the backfill and to provide a smoother transition while driving, because the “bump” is at least moved to the end of the approach slab. According to Johnson, the connection has performed relatively well over the years. One change was made after transverse cracking was noticed 4 ft to 5 ft from the bridge. It was determined that the reinforcement was “too high” in the slab, so the design was changed to have the connection steel deeper in the slab. The only other problem reported is that the far end of the approach slab sometimes settles. Plastic sheeting is required beneath the approach slab, not to reduce sliding friction, but to create a mud-jack barrier, so that mud is not lost into the voids of the base course, if mud-jacking must be performed. When the abutment is backfilled, drains are installed along the backside of the abutment. The first 3 ft from the abutment is free draining granular material. After that, typical fill (unspecified) is brought up in 8 in. to 12 in. lifts and compacted as best as possible.

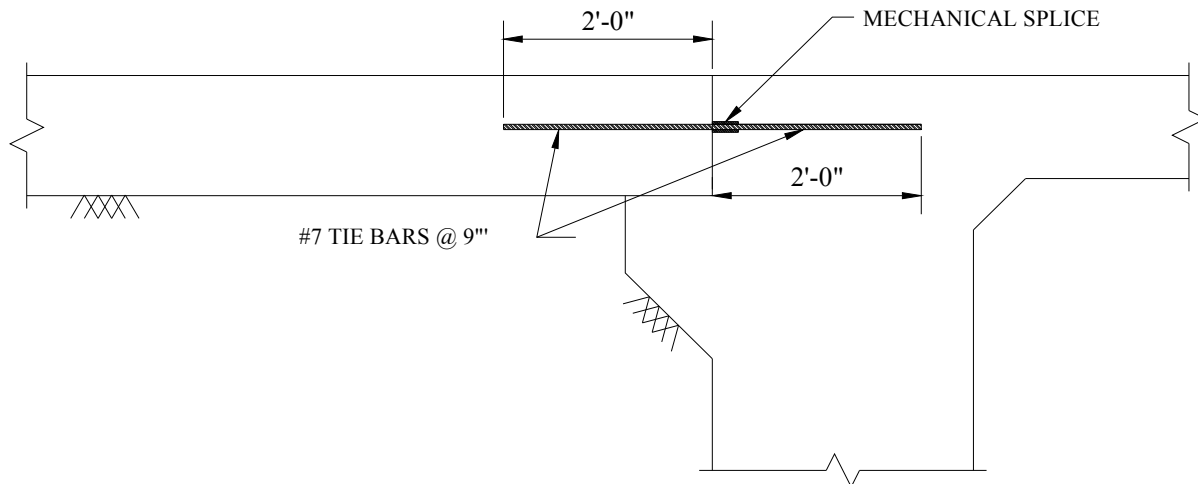


Figure 4.53. Typical South Dakota detail (Greimann et al., 2008)

4.17.2. Current and Past Research

Schaefer and Koch (1992) investigated the void development under bridge approaches in South Dakota, to model the soil behavior next to an integral abutment, and to develop recommendations for the maintenance of existing bridge end backfill systems and future design improvements. In total, 104 bridges were observed for the study. Of these, 90 had integral abutments and 14 had non-integral abutments. From the field study, voids were observed to primarily occur in structures having integral abutments and the void size was seen to generally increase as the length of the bridge increased. The observations also revealed that mud jacking the slab was not an effective solution to stop void formation. Voids were still found under the slab after mud jacking had taken place, the drainage was not corrected in most cases, and in some cases the approach slab was cracked due to water freezing between the mud jack and slab.

4.18. Tennessee

4.18.1 Standards, Specifications, and Details

The Tennessee DOT (TDOT) utilizes both simply supported girder bridges and integral abutment bridges. Figures 4.54 through 4.56 show the simply supported girder abutment. The three details differ by only their expansion control device. A simply supported girder rests on a 3 ft deep pile cap and has a 1.5 ft backwall. The backwall has a paving notch with 1 ft bearing for the approach slab. Figure 4.54 shows the strip seal expansion joint commonly seen in many bridges just behind the backwall. Figure 4.55 shows the bridge deck rigidly attached to the back wall with horizontal #6 bars. Figure 4.56 shows a void where an expansion joint can be placed. The location where the bridge deck attaches to the expansion device is a 1 ft deep by 1.5 ft long block for the expansion device attachment.

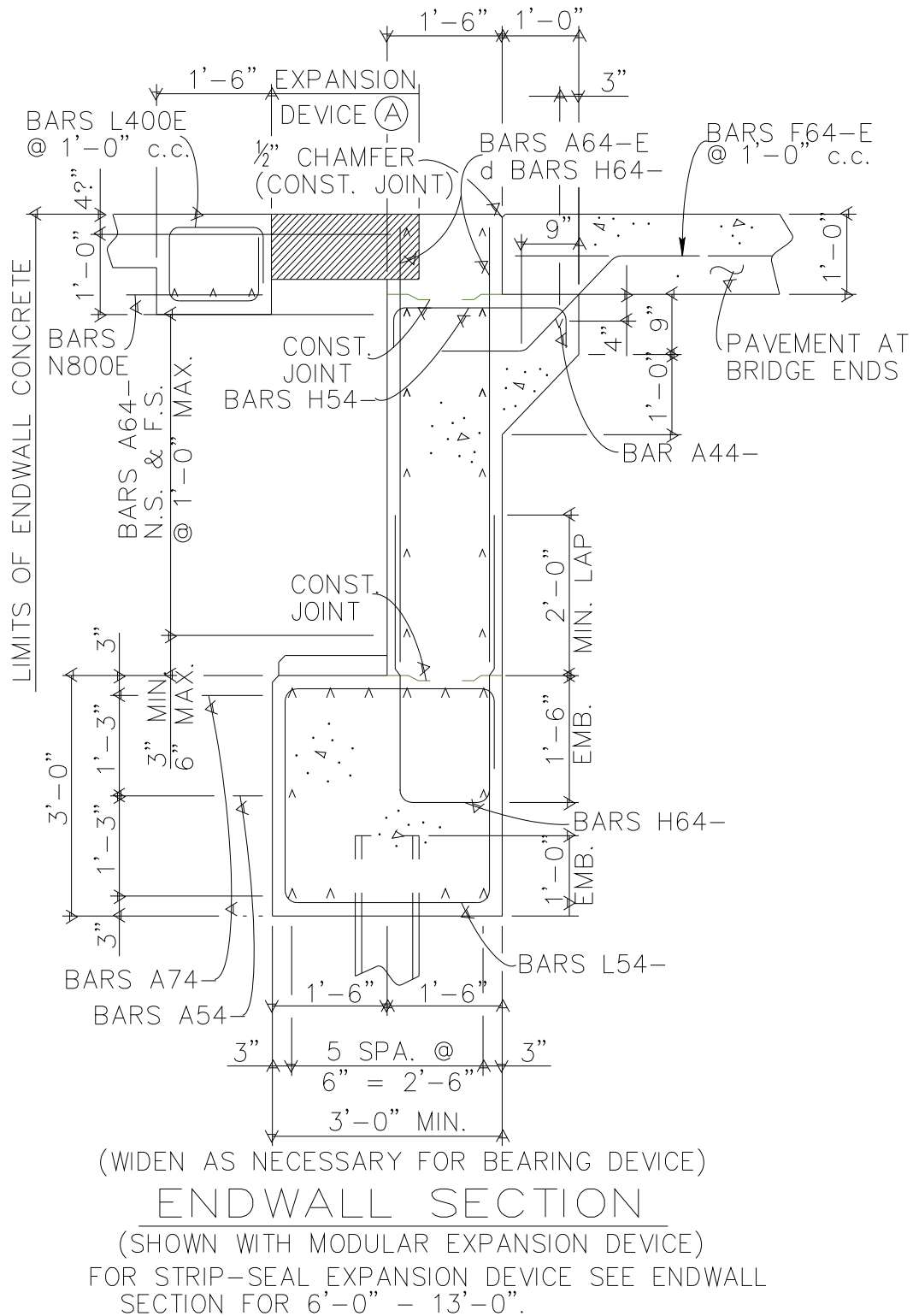


Figure 4.55. TDOT simply supported girder with connected bridge deck (2010)

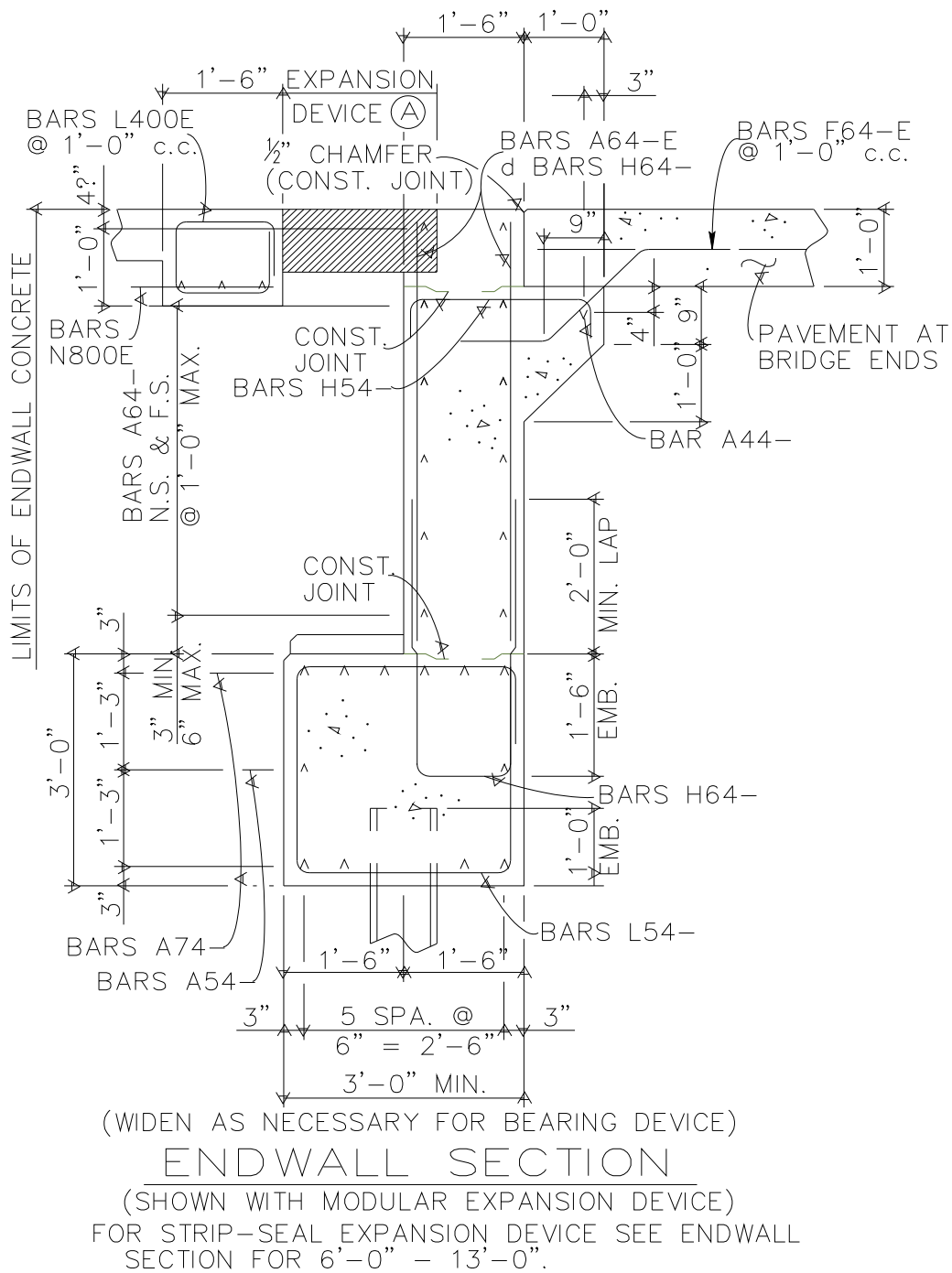


Figure 4.56. TDOT simply supported girder with alternate expansion device (2010)

Figure 4.57 shows the TDOT integral abutment. The integral abutment, similar to other states, utilizes vertical #5 bars on the outside edges of the abutment to “fix” the girders to the pile cap. The deck is attached to the abutment with bent #6 bars.

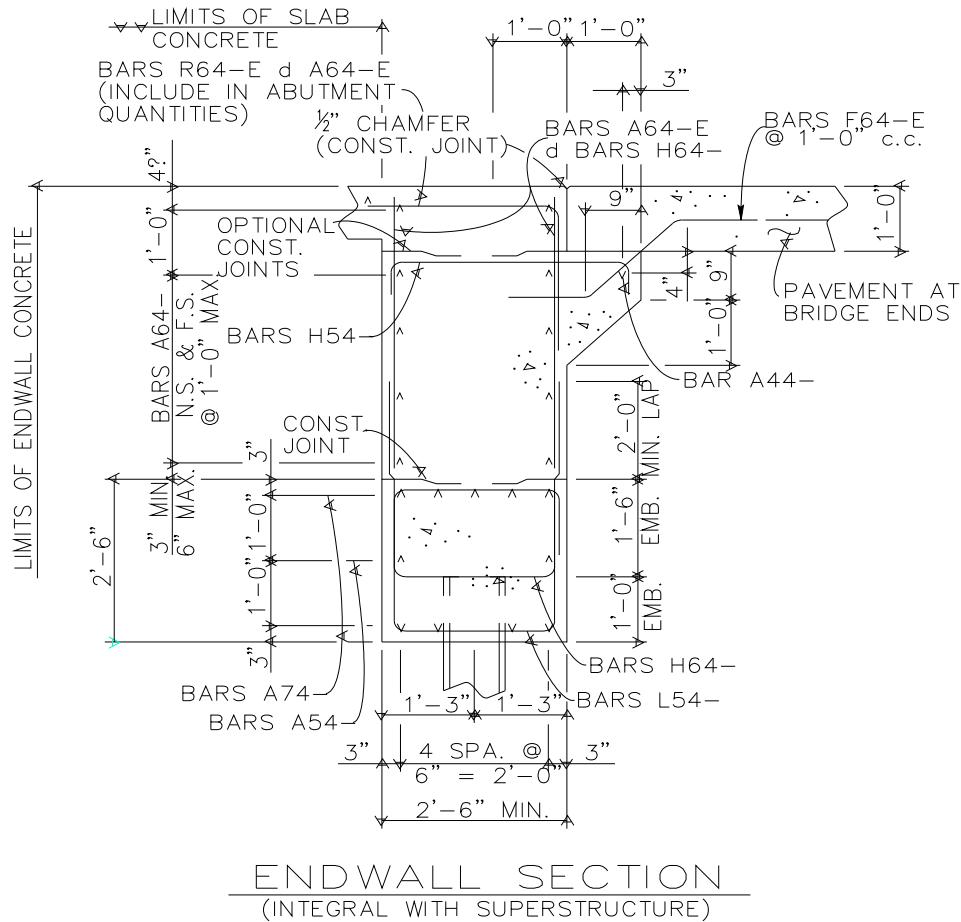


Figure 4.57. TDOT integral abutment (2010)

The approach slab is attached to the paving notch with a #6 bar at 1 ft on center. The connector bar is angled and extends out of the paving notch nose into the approach slab. The typical approach slab, shown in Figure 4.58, is 1 ft thick and 24 ft long. The slab has two layers of longitudinal reinforcing. The bottom layer, #6 bars at 6 in. on center 12 ft long, is located from the bridge out to the center of the approach slab. The second layer of reinforcing, #6 bars at 1 ft on center, extends the length and is at mid-thickness of the slab. The end of the approach slab next to the pavement rests on a sleeper slab. TDOT has four basic configurations of sleeper slabs as shown in Figure 4.59. Figure 4.59a shows the sleeper slab when asphalt pavement is used. The sleeper slab is an inverted-T that's stem is recessed 3 in. for the asphalt to overlay. A 2 in. expansion joint is located between the approach slab and stem of the sleeper slab. The joint is filled with a styrofoam forming strip. The top of the joint has a joint seal system that has an overall longitudinal length of 11.5 in. Figure 4.59b shows the sleeper slab for concrete pavement. The sleeper slab is a 3 ft slab with the concrete approach and pavement rest on the slab. The expansion joint is located between the approach slab and pavement with a similar joint as described for the asphalt slab. Figure 4.59c shows the sleeper slab configuration when no pavement type is designated. Again, the sleeper slab is an inverted-T; however, the stem of the T is not recessed, but at the same level as the pavement. Lastly, Figure 4.59d shows the sleeper slab when asphalt shoulders are used on the sides of the sleeper slab.

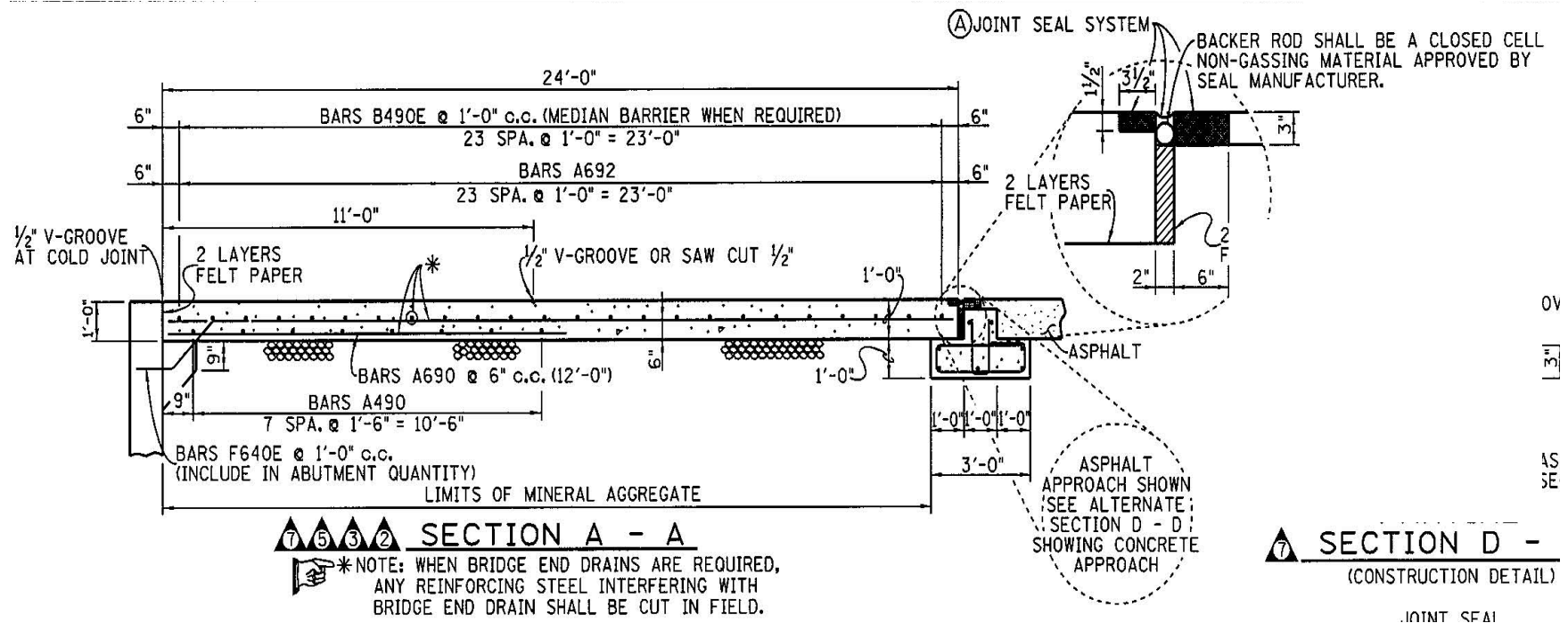
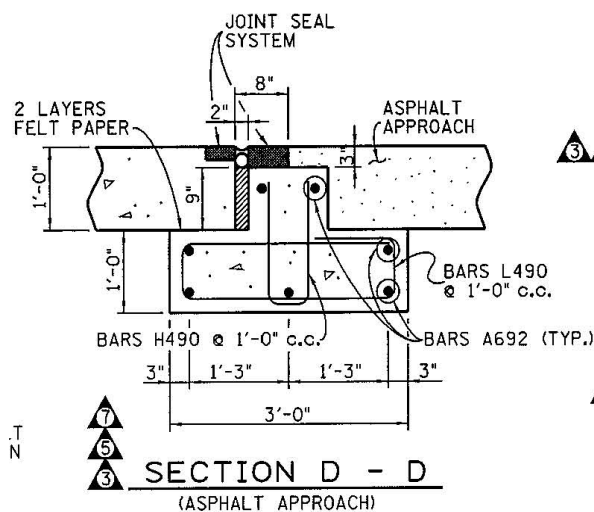
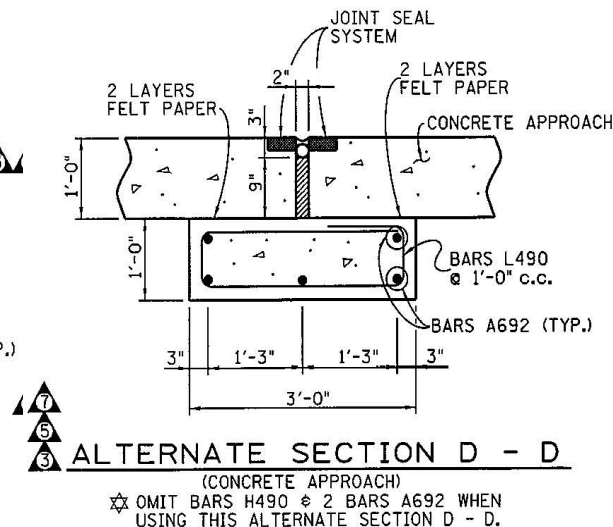


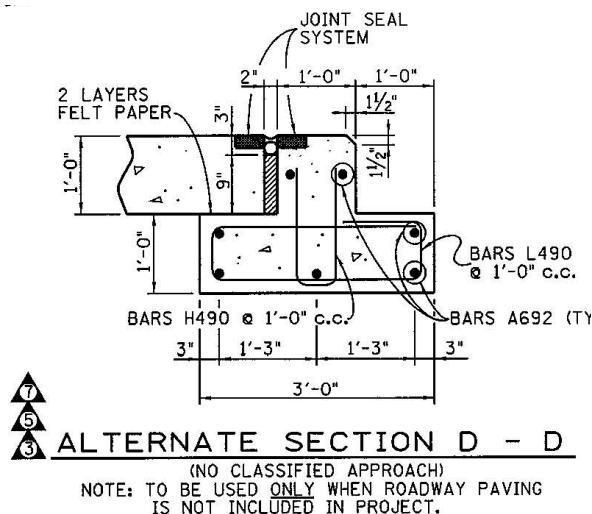
Figure 4.58. TDOT typical approach slab (2010)



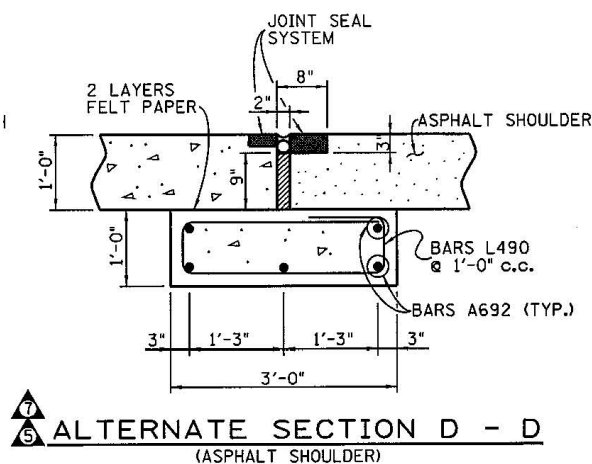
a. sleeper slab for asphalt pavement



b. sleeper slab for concrete pavement



c. sleeper slab for non-classified pavement



d. sleeper slab for asphalt shoulder

Figure 4.59. TDOT various sleeper slab configurations for the approach slab end (2010)

4.18.2. Embankment Material Requirements

The material that can be used for embankments shall be approved by the engineer and shall consist of what TDOT calls road and drainage excavation, channel excavation, and borrow excavation. The three types of materials are described as follows:

Road and drainage excavation: Road and drainage excavation material is unclassified material that includes material that is not classified as borrow or channel excavation.

Channel excavation: Channel excavation is also an unclassified material that has been removed during channel excavation.

Borrow excavation: Borrow excavations can be subcategorized as graded solid rock, non-solid rock material, and select material. The graded solid rock consist of sound non-degradable rock with a maximum size of 3 ft and at least 50% of the rock shall be evenly distributed between 1 ft and 3 ft in size. Thin material is not acceptable and the material shall have no more than 12% weight loss during a sodium sulfate soundness test. The non-solid rock material shall be of AASHTO M145 classification A-6 or better. The select material must meet the requirement set in the contract for the specific project.

When the materials that are used for embankments are soils, the material shall be placed in maximum lifts of 10 in. and compacted to a density of not less than 95% maximum density.

4.19. Virginia

4.19.1. Current and Past Research

Hoppe (1999) reviewed the practices of various state DOTs and how they design and construct approach slabs and compared them with the Virginia DOT (VDOT) practices. Forty-eight state transportation departments were surveyed to obtain feedback on the state of practice with 31 states responding. From the responding DOTs, 81% feel the primary advantage of the approach slab is for improved ride quality. Several states included reduced impact on the backwall and enhanced drainage control as secondary benefits. Disadvantages, however, were listed as increased cost, maintenance problems, and increased construction time.

The actual slab dimensions reported by the responding DOTs varied from 10 to 40 ft long and 8 to 17 in. thick. Most respondents construct full-width approach slabs. The slab connections to integral abutments were generally reported as dowel connections.

Of the responding DOTs, 49% indicated the use of more stringent material specification for bridge approaches as compared to general highway embankments. Most states limited the percentage of fine particles to reduce the material plasticity and provided better drainage. The allowable percentage passing the No. 200 sieve varied from less than 4% to as high as 20%. Most states require compaction effort of 95% Standard Proctor, while four states require 100% Standard Proctor.

Overall, Hoppe determined that the underlying settlement issues with the embankment and foundation soils needs to be resolved whether an approach slab is used or not. The presence of an approach slab has no effect on the magnitude of the differential settlement that will ultimately develop. Hoppe noted that in Germany, approach slabs are seldom used; however, strict material and compaction requirements are enforced in combination with ground improvement techniques.

VDOT has been using approach slabs buried 2 to 4 in. below the final grade for an asphalt overlay without creating feathering problems at the bridge end. By burying the approach slab, it becomes unsuitable to use on integral bridges unless longitudinal movement of the abutment is accounted for. One issue with this design is that vertical pavement shearing occurs around the edge of the approach slab if excessive consolidation occurs with the underlying soils, as shown in Figure 4.60. The shear cracks can then lead to water infiltration and erosion under the slab.



Figure 4.60. Approach slab distress attributable to foundation soil settlement (Route 10 westbound lane over the Appomattox River) (Hoppe, 1999)

Hoppe also proposes burying the approach slab to a depth of approximately 28 in. below the surface and sloping it away from the abutment as shown in Figure 4.61. Hoppe states that Massachusetts has a similar standard design detail, which allows for drainage of subsurface moisture to be deposited away from the backwall.

VDOT has also changed details at the pavement/backwall interface to eliminate the erosion problem that typically occurs there. The detail consists of cantilevering the bridge deck over the abutment approximately 4 in. and placing a drip bead on the underside as shown in Figure 4.62. The detail is only being used for non-approach slab abutments and no comment was made by Hoppe if the detail was working successfully.

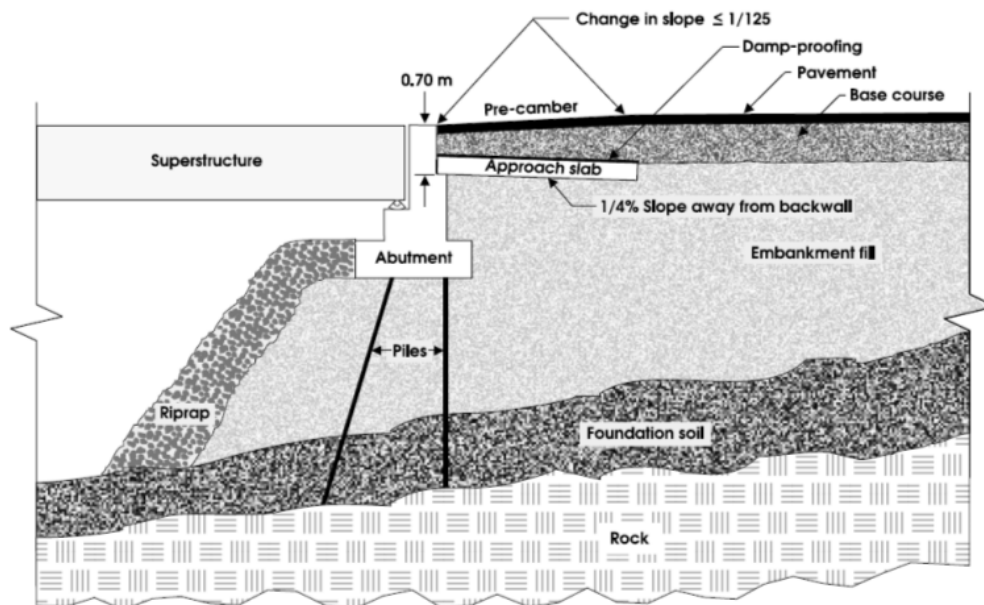


Figure 4.61. VDOT proposed approach slab details for non-integral bridges (Hoppe, 1999)

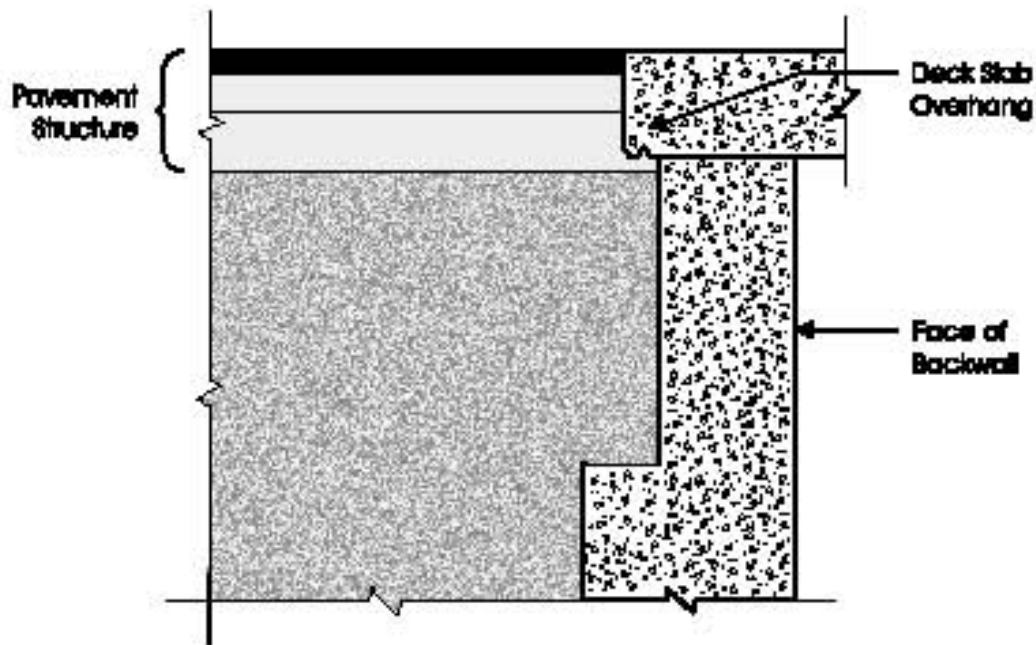


Figure 4.62. Erosion control design detail for bridge without approach slabs (Hoppe, 1999)

4.20. Washington

4.20.1. Standards, Specifications, and Details

The Washington State DOT (WSDOT) (2009) typical approach slab plan and section is shown in Figure 4.63. Figure 4.64 represents typical approach slabs that are rigidly attached to the bridges (e.g., when simply supported girders are used). The attachment is made by #5 reinforcement bars at 12 in. on center bent at 45 degrees into the approach slab. For bridges such as integral abutment bridges with expansion joints at the approach slab, WSDOT places the expansion joint at the bridge abutment as shown in Figure 4.65. The approach slab is connected to the abutment with horizontal anchor rods that have their end anchors surrounded by 1 in. polystyrene as detailed in Figure 4.66.

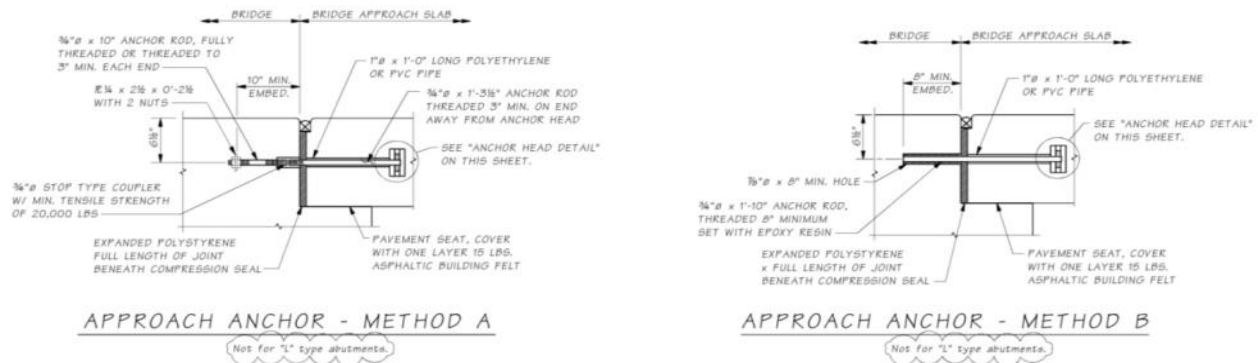


Figure 4.65. WSDOT expansion joint connection at approach slab-bridge joint interface (2009)

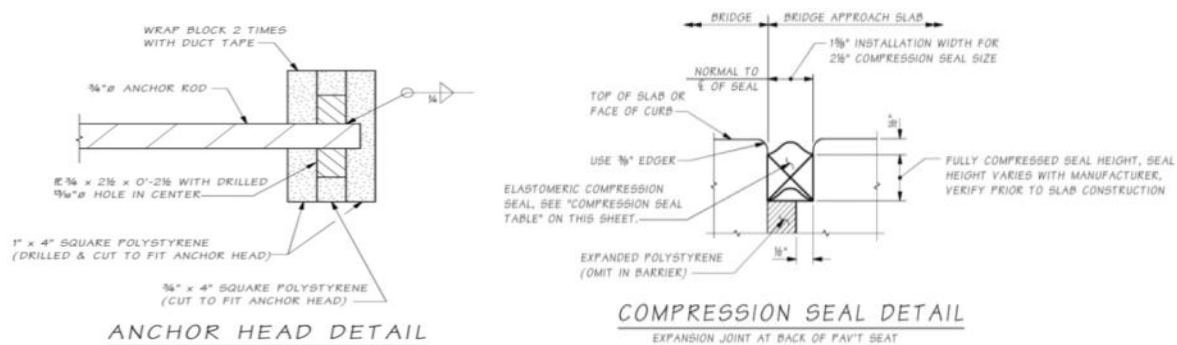
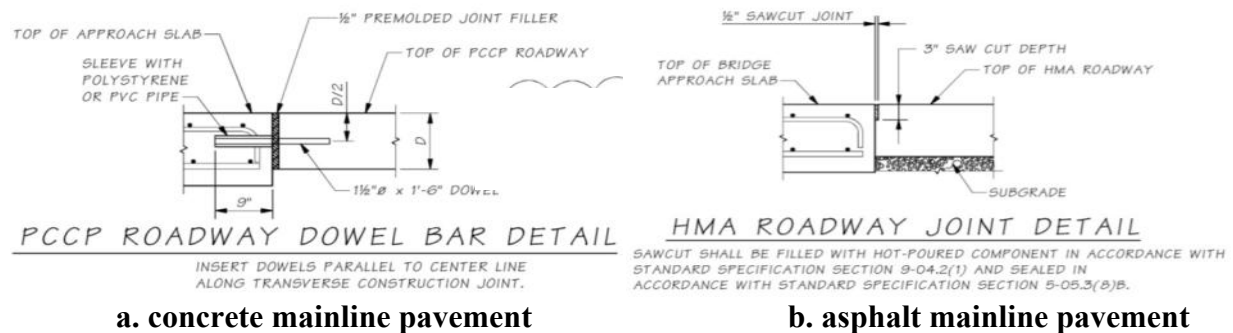


Figure 4.66. WSDOT expansion joint details for anchor head and compression seal (2009)

The approach slab end detail depends on the material used for the pavement. Figure 4.67 shows the two details. Figure 4.67a shows the approach slab end used with concrete pavement. The approach slab is attached to the pavement with a sleeved dowel bar. It includes a 0.5 in. pre-molded joint filler at the interface between the approach slab and pavement. The other approach slab detail, shown in Figure 4.67b, is used for asphalt pavement. The asphalt pavement butts directly against the end of the approach slab. A 0.5 in. wide saw cut 3 in. deep is located at the interface of the approach slab and asphalt pavement. WSDOT has a field replacement paving notch detail that is shown in Figure 4.68. The replacement involves anchoring a steel WT 12 x 47 to the remnants of the existing paving notch.



a. concrete mainline pavement

b. asphalt mainline pavement

Figure 4.67. WSDOT approach slab to mainline pavement joint typical detail (2009)

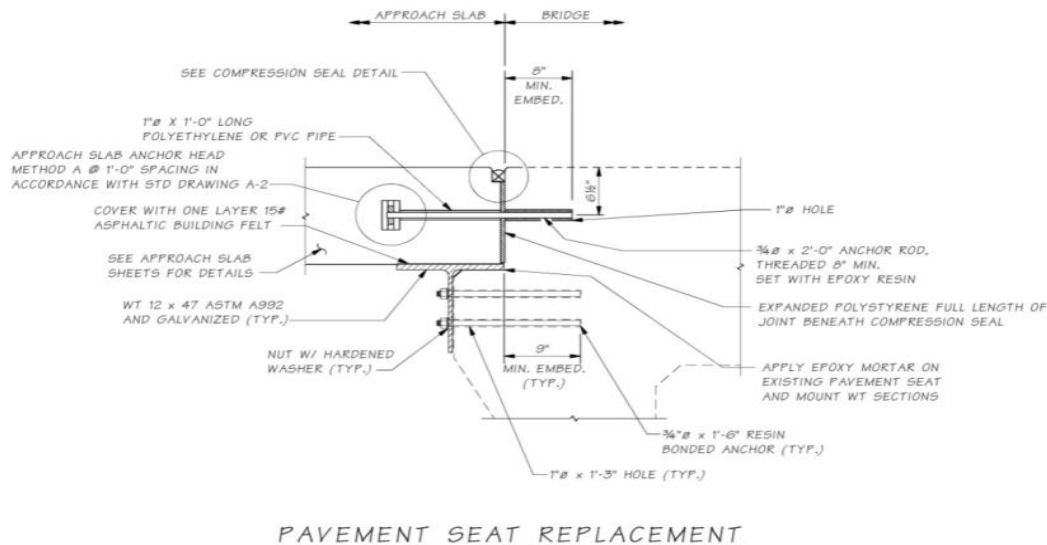


Figure 4.68. WSDOT paving notch field replacement detail (2009)

4.20.2. Embankment Material

WSDOT classifies embankment material as either rock embankment or earth embankment. The rock embankment has material in all or any part of the embankment containing 25% or more of gravel or stone that is 4 in. or greater in diameter. The earth embankment is any other material that is not used in a rock embankment. Bridge approach embankments shall be compacted to at least 95% of the maximum density.

4.21. Wisconsin

4.21.1. Current and Past Research

Helwany et al. (2007) conducted four full-scale bridge case studies comparing Wisconsin DOT (WisDOT) structural backfill with geosynthetic-reinforced backfill or flowable backfill. Two of the bridges had dense sand as the foundation soil, while the other two had hard silty clay underlying silt or loose sand. Each bridge had structural backfill placed in accordance to WisDOT specifications behind one of the abutments, while the other abutment used one of the alternate backfills for each type of soil. The bridges were monitored for up to seven years. Inclinoimeters were used to measure horizontal movement and elevations were taken of the roadway to determine vertical movement.

Based on the field testing and observations, Helwany et al. concluded that the approach fills on granular foundation soils have minimal movement compared to the approaches on cohesive fills. The two alternate backfills performed the same or worse than the structural backfill, when placed on granular foundation soils. However, when the alternative backfills were placed on cohesive foundation soils, they outperformed (smaller movements) the structural back fill. The cost of flowable fill was found to be greater than geosynthetic-reinforced fill for small quantity jobs.

5. IN-SERVICE BRIDGE TESTING AND PERFORMANCE

In July 2009 the ISU BEC field tested nine in-service bridges to better understand the geometric conditions and in situ performance under highway loads. The bridges were located in different regions of Ohio and were selected based on design, fill heights, ride history, and whether they were currently a “good” or a “problematic” bridge. Figure 5.1 shows the location of the bridges tested.

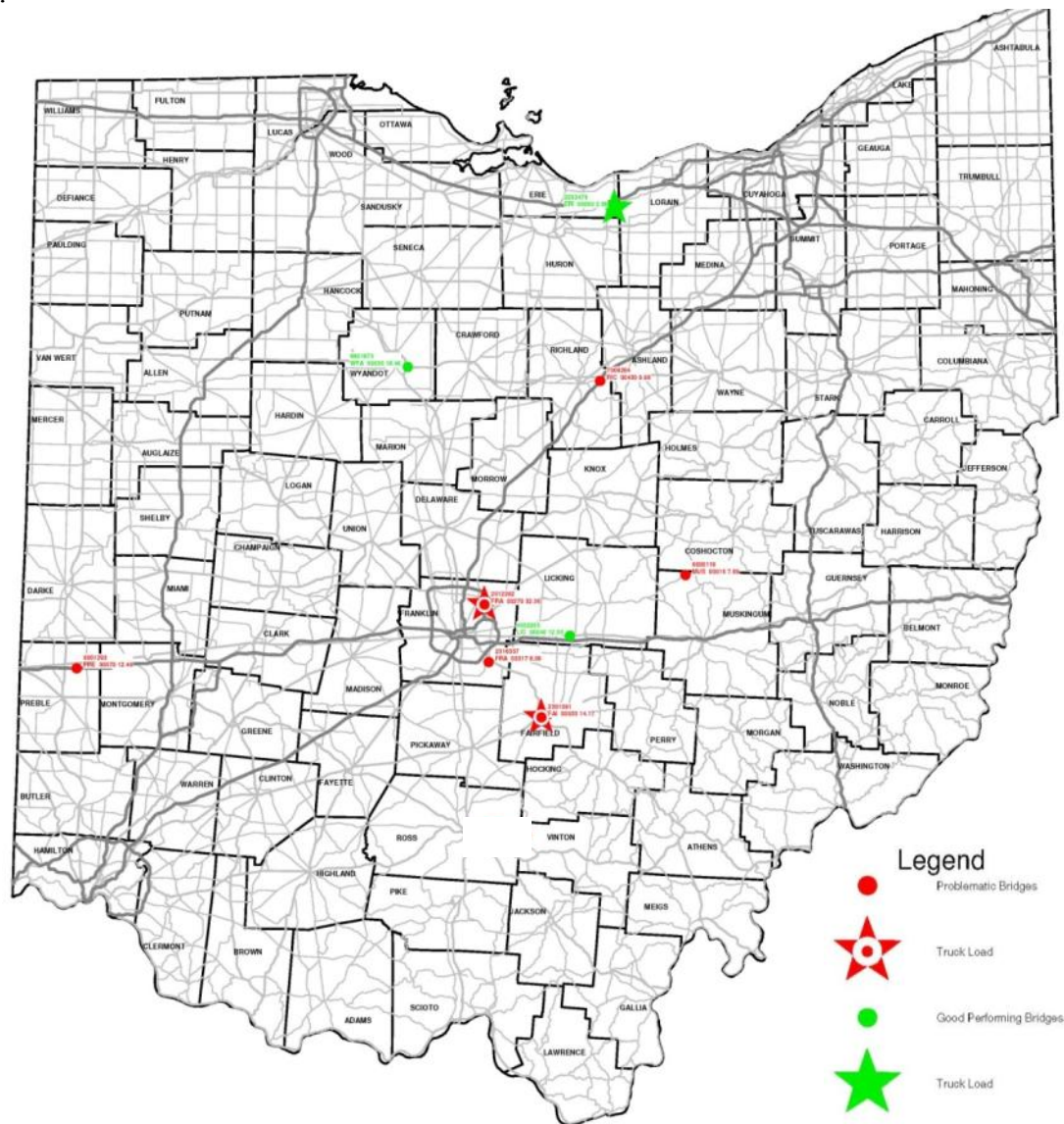


Figure 5.1. Location of in-service bridges tested

5.1. Geometric Bridge Testing and Support System Evaluation

The global geometry evaluation consisted of measuring the geometry of the bridge surface with a laser-based survey system. The survey system, shown in Figure 5.2, consisted of a Trimble SPS930 laser-guided total station and a 360 degree prism mounted on a mono-wheeled cart. The

mounted prism was pushed across the bridge at a slow speed, while the total station automatic followed and recorded distance, elevation, and azimuth information. With this information, data on the overall vertical geometry could be collected. Tests were conducted in both vehicle wheel lines of the driving lane.



a. Trimble SPS930 laser guided total station



b. 360 degree prism and mono-wheel cart

Figure 5.2. Bridge global geometric evaluation system

In addition to global geometry and profile data, International Roughness Index (IRI) and Falling Weight Deflectometer (FWD) data information was obtained by ODOT for each bridge. The profile and IRI data were collected with an inertial road profiler mounted on a moving vehicle. The vehicle was driven across the approach slabs and bridge while data were being collected. Both wheel lines tested during the global geometry test also had their profile and IRI evaluated to study the conditions at a more local level. Herein, IRI data were analyzed on a continuous basis with a sliding base length of 25 ft to allow localized roughness evaluation.

The FWD testing was conducted on both approaches, or departure slabs, in some cases, for each bridge. The FWD, shown in Figure 5.3, was tested on each side of the bridge-approach slab joint, then every 5 ft longitudinally on the approach slab until reaching the approach-pavement joint. The FWD was then tested on each side of the approach-pavement joint. The pavement was then tested every 5 ft up to 25 ft away from the joint. The FWD tests were only conducted down the center line of the driving lane.



Figure 5.3. ODOT falling weight deflectometer (FWD) used for approach testing

5.2. Live Load Testing

Live load testing of three of the nine bridges consisted of monitoring the strain in the approach slab and girders, displacement and rotation of the abutment, and vertical joint movement of the approach slab. A fully loaded three-axle dump truck (approximately 48,000 lbs) was used to load each of the bridges. The truck was driven in two load cases. The one load case placed the truck's passenger tire 2 ft away from the edge of the bridge barrier rail curb. The other load case located the truck in the center of the driving lane, mimicking the location of normal traffic. In the interest of brevity, only data from the normal traffic position test is presented here. (Similar behaviors were found for both truck positions.)

For simplicity, instrumentation was concentrated at one end of the bridge (e.g., only one slab, only one span, etc.). The approach slabs were installed with four Bridge Diagnostics, Inc. (BDI) strain transducers both located in a two-by-two pattern. The strain transducers were placed at the third point of the slab in longitudinal and transverse directions. Figure 5.4 shows the typical location of the approach strain gauges and the orange steel protective caps over the gauges. The approach slab joints were instrumented with displacement transducers to monitor vertical differential movements at the joints. Two transducers were placed at the bridge-approach joint and two were placed at the approach-pavement joint. A typical transducer setup is shown in Figure 5.5. Typically, the transducers were transversely located at the edge of the lane and near the curb of the bridge.



Figure 5.4. Typical strain and displacement transducers approach slab layout



Figure 5.5. Displacement transducer at approach-pavement joint

Again, only one abutment for each bridge was instrumented. The abutment of the bridge was monitored for both horizontal translation and rotation. Two displacement transducers, one near each of the outside girders, were used to monitor horizontal translation of the abutment and in the same vertical plane a tilt meter was placed at mid-height of the backwall. Figure 5.6 shows a typical abutment monitoring setup.



Figure 5.6. Abutment horizontal translation and rotation monitoring

Lastly, the girders located under the driving lane were instrumented at the midspan and near the abutment of the span closest to the approach slab and abutment being monitored. The girder strains were monitored with BDI strain transducers. The typical gauge attachment is shown in Figure 5.7.

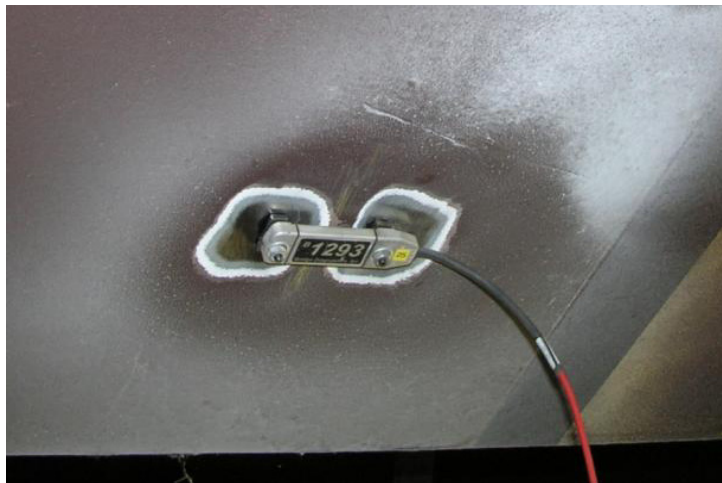


Figure 5.7. Girder strain monitoring

5.3. FAI 33-14.17

5.3.1. FAI 33-14.17 Bridge Description and Evaluation

The FAI 33-14.17 northbound bridge was investigated during the study. The bridge is a two-lane three span bridge with a total length of approximately 183 ft and a skew of 23 degrees. The bridge was built in 2001. Figure 5.8 shows the profile of the bridge. The middle span has a length of 78 ft and spans over Crumley Road. The end spans are 52 ft long. The bridge consists of six 3 ft 9 in. deep pre-stressed concrete I-beams that are composite with the deck. The girders are semi-integrally attached to the abutment. The foundation consists of battered steel piles with concrete caps. The bridge piers are also located on steel piles. The bridge has 30 ft long approach slabs that are attached to the abutment with an angled #8 bar and rest on a 6 in. long paving notch.



Figure 5.8. FAI 33-14.17 bridge

5.3.2. FAI 33-14.17 Visual Bridge Evaluation

Observations during the field investigation found a crack in the north approach slab of the bridge. The crack extended the full longitudinal length of the slab and was located primarily in the driving lane of the bridge. Figure 5.9 shows the approach slab crack. In addition to the approach slab crack, an asphalt wedge was observed on both ends of the bridge extending approximately 70 ft from the ends of the approach slab. The asphalt wedge had oil staining on the surface at both the entrance and exit to the bridge. Generally, the presence of oil staining

indicates a bump condition just prior to the stain. Oil staining can be caused by the bump exciting the vehicle vertically which in turn causes increased inertial forces on leaking engine oil droplets that then fall to the pavement on the downward portion of the excitation. Figure 5.10 shows the asphalt wedge and oil staining at the exit of the bridge.

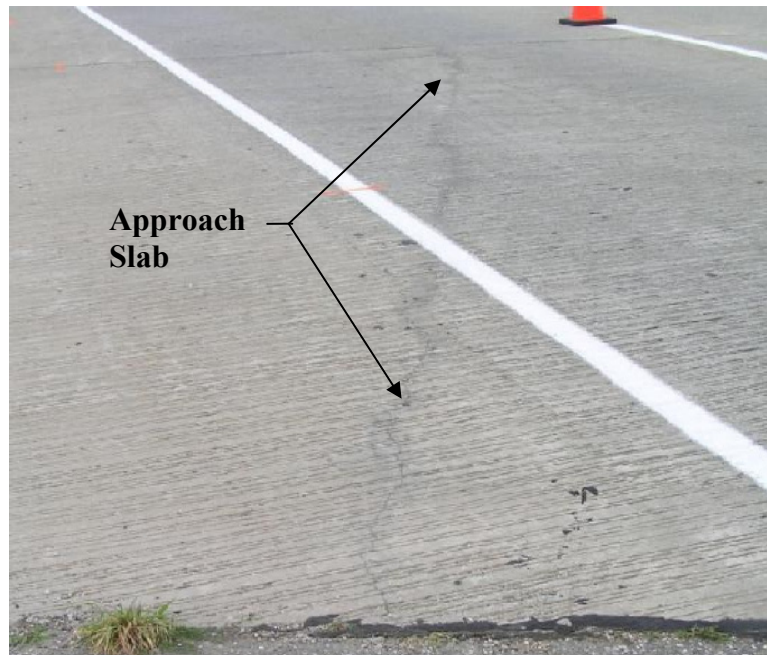


Figure 5.9. FAI 33-14.17 approach slab cracking

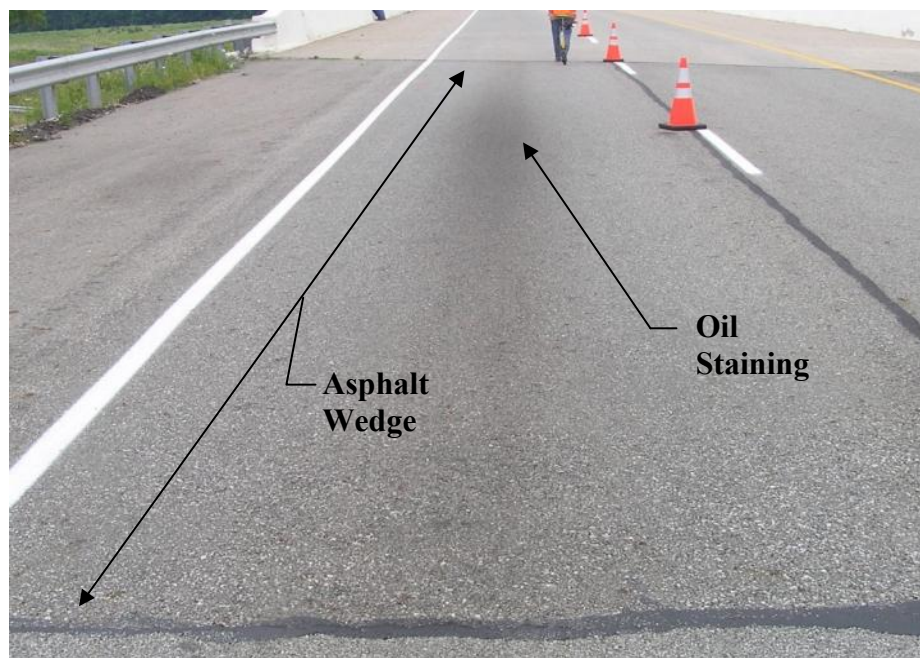


Figure 5.10. FAI 33-14.17 asphalt wedge and oil staining located at exit of bridge

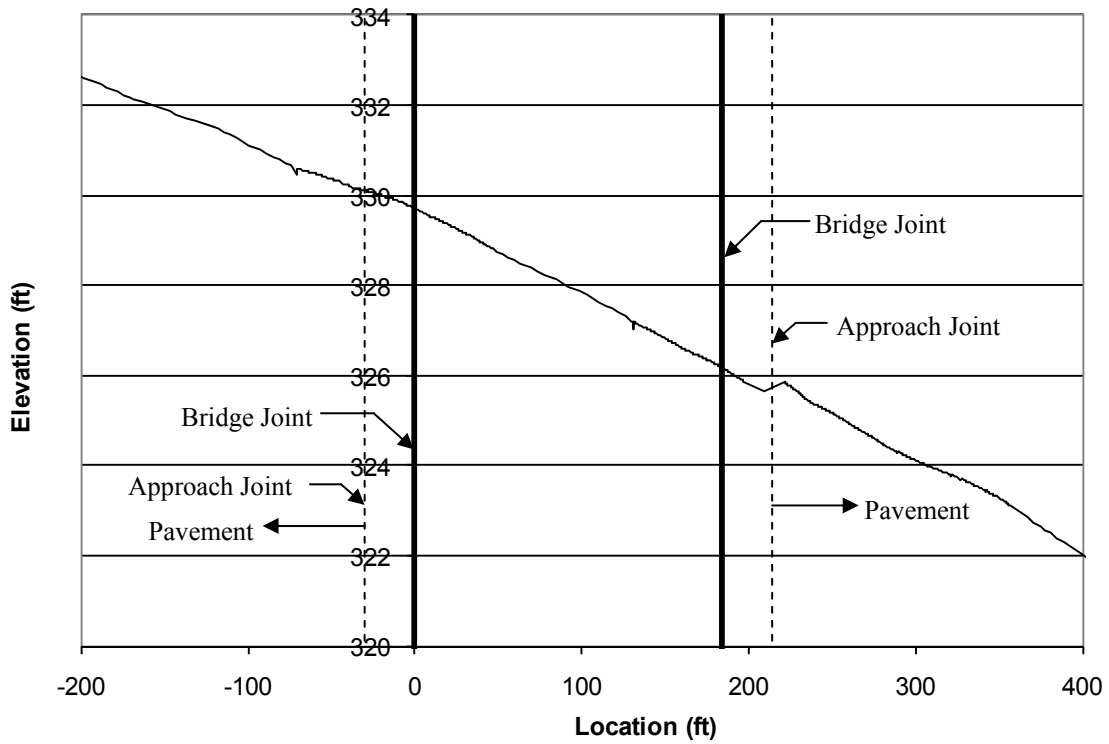
5.3.3. FAI 33-14.17 Geometric Bridge Testing and Support System Evaluation

Four types of “geometric” data for the FAI 33-14.17 northbound bridge were collected and evaluated. Specifically, the global geometry was evaluated using a laser-based survey system, the local roadway condition was evaluated using IRI data collected by ODOT, the support system stiffness was evaluated from FWD test results, and the support system was further evaluated through a review of the overall support system depth.

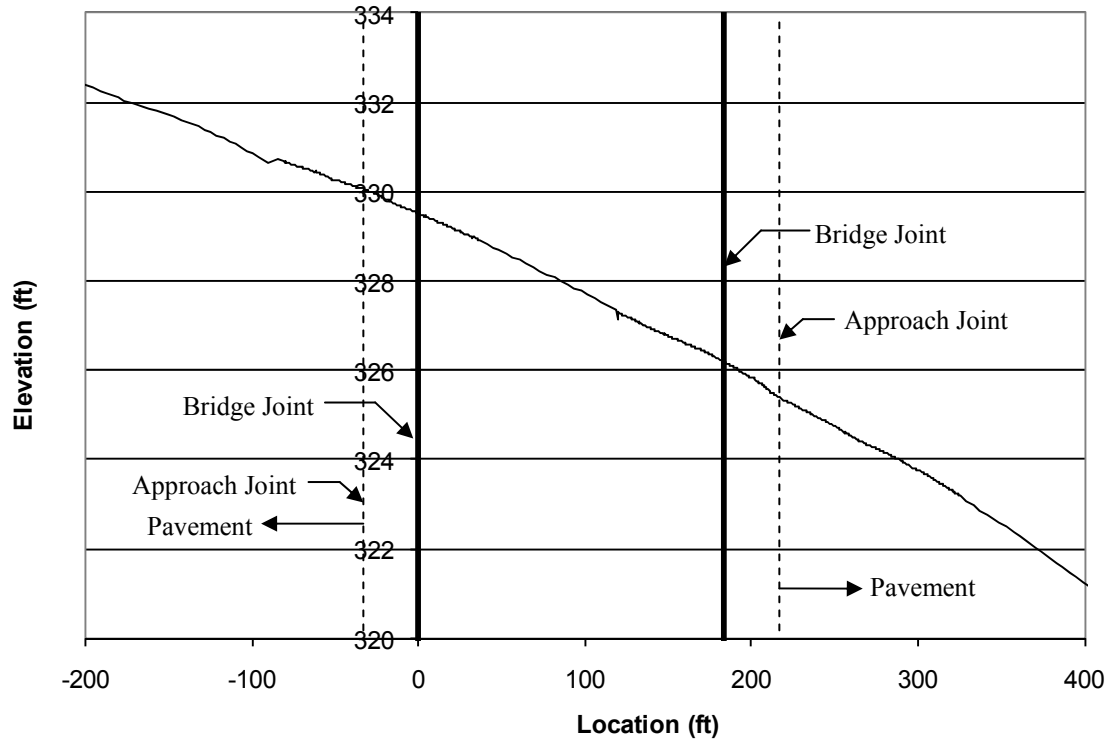
Figures 5.11a and 5.11b show the results of the laser-based survey. As can be seen, there is an overall slope of the bridge with three global geometric discontinuities observed. The first global discontinuity occurs at the entry end of the bridge and can be observed in both the right and left wheel lines. The location of the discontinuity relative to the bridge indicates that the discontinuity may be at the end of the approach slab system. The second discontinuity occurs on the bridge and can, again, be seen in both wheel lines. This discontinuity may be occurring at an on-bridge joint. The third and final discontinuity occurs only in the right wheel line after the exit end of the bridge. This discontinuity is a noticeable “ramp” occurring in the region of the approach slab.

The raw profiler data are shown in Figures 5.11c and 5.11d. From these data it can be observed that both left and right wheel lines tend to track well together before/after the approach slabs. A general agreement in measurements can also be seen on the bridge itself. However, the approach slab regions show differing profiler data in the approach slab regions. The difference is most marked on the exit end of the bridge—possibly related to the right wheel line global geometric discontinuity noted above. The IRI results continue to corroborate the findings noted above. Further, the IRI results in the approach slab regions show IRI values approaching 600 in./mile. It should be noted that here, and in subsequent similar discussions, that maximum, instantaneous values will be cited. It is important to note that these represent the worst measurement for the site and not the site average. These instantaneous values speak to localized roughness, which usually occurs at the bridge-pavement interface.

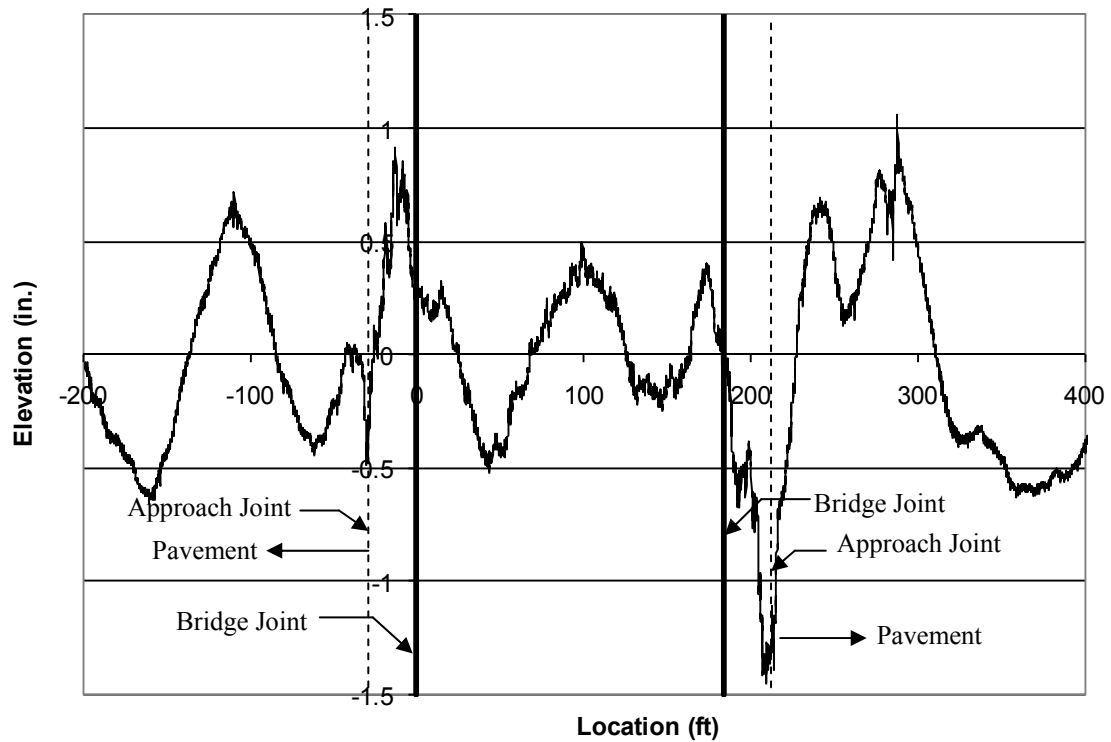
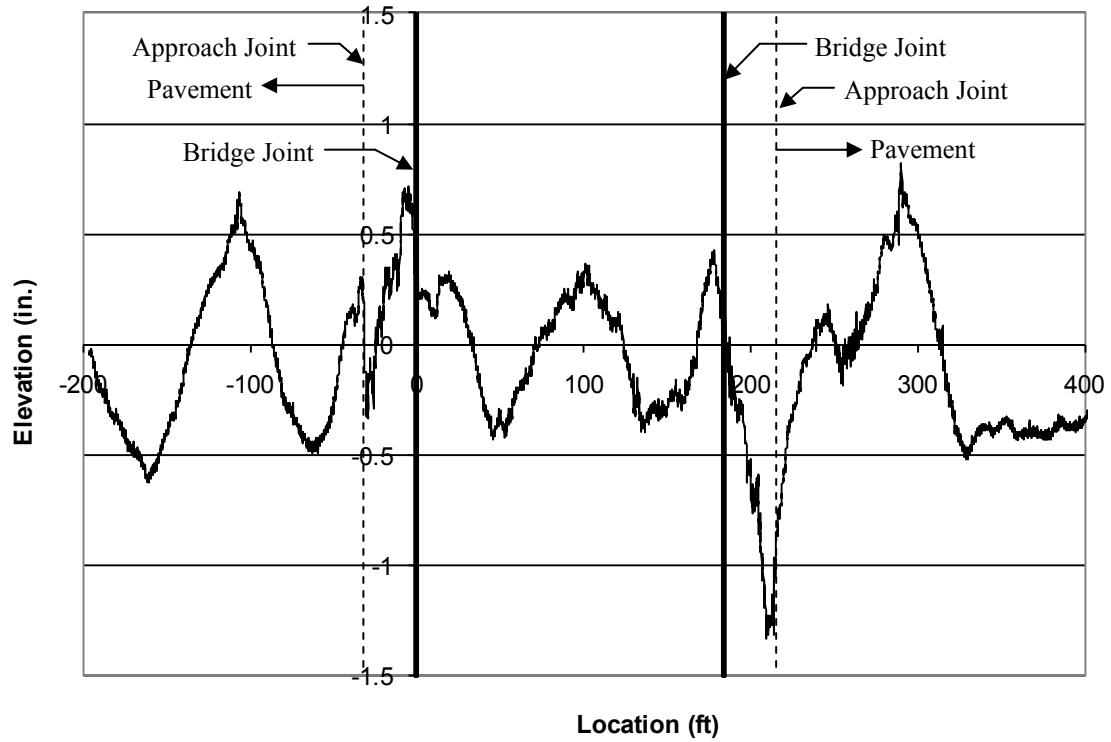
Observations from the FWD test results show a very stiff support system close to the bridge ends and that this support system seems to soften moving away from the bridge ends. The approximate fill heights, however, indicate only a minimal difference in the fill depths moving away from the bridge ends. Collectively, this would indicate that the stiffness differences are derived from differences in construction (e.g., soil compaction levels).

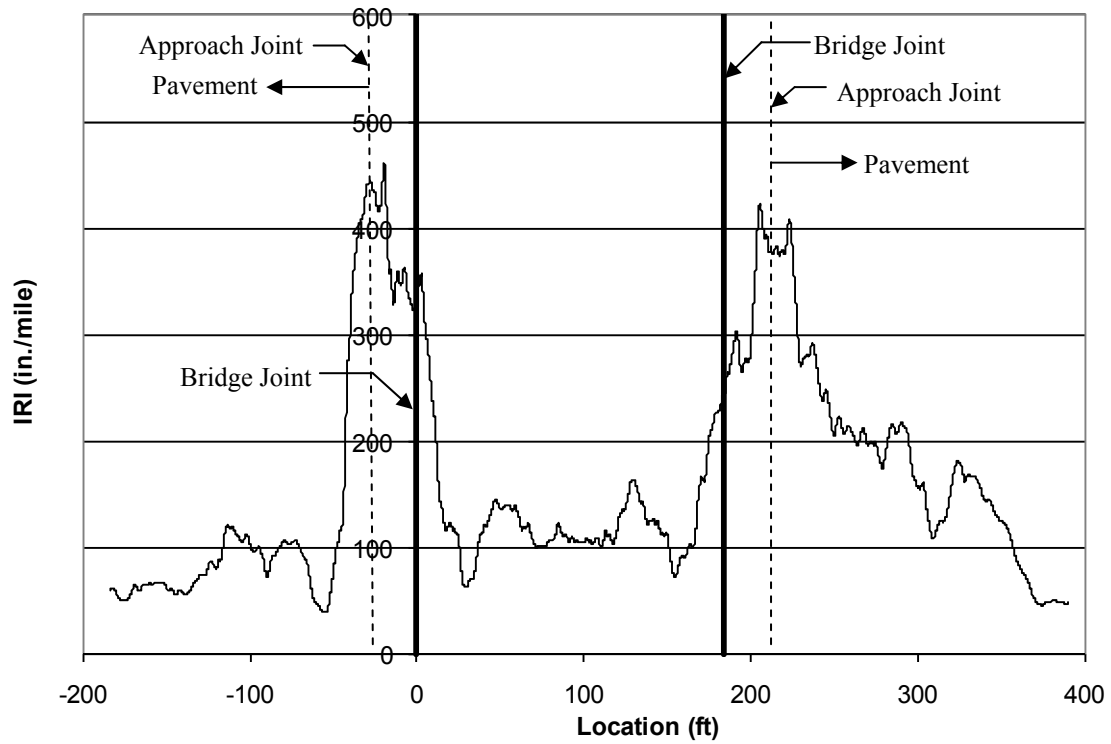


a. Absolute survey data: right wheel line

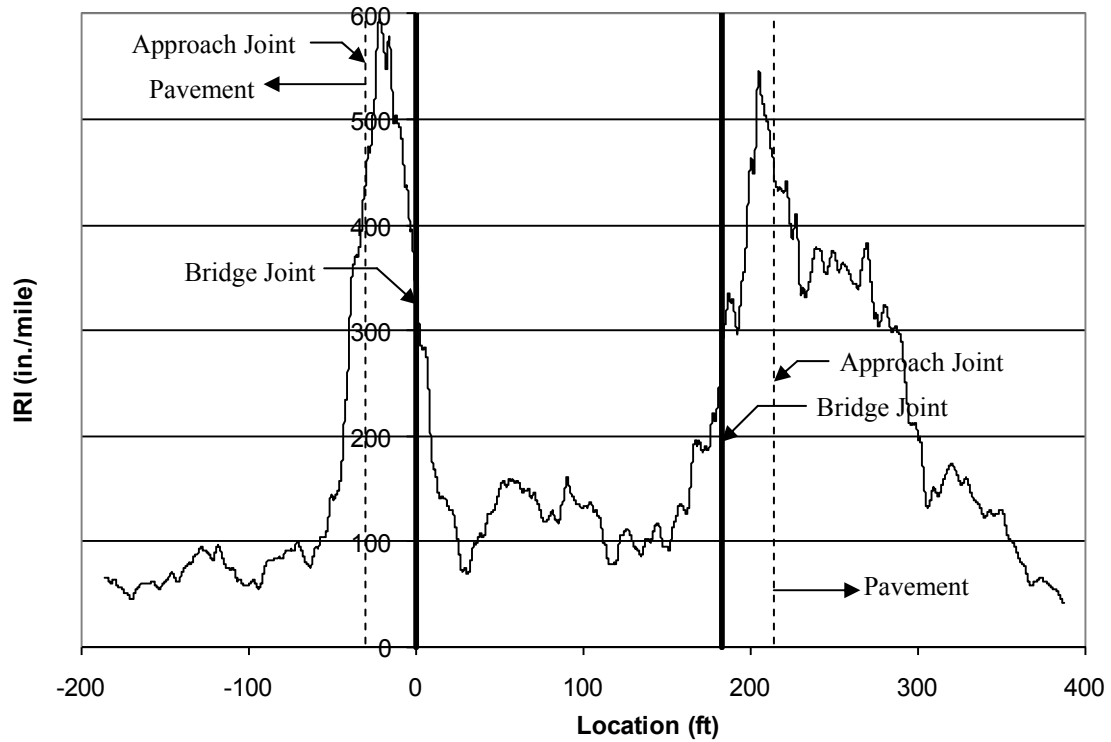


b. Absolute survey data: left wheel line

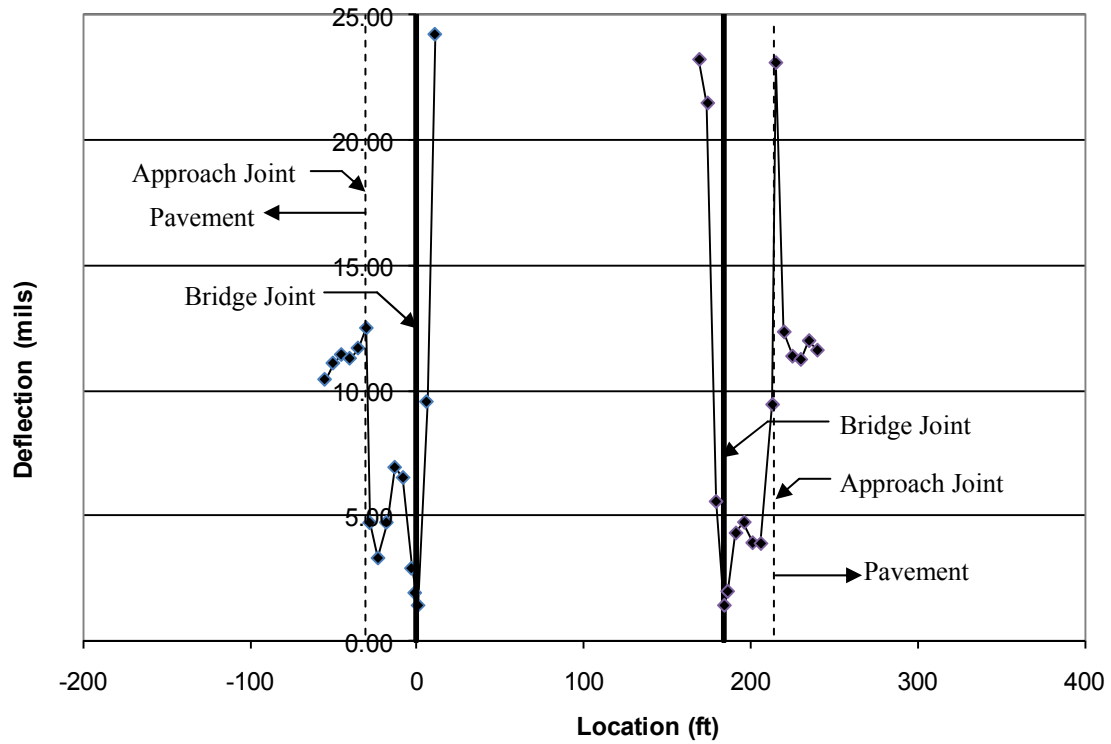




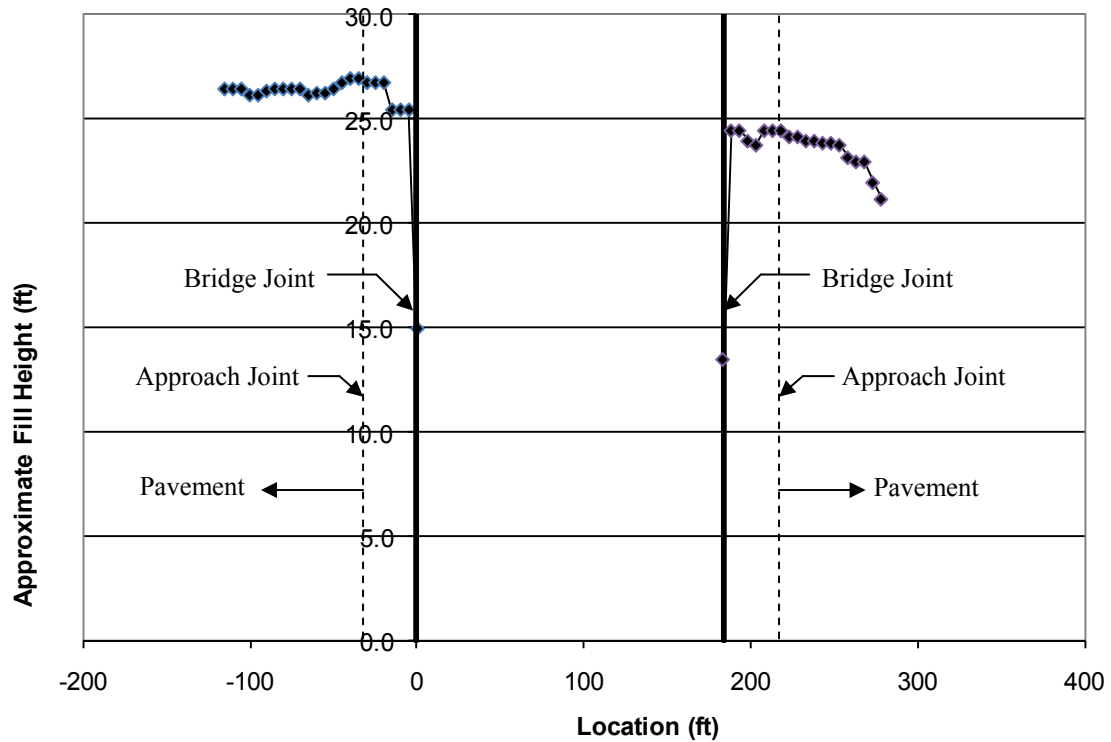
e. IRI: right wheel line



f. IRI: left wheel line



g. FWD test results



h. Approximate fill height

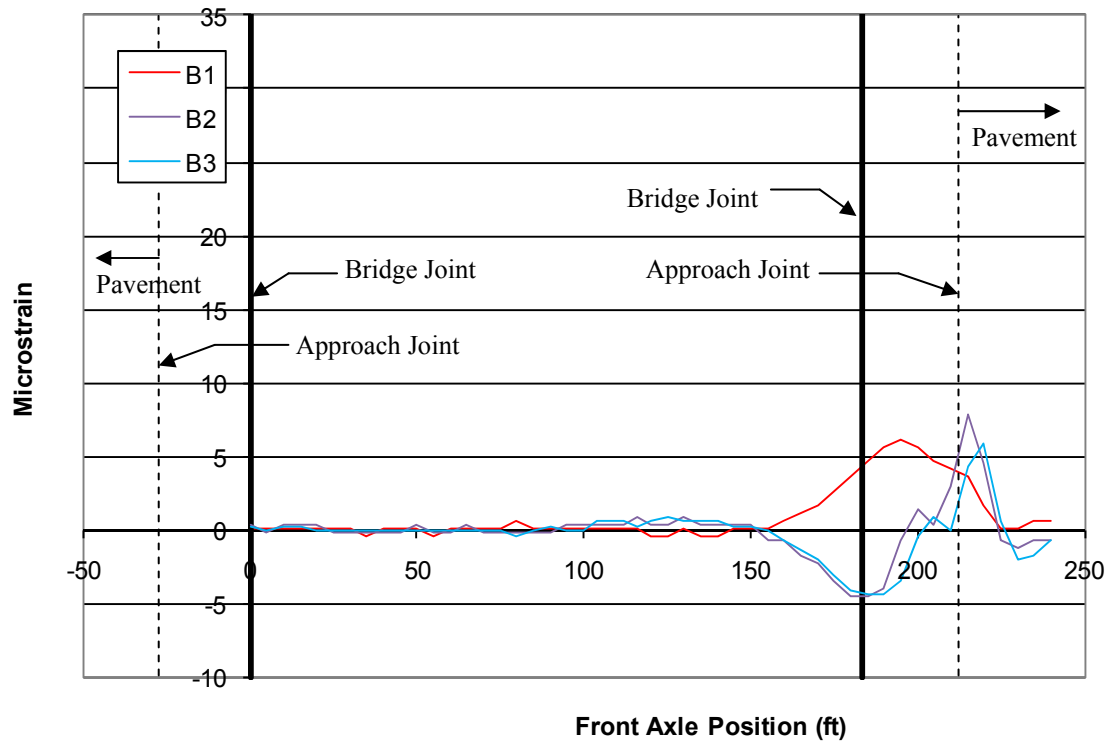
Figure 5.11. FAI 33-14.17 bridge geometric testing results

5.3.4. FAI 33-14.17 Live Load Testing

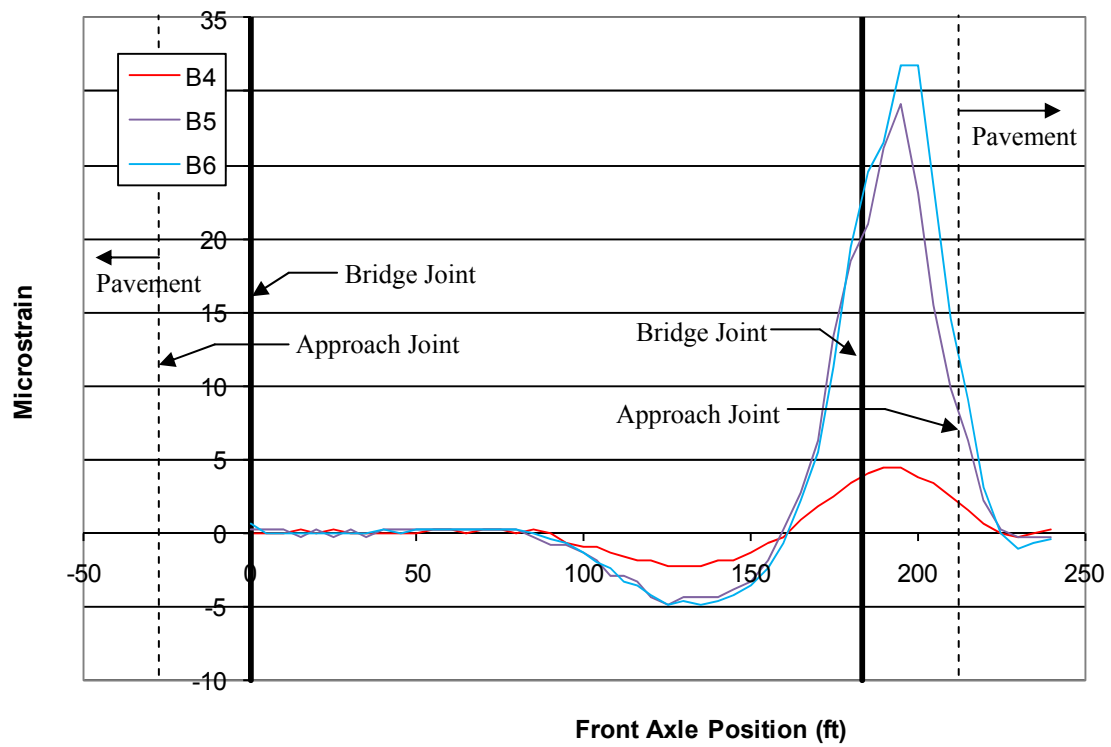
The FAI 33-14.17 bridge was load tested with a loaded legal truck to understand how the bridge behaves under typical live loads. As shown in Figure 5.12, the bridge was instrumented with strain gages on three girders (at two cross sections), with strain gages on the top of the approach slab, with deflection gages measuring relative movement of the approach slab corners, and with abutment rotation and translation sensors.

From the end of the girder strain gages, relatively low strain levels were observed (See Figure 5.13a). However, the reversal of strain sign at this location indicates some unintended end restraint. It also appears that there may be some bending transferred from the approach slab to the bridge. The girder strain response near midspan further shows the unintended end restraint and load transfer from the approach slab to the bridge. Also evident from these data is the overall continuity within the entire bridge. From Figure 5.13d, the approach slab does not appear to be deflecting independently (e.g., it is well connected to the adjoining elements resulting in negligible differential displacement). Note that the “spikes” in the data are likely from outside electromagnetic (EM) interference.

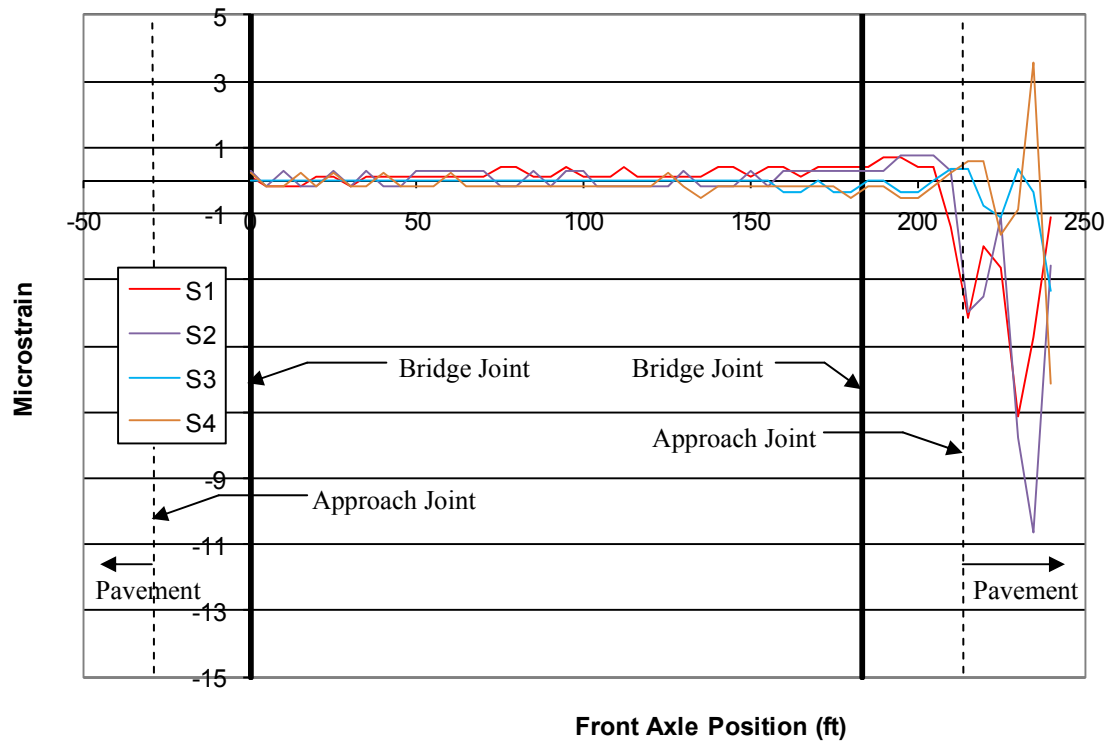
From Figure 5.13e, the abutment does not appear to be rotating under live load (and this is likely the intended behavior). However, Figure 5.13f shows that the abutment is translating. Furthermore, a reversal of movements is apparent—indicating that the abutment is moving both into and away from the supporting soil. This is likely an unintended behavior and potentially could cause for the formation of voids behind the abutment if the support material was not properly designed and installed.



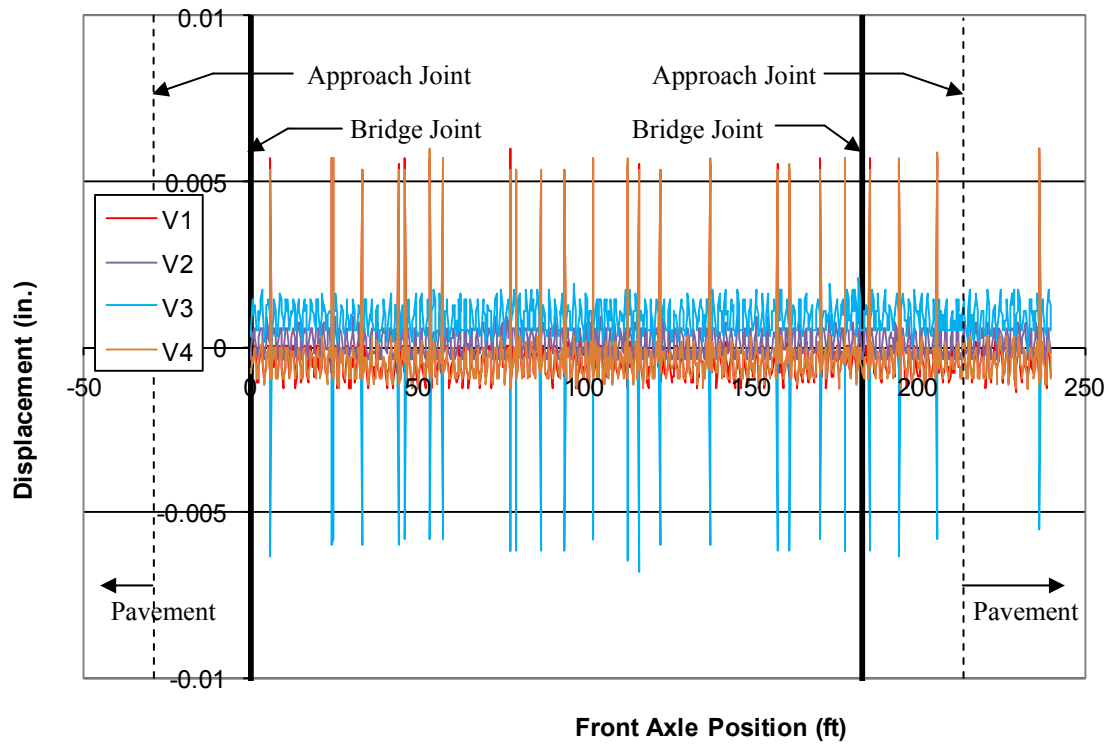
a. Girder strain: near end support



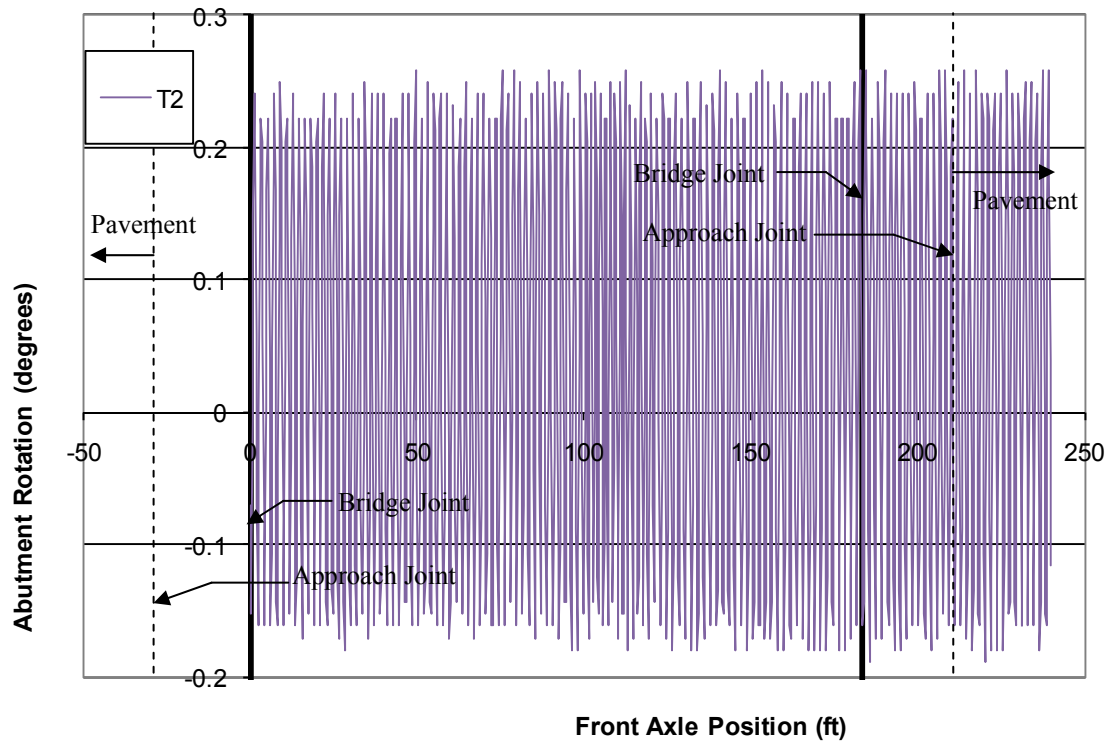
b. Girder strain: near mid-span



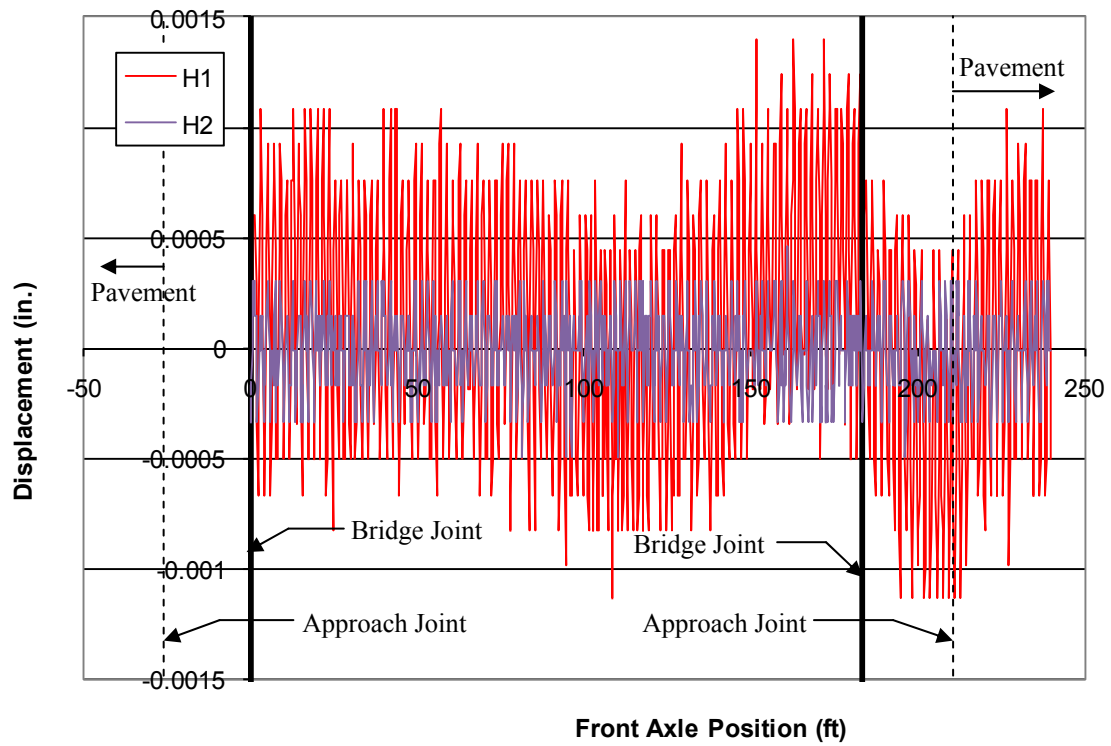
c. Approach slab strain



d. Approach slab vertical movement



e. Abutment rotation



f. Abutment translation

Figure 5.13. FAI 33-14.17 bridge live load testing results

5.4. MUS 16-7.69

5.4.1. MUS 16-7.69 Bridge Description

MUS 16-7.69 is a two-lane, 54 ft, single span bridge located above Schoolhouse Road that was built in 2001. The westbound bridge was investigated. The bridge consists of five W36x170 rolled steel beams with a composite reinforce concrete deck. The bridge has a 23 degree skew at each end. The beams are integrally attached to the abutment using the ODOT typical integral abutment design. The foundation consists of HP10x42 steel pile with 16 in. diameter sleeves. The bridge also has a 45 to 50 ft MSE wall located at each abutment. Figure 5.14 shows the profile of the bridge and MSE wall. The approach slabs are 25 ft long reinforced concrete. The approach slabs are connected to the bridge with #8 diagonal bars. The approach slab has 6 in. of longitudinal bearing at the paving notch. Curbs are located on each shoulder of the slab.



Figure 5.14. MUS 16-7.69 profile and MSE wall

5.4.2. MUS 16-7.69 Visual Bridge Evaluation

The MUS 16-7.69 bridge had multiple signs of distress causing ride quality issues. On the entrance side of the bridge, an 81 ft long asphalt wedge was located over both the mainline pavement and the approach slab. A small dip was located in the mainline pavement approximately 60 ft away from the bridge abutment. At some point, the joint had been filled with grout, which exacerbated the bump problem. The entrance edge of the bridge approach slab was also noted to have approximately 1.5 in. of differential settlement relative to the pavement, as shown in Figure 5.15. Similar issues were present on the exit end of the bridge. A 150 ft long asphalt wedge had been placed starting at the approach slab on the mainline pavement. The joint between the pavement and the approach slab did not allow for bridge or pavement expansion. The joint, at some point, was filled with grout causing a hump as seen in Figure 5.16. The approach joint, in combination with a large dip located just past the approach slab (seen in Figure 5.17), was causing oil staining and what appeared to be scrape marks from cars bottoming out.

Some loss of fill was also evident at locations between the abutment and the MSE as shown in Figure 5.18.



Figure 5.15. MUS 16-7.69 approach slab settlement relative to main line pavement



Figure 5.16. Approach to pavement joint filled with grout causing a hump



Figure 5.17 Dip in pavement when exiting the bridge



Figure 5.18. Fill loss between the MSE wall and the bridge abutment

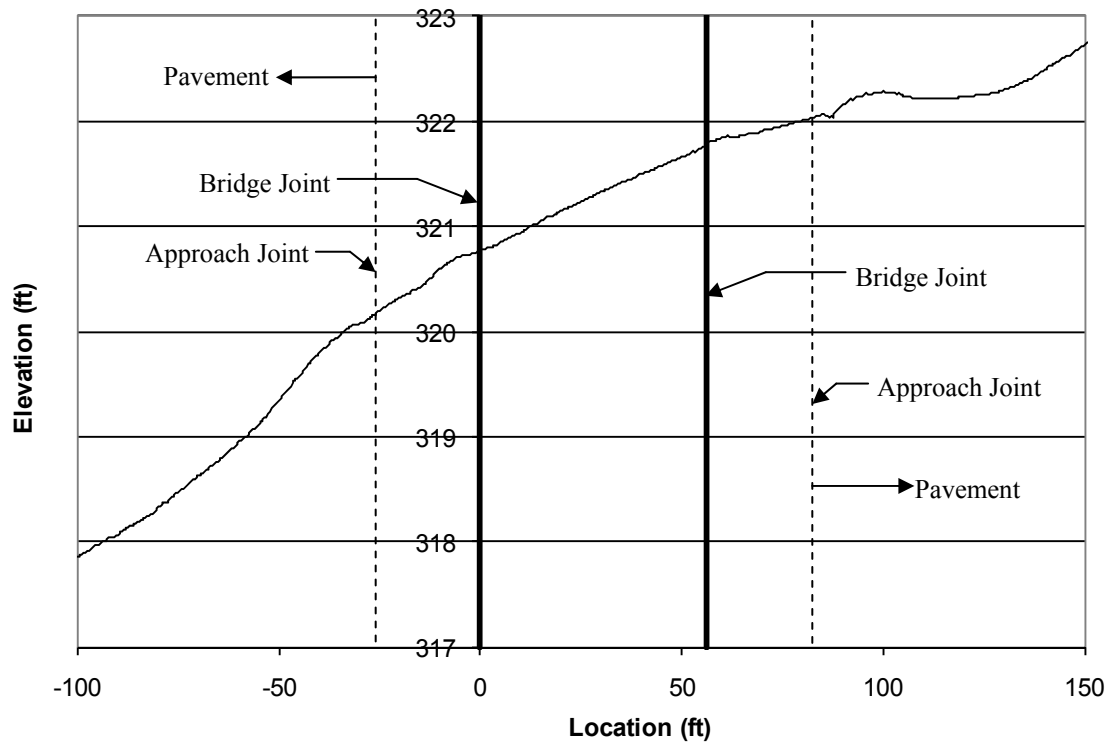
5.4.3. MUS 16-7.69 Geometric Bridge Testing and Support System Evaluation

Four types of “geometric” data for the MUS 16-7.69 bridge were collected and evaluated. Specifically, the global geometry was evaluated using a laser-based survey system, the local roadway condition was evaluated using IRI data collected by ODOT, the support system stiffness was evaluated from FWD test results, and the support system was further evaluated through a review of the overall support system depth.

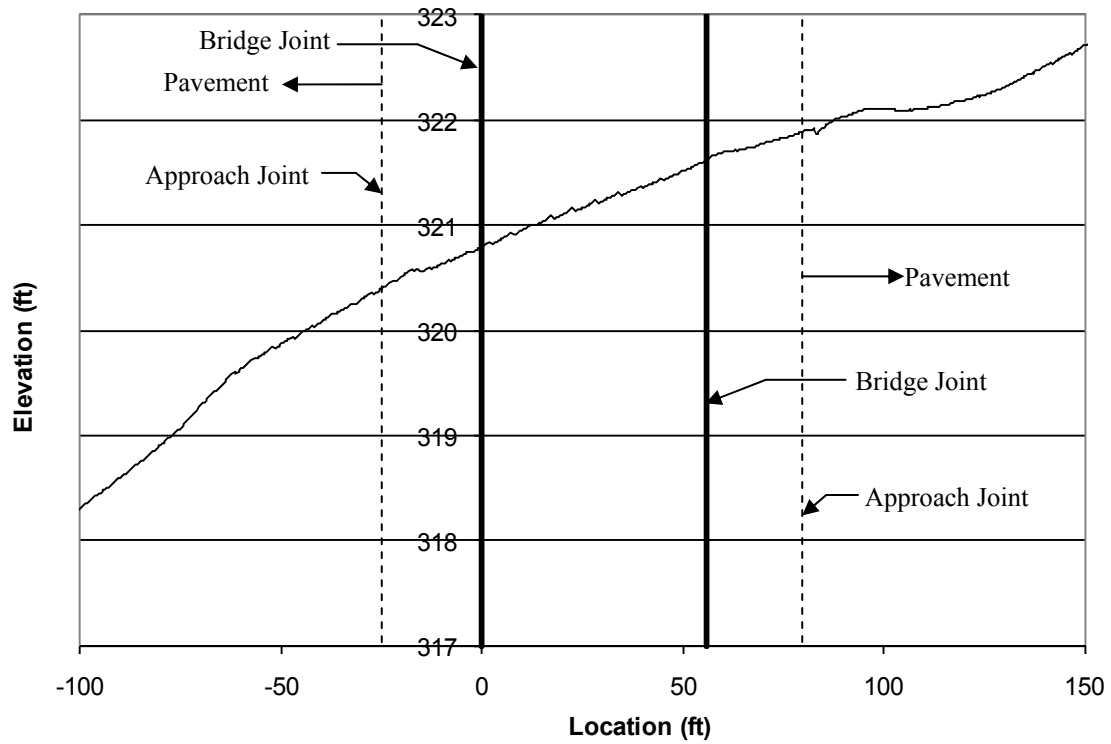
Figure s5.19a and 5.19b show the results of the laser-based survey. As can be seen, the bridge has a notable global slope to it. The most interesting observation from these data is the prominent dip and hump located at the exit end of the bridge. This is likely the dip shown in a previous photograph.

The profiler data (both in raw form and in computed IRI form) indicate that the bridge geometry at the local level is widely varying. Furthermore, the variances are consistent between the left and right wheel lines. The computed IRI data indicate that both wheel lines have IRI values approaching 800 in. per mile. Interestingly, the IRI data are not the “worst” in the area of the dip and hump mentioned previously (which might seem to be the worst from a driver perspective). Rather, the worst IRI values are on either the entrance end or on the bridge itself. These data indicate multiple factors may be impacting the ride quality.

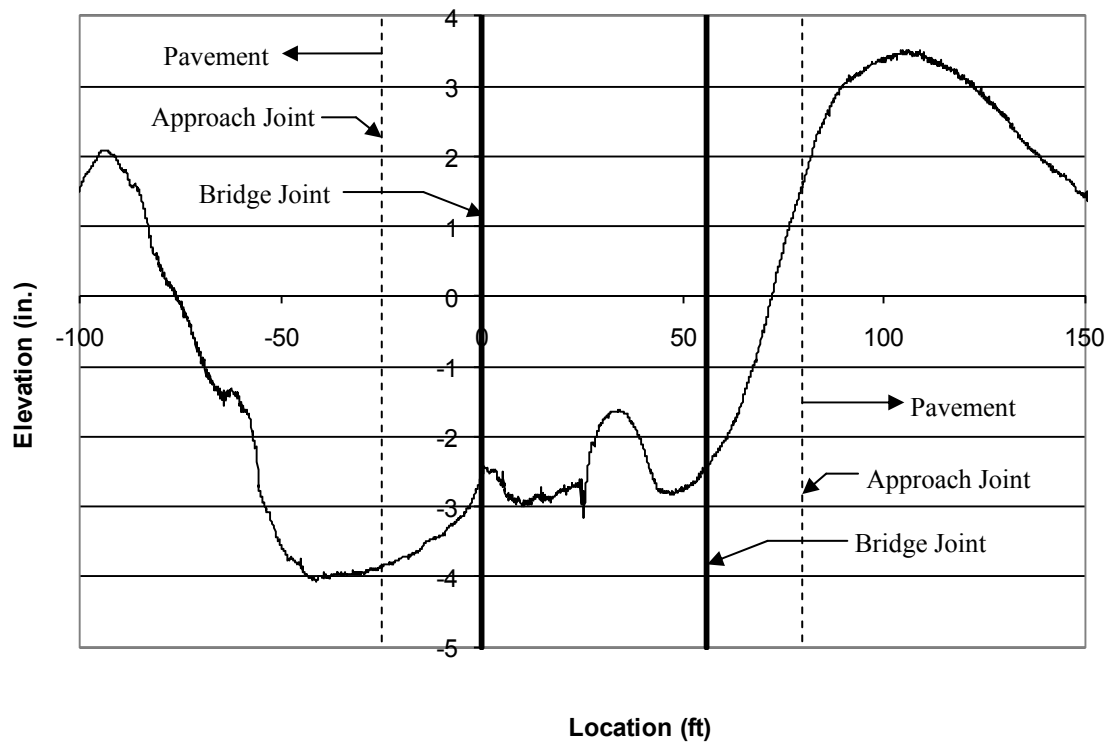
The FWD test results show that, again, the support system stiffness is widely variable. Immediately at the bridge abutment, the support system is very stiff (as expected). The support system consistently softens away from the bridge. At the exit end of the bridge is a gain in stiffness. The location of this stiffness again coincides with the dip and hump mentioned previously. The fill height is quite large with a consistent variation. Such large fill depths give greater opportunity for less than optimum post-construction soil compaction.



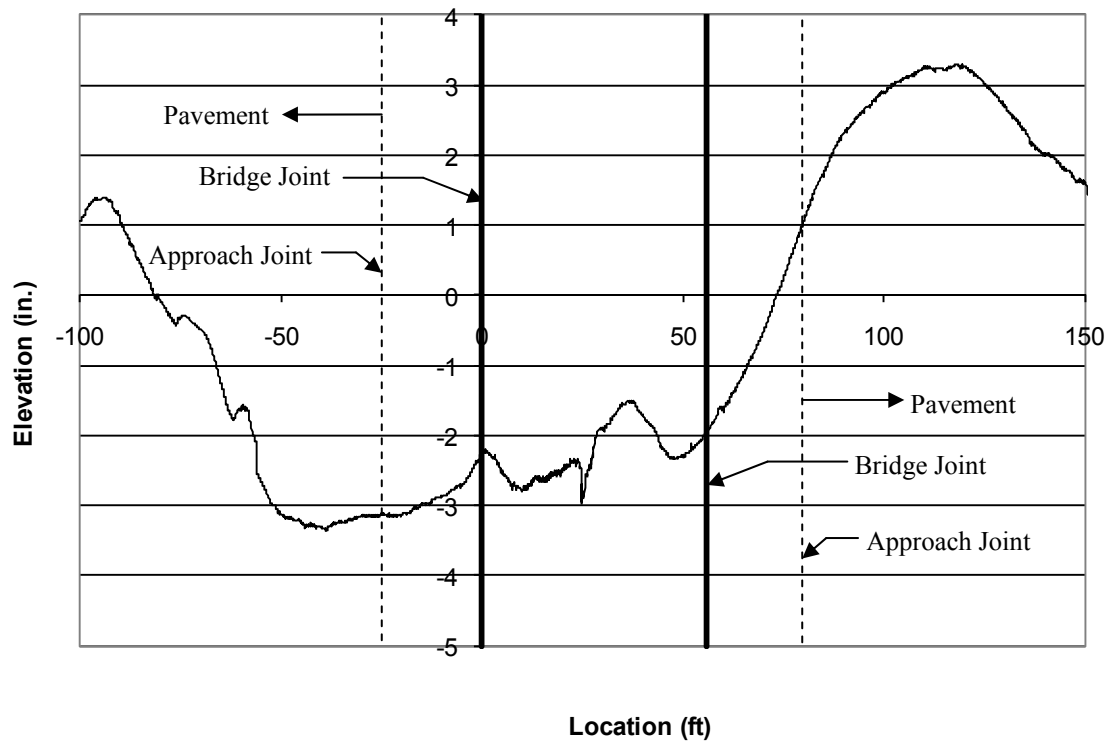
a. Absolute survey data: right wheel line



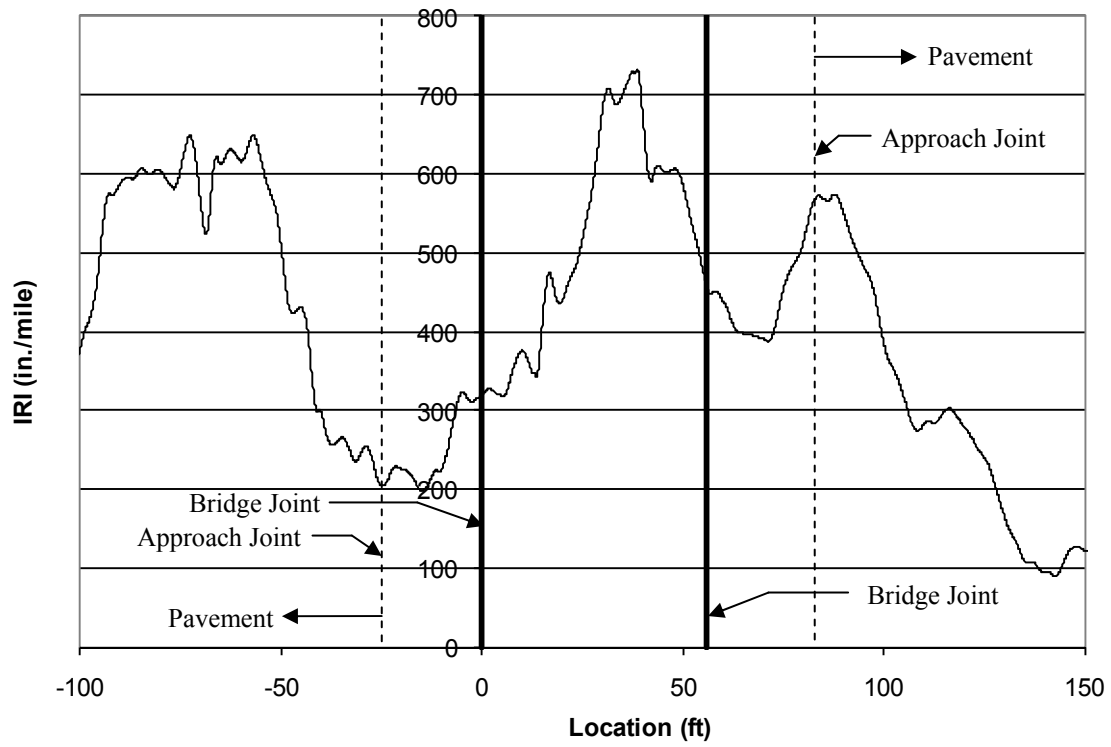
b. Absolute survey data: left wheel line



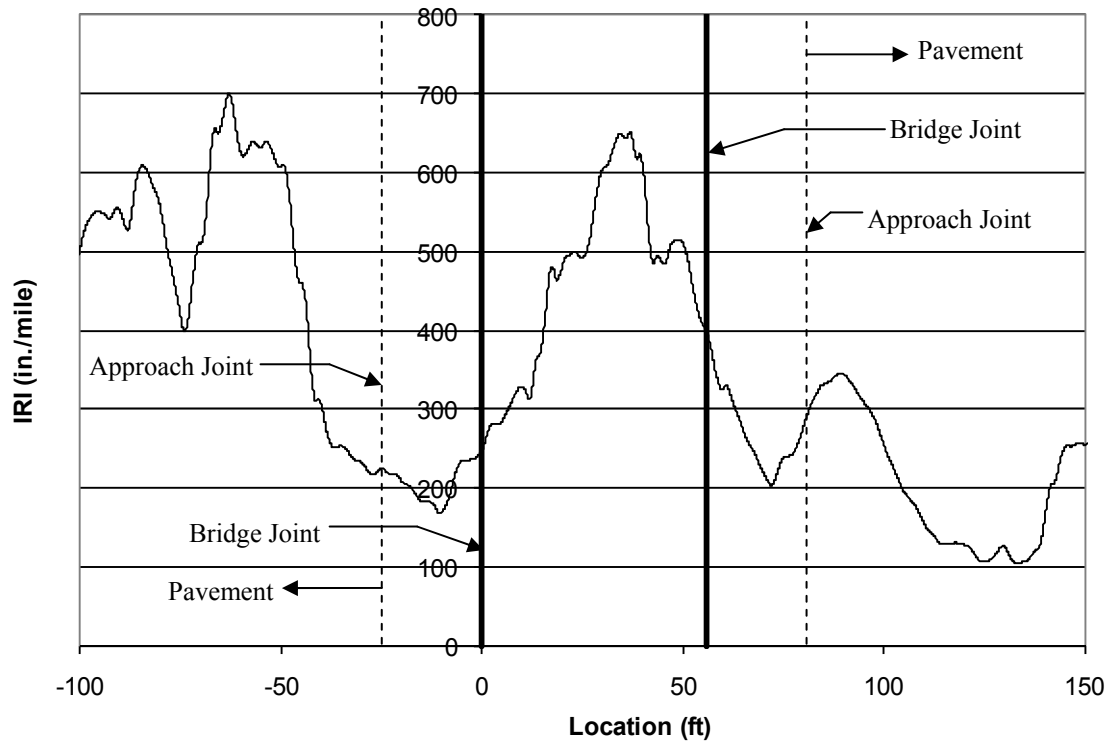
c. Raw profiler data: right wheel line



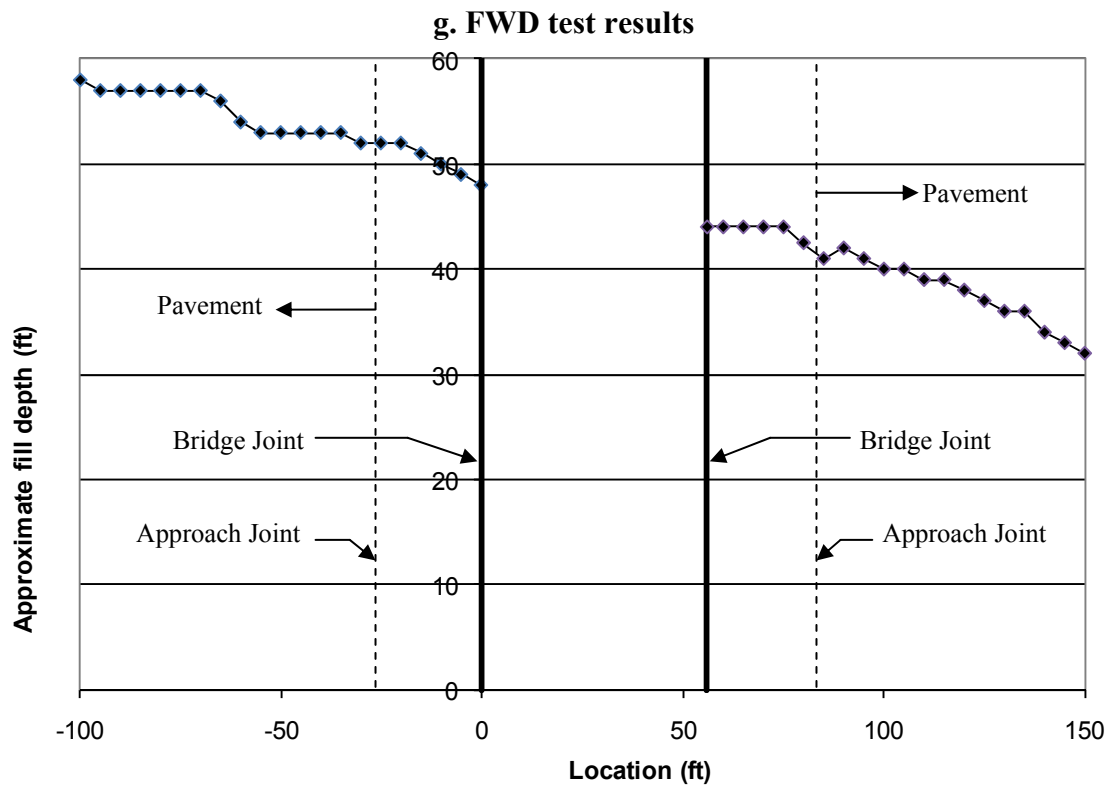
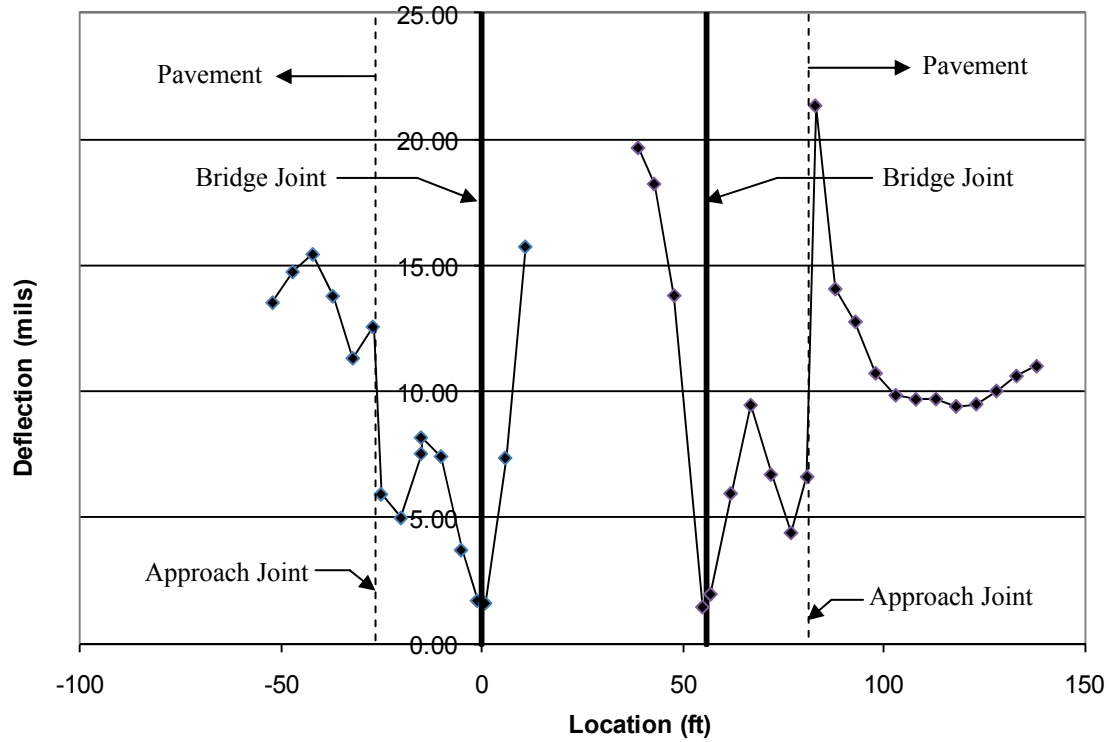
d. Raw profiler data: left wheel line



e. IRI: right wheel line



f. IRI: left wheel line



h. Approximate fill height

Figure 5.19. MUS 16-7.69 bridge geometric testing results

5.5. RIC 430-9.98

5.5.1. RIC 430-9.98 Bridge Description

RIC 430-9.98 is a two-equal span bridge built in 2001 at an existing bridge location. The bridge consists of five 50 in. deep steel plate girder beams with a composite reinforced concrete deck that crosses over Interstate 71. Figure 5.20 shows the underside of the bridge. The bridge has a skew of 35 degrees at both ends. The bridge rests on shallow spread footings, which bear on rock (See Figure 5.21). The girders are connected to the abutment by a semi-integral connection. The approach slabs are 25 ft long and rest on a 6 in. paving notch. The approach slab is attached to the abutment with diagonal #8 bars and has the parapet walls located on the approach slab.



Figure 5.20. RIC 430-9.98 crossing over I-71



Figure 5.21. Abutment conditions at RIC 430-9.98

5.5.2. RIC 430-9.98 Visual Bridge Evaluation

About a month prior to conducting the field test, the bridge approach slabs were mud-jacked, a 50 ft long asphalt wedge was placed on the pavement leading up to the approach slab, and the approach pavement was ground. Figure 5.22 shows the newly surfaced approach on the bridge.

Upon investigation, the approach slab has settled causing the parapet's vertical joint at the bridge to be wider at the top than at the bottom, as shown in Figure 5.23. A 20 in. deep void was found between the wing wall and approach slab, as shown in Figure 5.24.



Figure 5.22. RIC 430-9.98 newly placed asphalt wedge and mud-jacked slab



Figure 5.23. Tapered vertical gap at bridge to approach slab joint indicating settlement

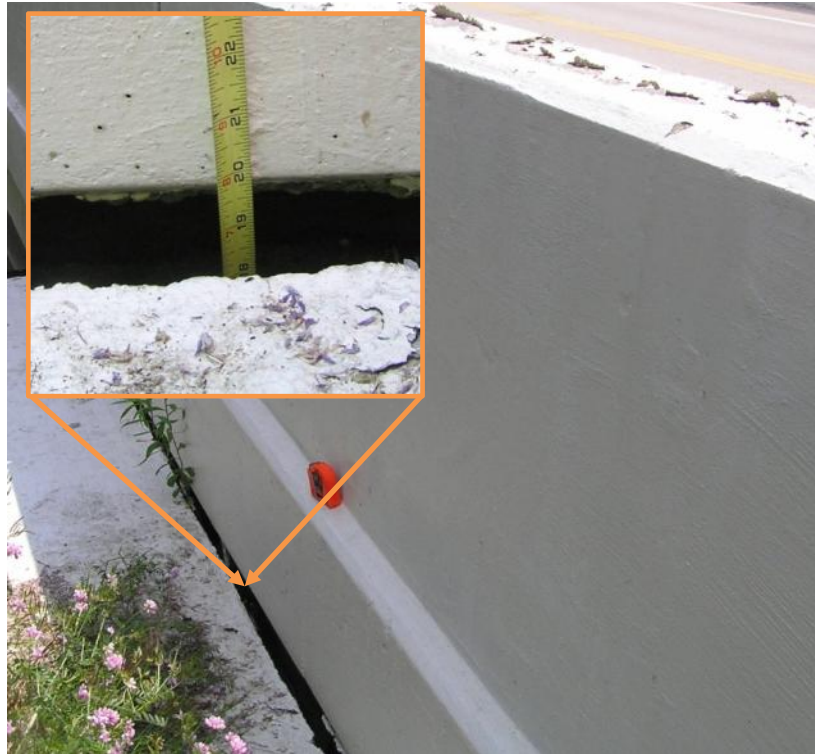


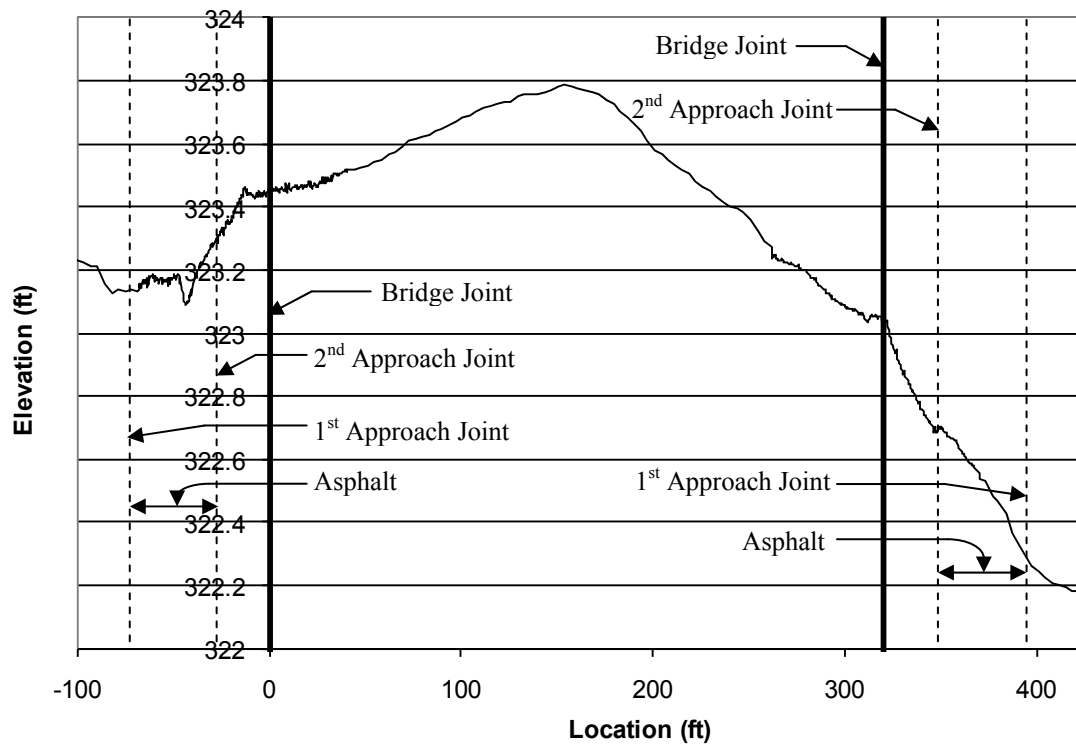
Figure 5.24. Void under approach

5.5.3. RIC 430-9.98 Geometric Bridge Testing and Support System Evaluation

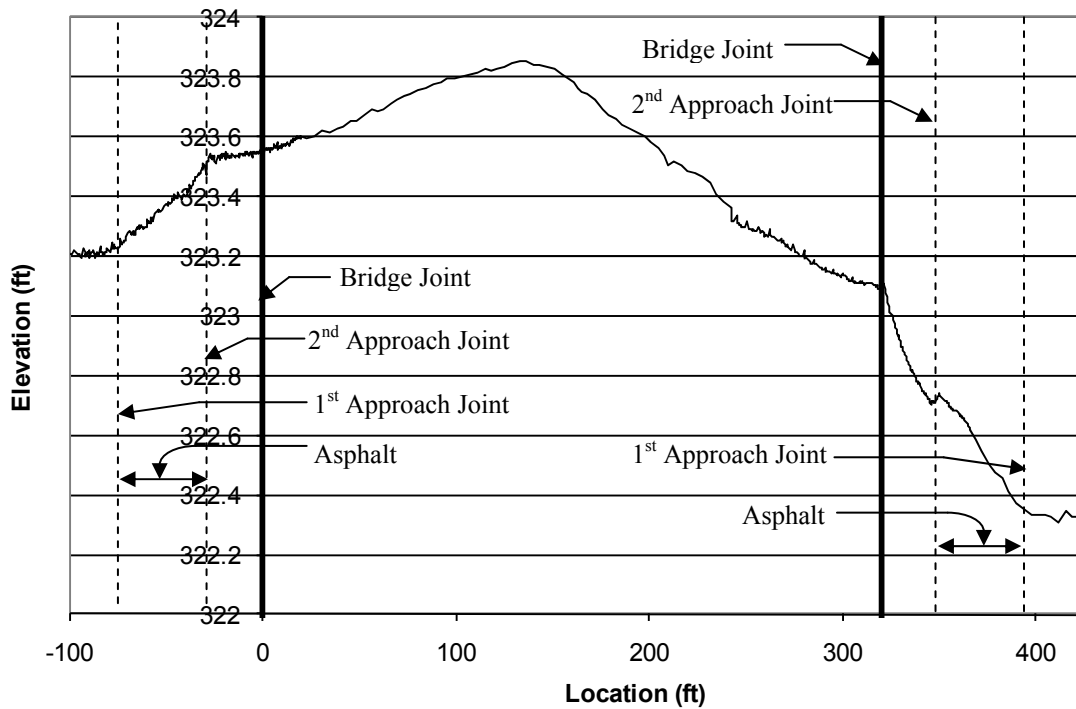
Four types of “geometric” data for the RIC 430-9.98 bridge were collected and evaluated. Specifically, the global geometry was evaluated using a laser-based survey system, the local roadway condition was evaluated using IRI data collected by ODOT, the support system stiffness was evaluated from FWD test results, and the support system was further evaluated through a review of the overall support system depth.

Figures 5.25a and 5.25b show the results of the laser-based survey. Generally, consistency is good between the survey data from the left and right wheel lines. The one exception is on the entrance end of the bridge, where the right wheel line has greater variability. This variability will also be evident in the calculated IRI data. Overall, the bridge has a sweeping vertical geometry that, in Figures 5.25a and 5.25b, appears relatively smooth—a fact that is again corroborated with the IRI measurements.

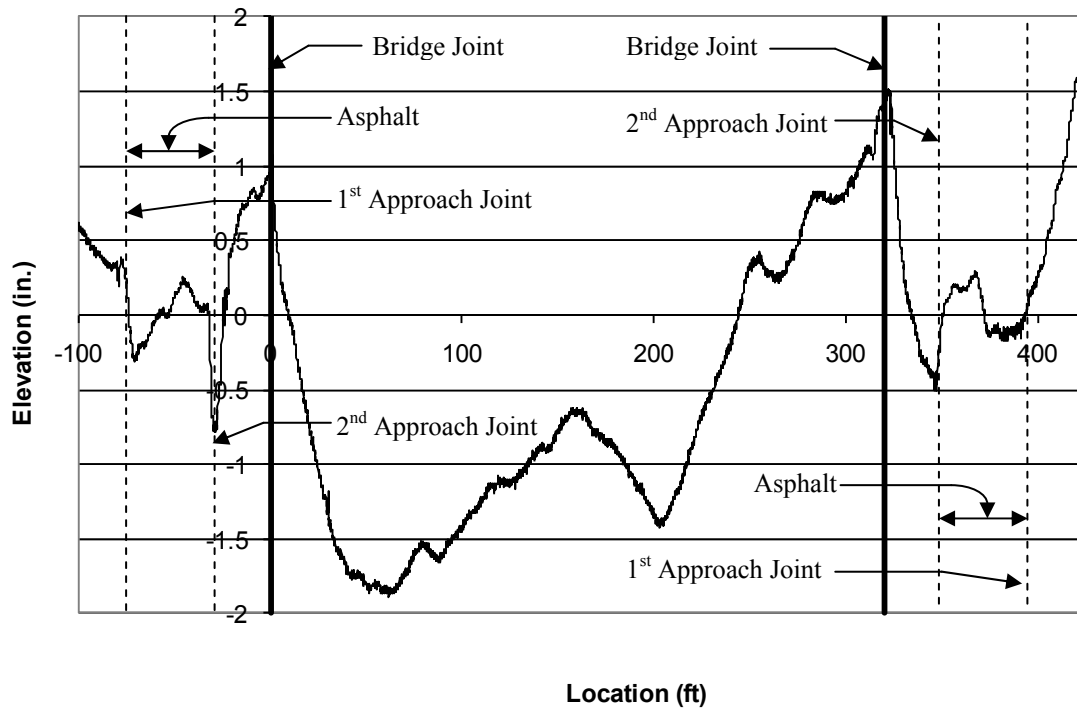
The raw profiler data indicates that, with the exception of the entrance end of the bridge, the two wheel lines show similar characteristics. The difference is even more obvious upon seeing the calculated IRI data, where the right wheel line has IRI values of more than 600 at the entrance. Consistent with the global measurements, the calculated IRI values on the bridge indicate a generally smooth and consistent surface. The problem areas seem to be concentrated at the approach pavement and the bridge joint. The FWD test results indicate a rapid degradation in the support system stiffness on the entrance end. The speculation is that this rapid change in support stiffness may be a source of the entrance conditions mentioned previously.



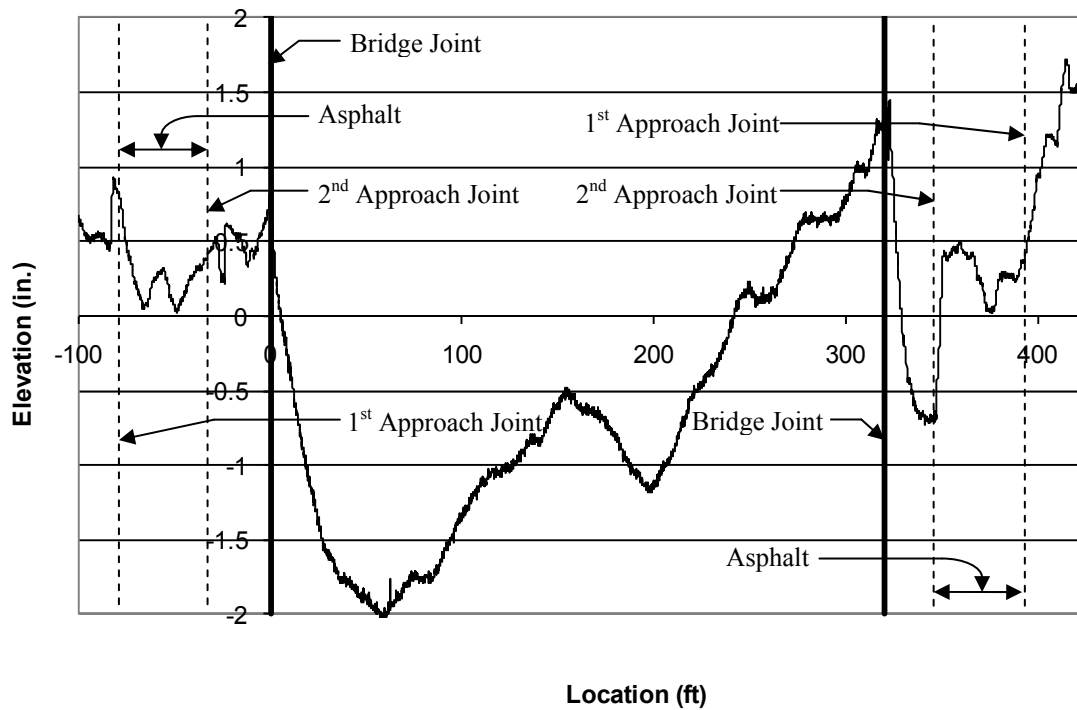
a. Absolute survey data: right wheel line



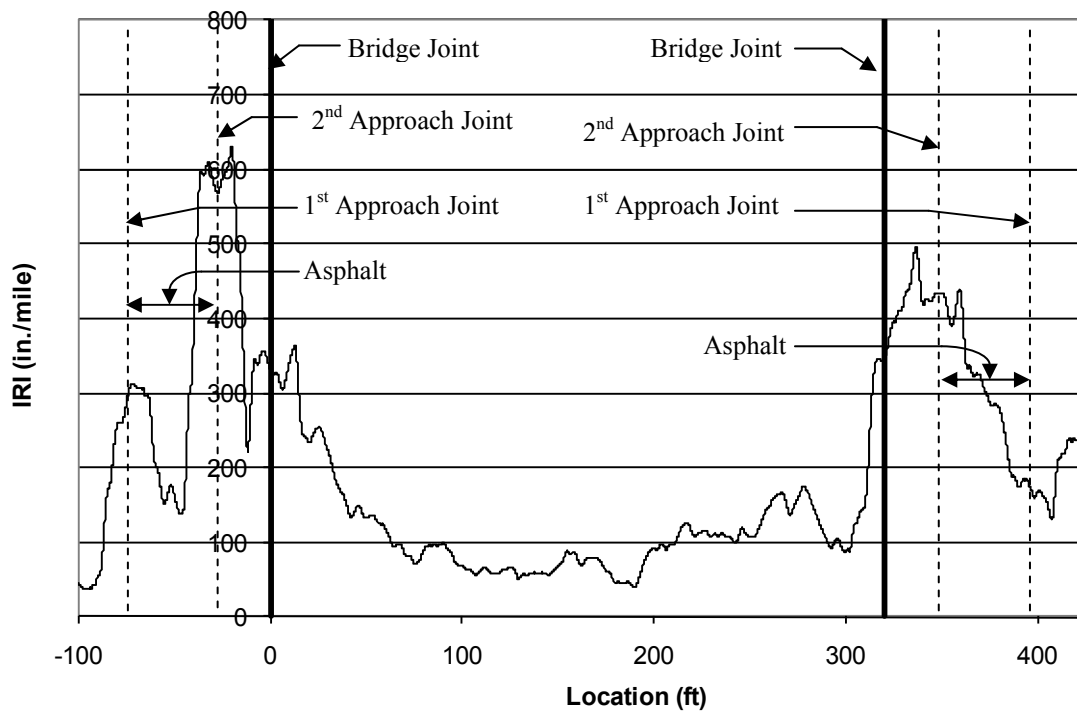
b. Absolute survey data: left wheel line



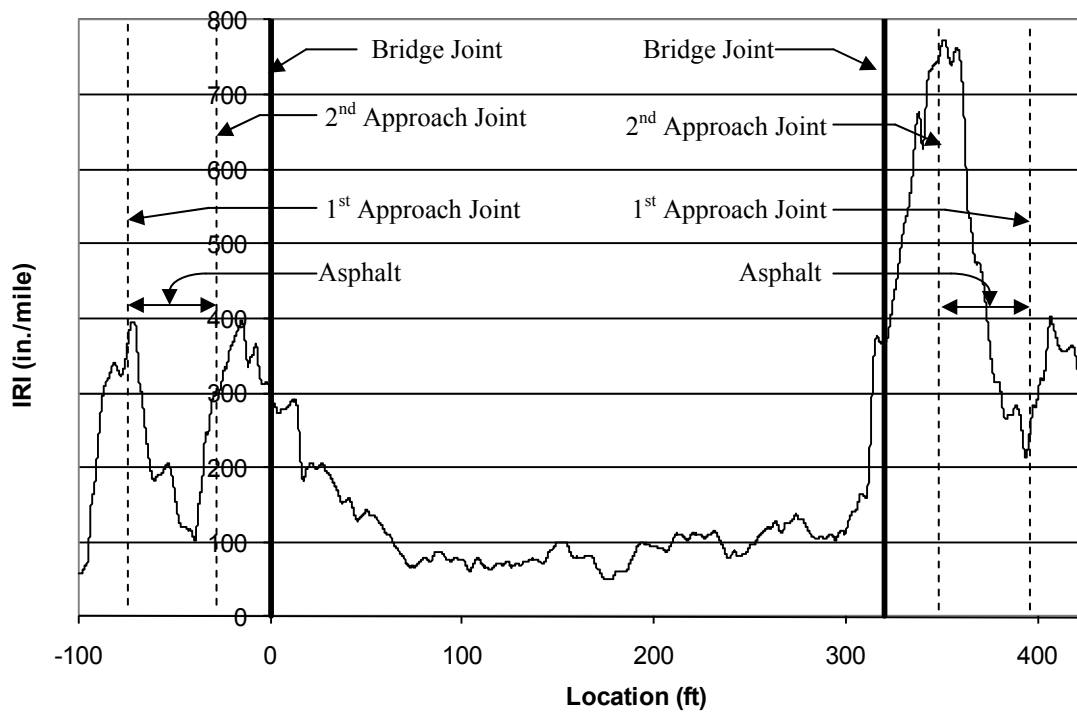
c. Raw profiler data: right wheel line



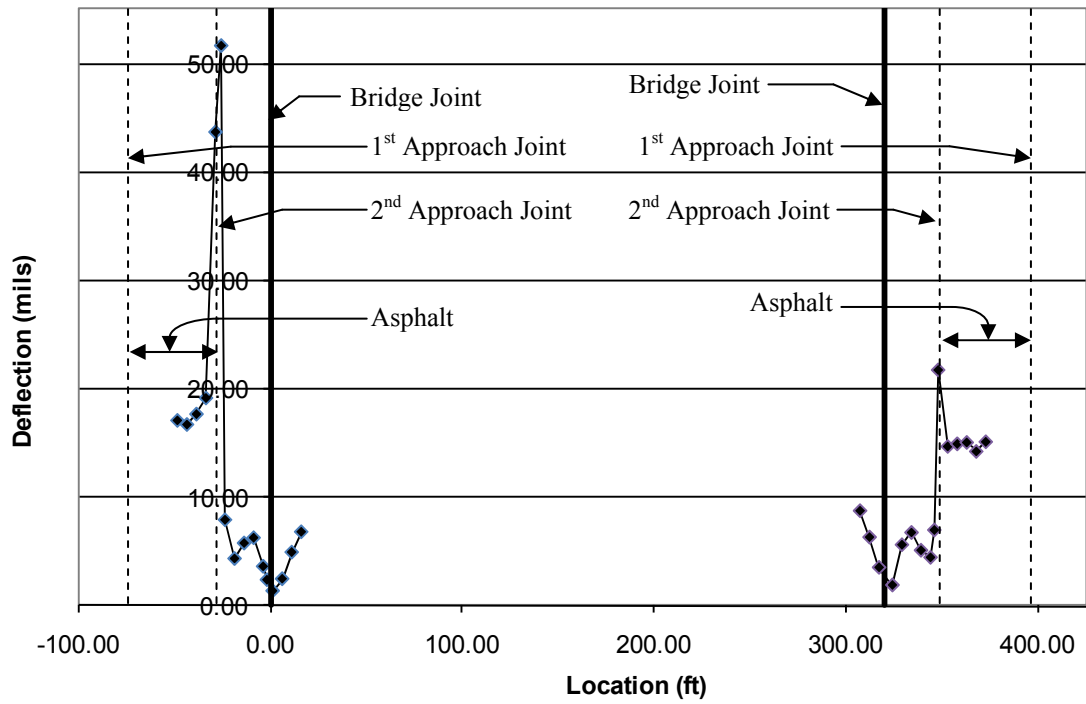
d. Raw profiler data: left wheel line



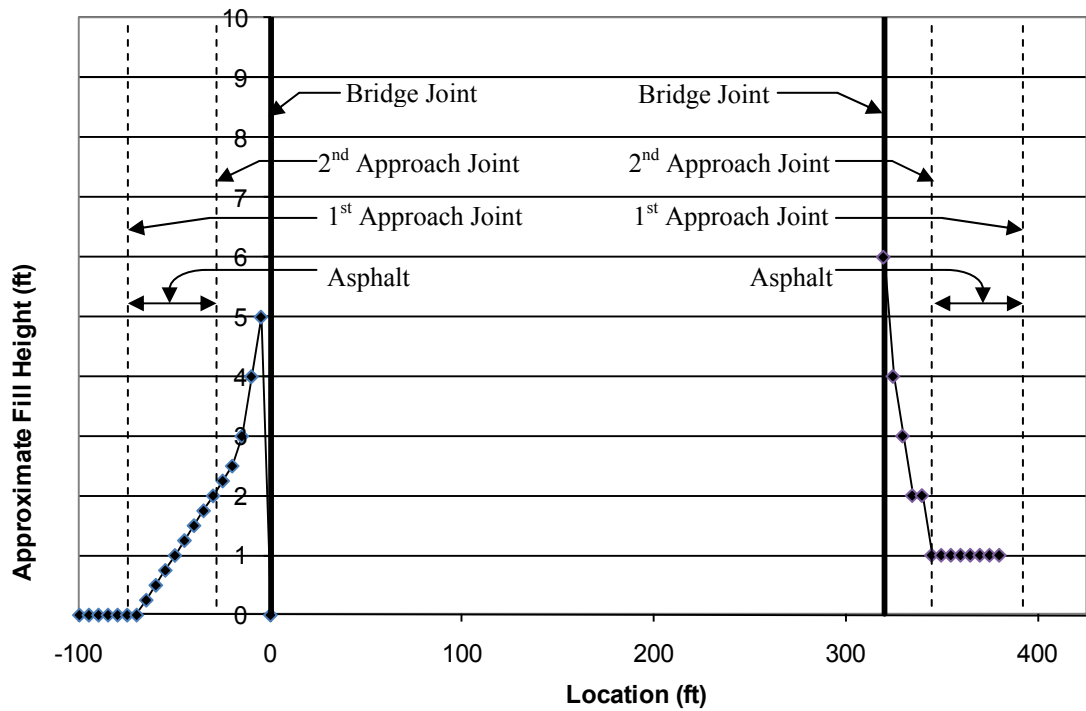
e. IRI: right wheel line



f. IRI: left wheel line



g. FWD test results



h. Approximate fill height

Figure 5.25. RIC 430-9.98 bridge geometric testing results

5.6. FRA 317-8.09

5.6.1. FRA 317-8.09 Bridge Description

FRA 317-8.09 is a 170 ft three-span bridge built circa 1970 at an existing bridge location. The bridge investigated is a two-lane bridge carrying northbound traffic over Blacklick Creek. The bridge is constructed of six rolled W36x135 steel beams with a non-composite reinforced concrete deck. The abutments consist of piles with concrete caps. The girders are attached to the pile cap as an ODOT typical integral abutment. The bridge plans show a 25 ft approach slab unattached on a 6 in. paving notch; however, during the site investigation, no approach slab was evident. Figure 5.26 shows a profile of the bridge.



Figure 5.26. Profile view of FRA 317-8.09

5.6.2. FRA 317-8.09 Visual Bridge Evaluation

The site investigation of the FRA 317-8.09 bridge showed two asphalt wedges placed up to each bridge abutment. The first asphalt wedge was 2 ft wide in the longitudinal direction and located at the abutment as shown in Figure 5.27. The next asphalt wedge, which appeared to be placed several years after the first, was 50 ft long and extended over the 2 ft wedge up to the abutment. Cracking at the pavement-to-bridge abutment joint was also evident and can be seen in Figure 5.27. Evidence of the pavement settling at the bridge abutment was seen at the shoulder of the bridge. Figure 5.28 shows more than a 1 in. difference in the elevation of the pavement and the bridge. The expansion joint material located around the integral abutment bridge has deteriorated allowing soil on the back side of the abutment to erode through the joints.

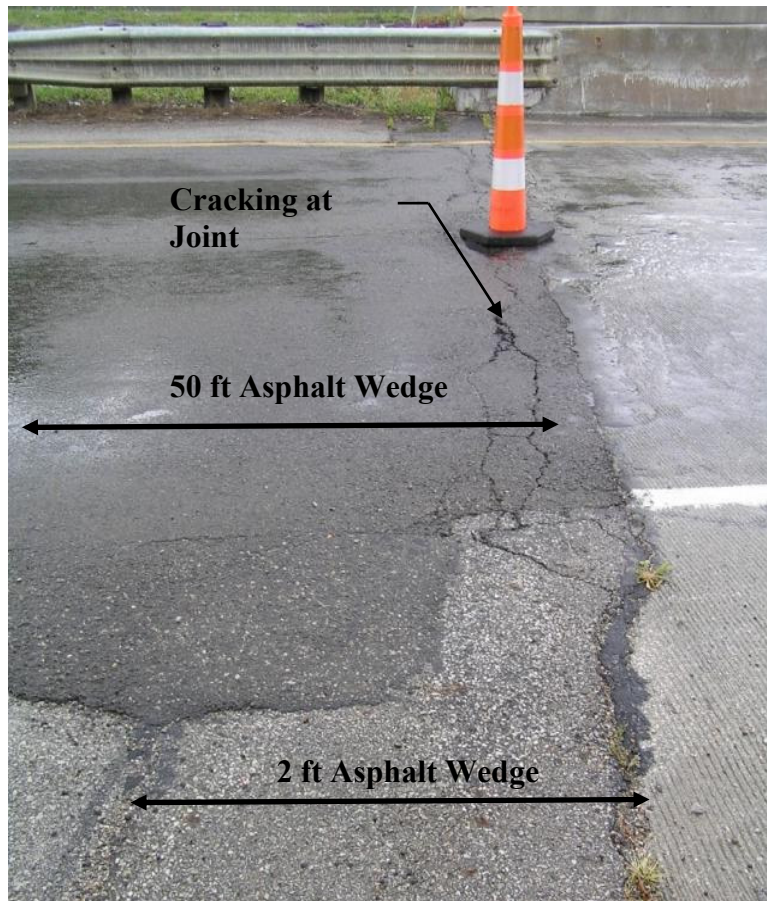


Figure 5.27. Asphalt wedges at bridge abutment



Figure 5.28. Differential settlement at pavement and bridge abutment

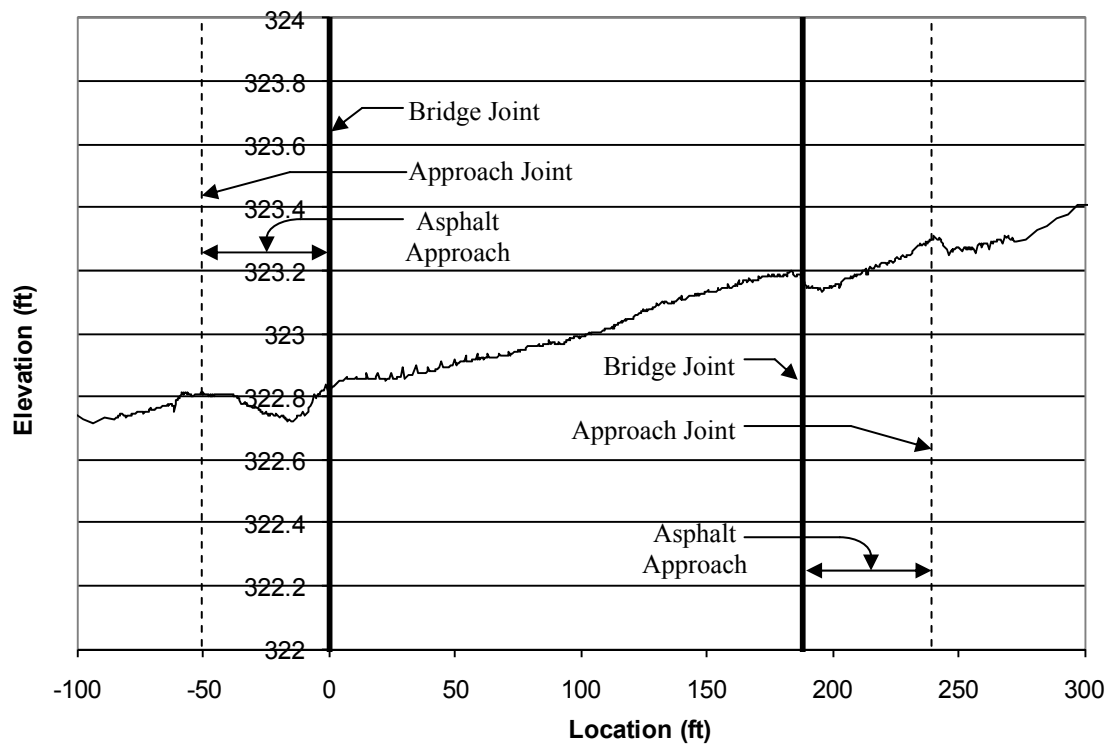
5.6.3. FRA 317-8.09 Geometric Bridge Testing and Support System Evaluation

Four types of “geometric” data for the FRA 317-8.09 bridge were collected and evaluated. Specifically, the global geometry was evaluated using a laser-based survey system, the local roadway condition was evaluated using IRI data collected by ODOT, the support system stiffness was evaluated from FWD test results, and the support system was further evaluated through a review of the overall support system depth.

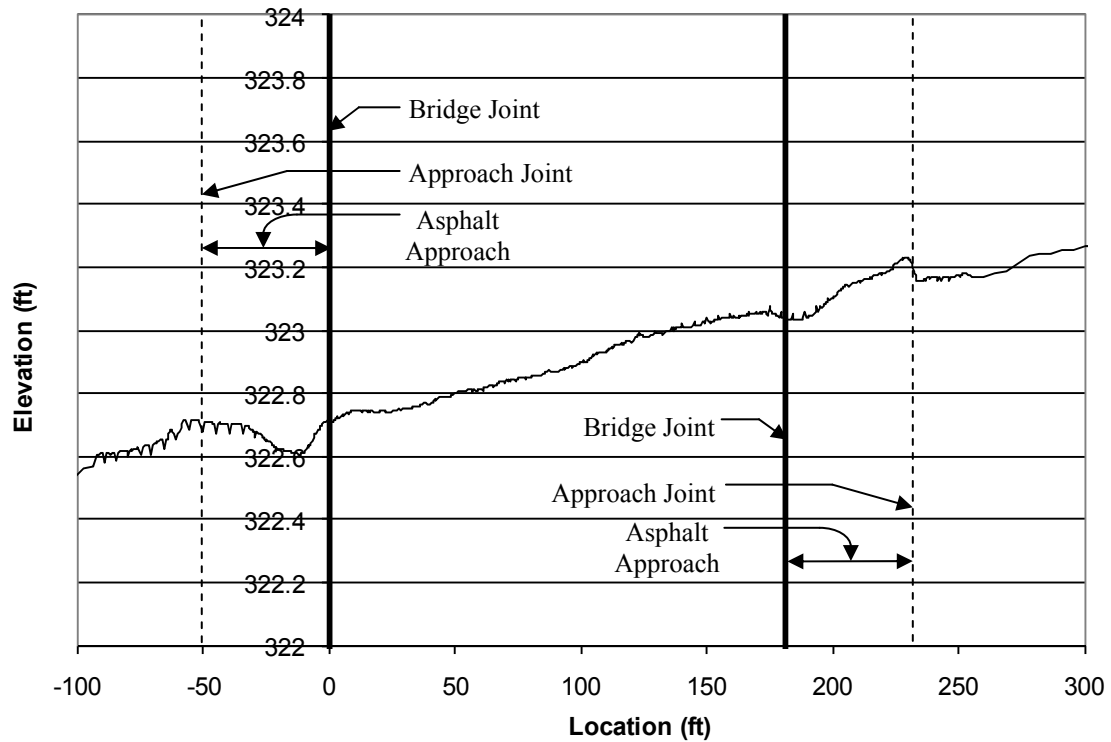
Figures 5.29a and 5.29b show the results of the laser-based survey. The data from the two wheel paths indicate that, generally, the two wheel paths are geometrically similar. Further observation of the data does not show a smoothly flowing geometry. In both wheel lines, prominent dips can be observed at both the entrance and exit ends and also a large hump on the exit. Note that the “fuzzy” appearance of the data is not believed to be real; rather, it is believed to be a remnant of errors propagating through the process of stitching together multiple data sets.

The profiler and IRI data shown in Figures 5.29c through 5.29f show that the local geometry of the bridge is comparable in each wheel path and that there is significant local variation. The calculated IRI data indicate that the worst ride characteristics are on the exit end of the bridge. This fact compares well with the findings from the global geometric measurements.

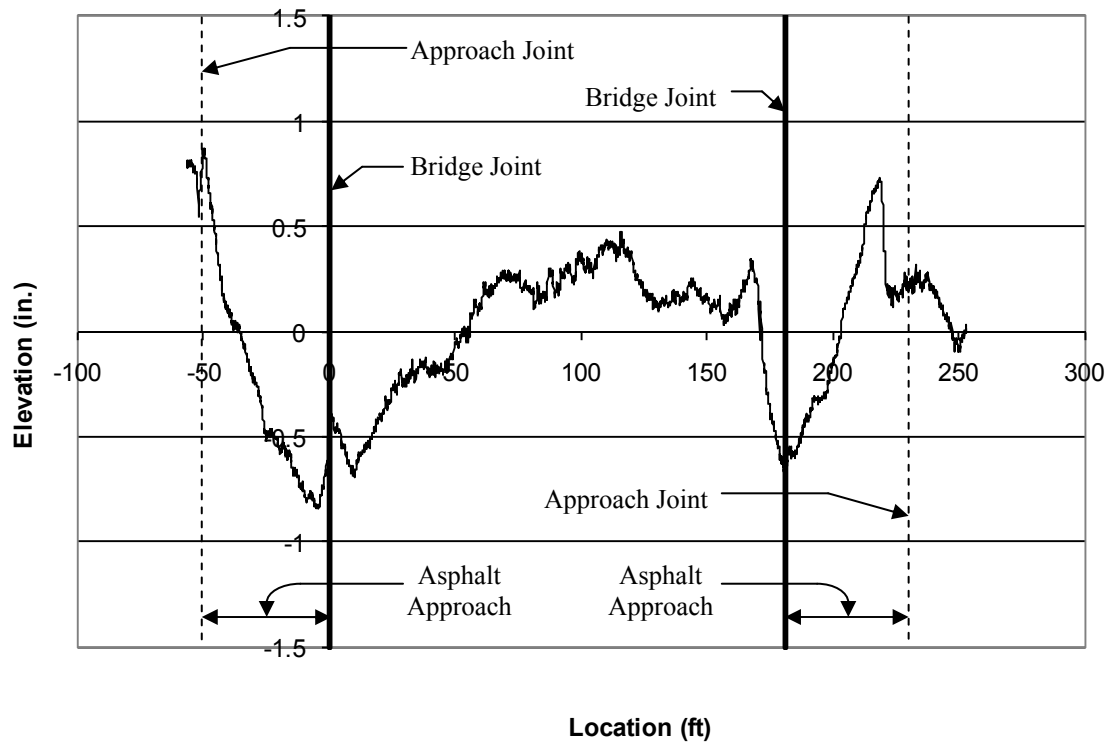
The FWD test results indicate that there is general similarity in the stiffness on and off the bridge. However, it can clearly be seen that the support system stiffness is extremely high in the immediate vicinity of the bridge abutment. The approximate fill heights vary significantly from the entrance to the exit bridge ends. These differences may contribute to the difference in observed roadway geometry (both local and global).



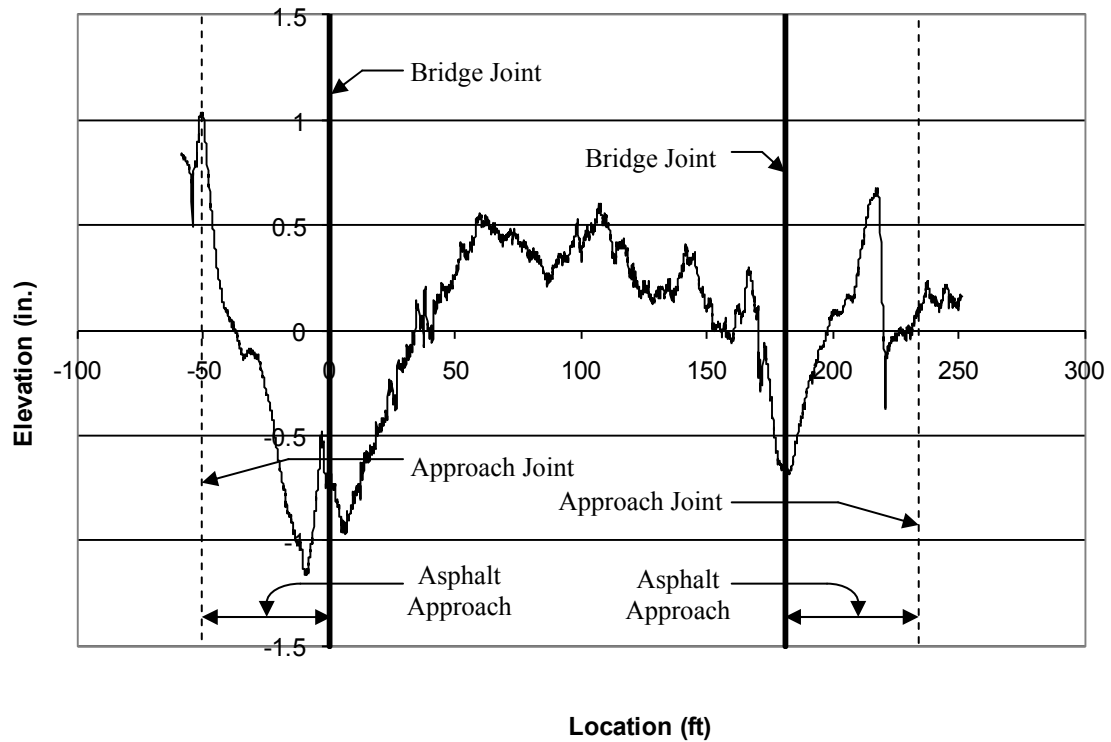
a. Absolute survey data: right wheel line



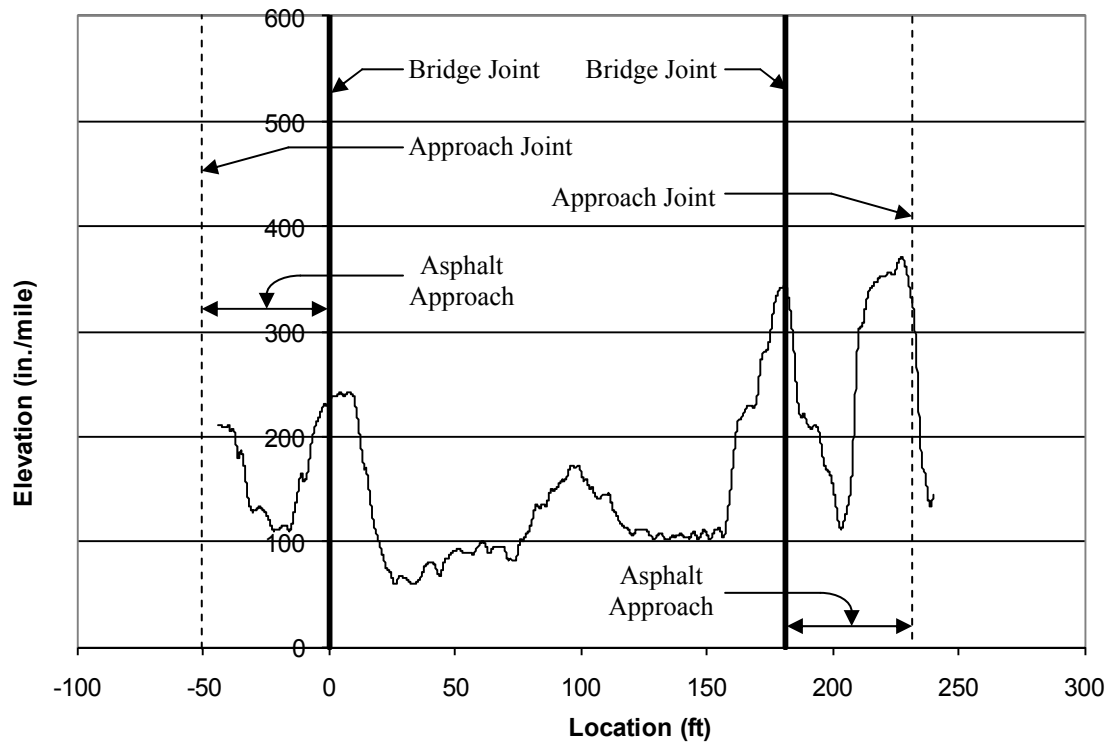
b. Absolute survey data: left wheel line



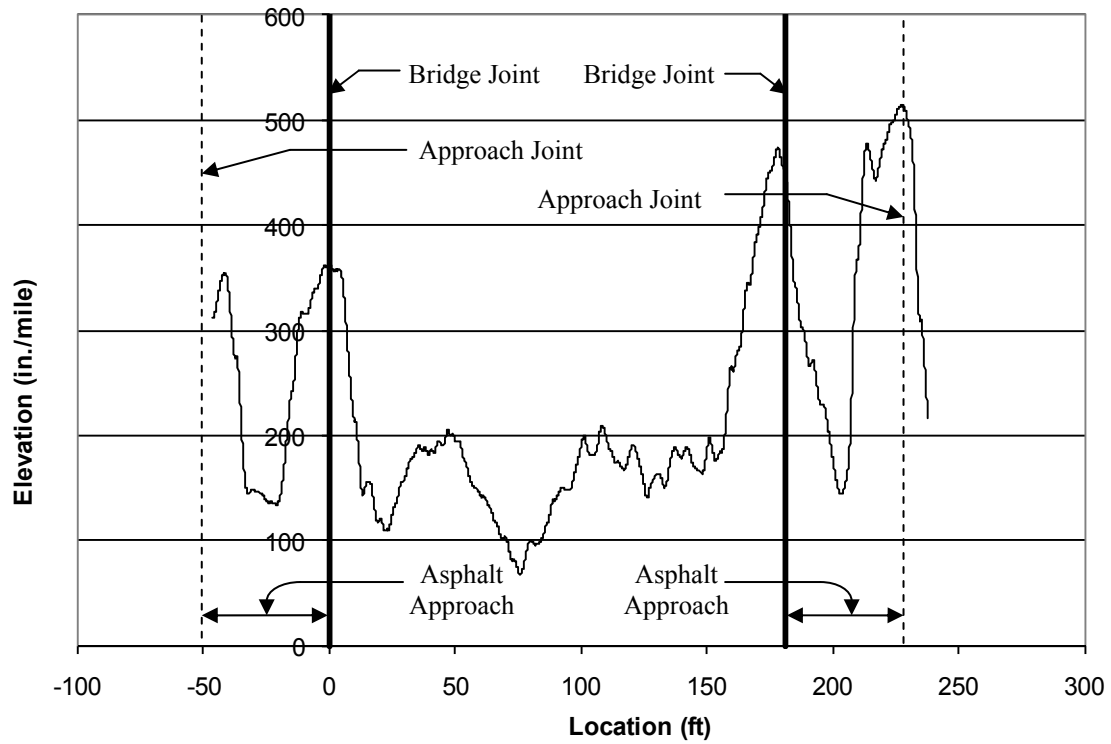
c. Raw profiler data: right wheel line



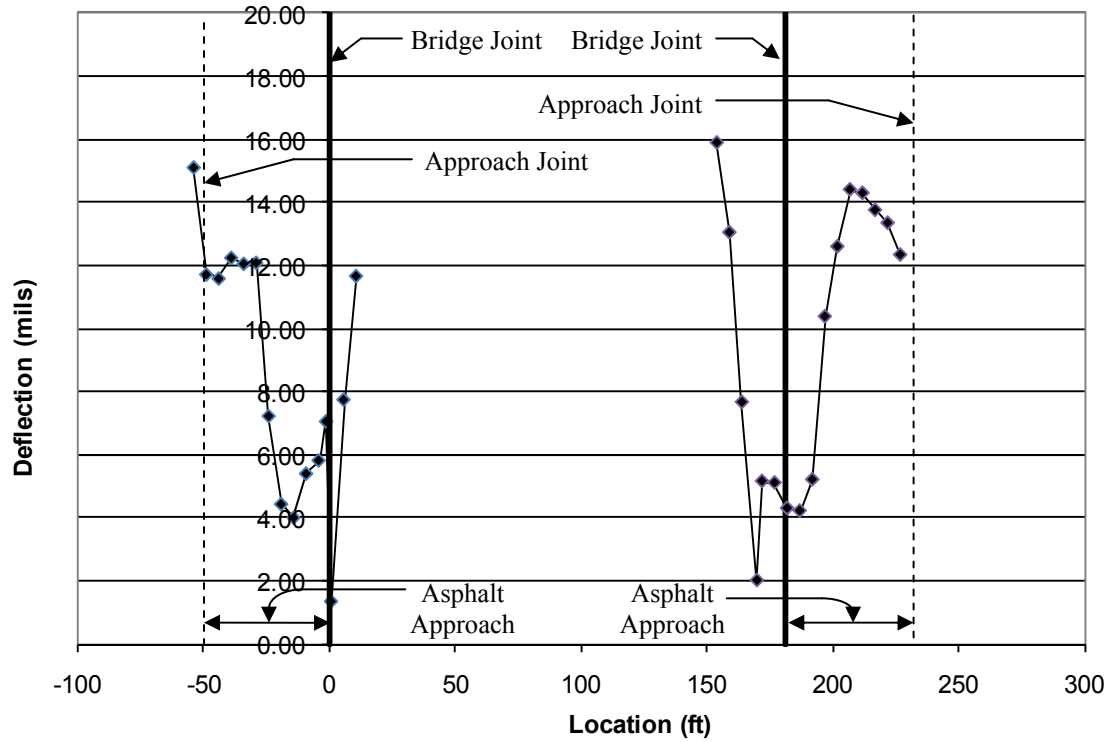
d. Raw profiler data: left wheel line



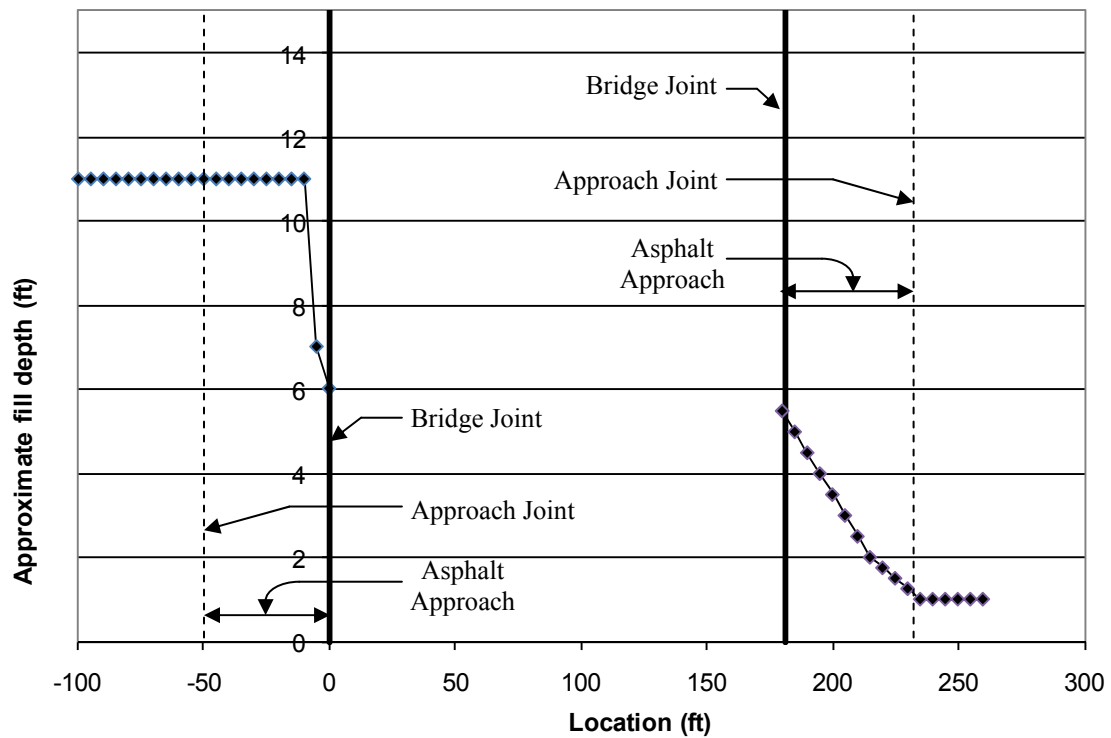
e. IRI: right wheel line



f. IRI: left wheel line



g. FWD test results



h. Approximate fill height

Figure 5.29. FRA 317-8.09 bridge geometric testing results

5.7. PRE 70-12.49

5.7.1. PRE 70-12.49 Bridge Description

The original PRE 70-12.49 bridge was built in about 1962 at a new bridge site. The bridge crosses over Price Creek and Price Creek Road as shown in Figure 5.30. The eastbound bridge was investigated and consists of four spans with a total length of 220 ft and the longest span being 63 ft. In the past decade, the bridge was widened, adding three more steel beams and changing the abutments to an integral or semi-integral type abutment. Figure 5.31 shows the bridge beams and abutments. The abutments consist of vertical and battered steel piles with concrete caps. The bridge has no skew. The approach slabs are 25 ft long concrete and are supported on a 6 in. paving notch. No reinforcing attachment of the approach to the abutment is noted on the plans. The approach slabs appear to be relatively new.



Figure 5.30. Profile of PRE 70-12.49 prior to widening



a. Widened bridge



b. Bridge abutment

Figure 5.31. PRE 70-12.49 after bridge widening

5.7.2. PRE 70-12.49 Visual Bridge Evaluation

Visual inspection of the bridge revealed the bridge-to-approach slab joint to be in good condition. However, the interface between the approach slab and pavement was in poor condition. Figures 5.32a and 5.32b show the approach-to-bridge and approach-to-pavement joints. The pavement asphalt at the joint has deteriorated creating a poor riding surface at both the entrance and exit of the bridge. At some location, the joint has deteriorated up to 16 in. and had been filled with asphalt patch materials. The approach slab also appears to be settling as indicated by the joint in the barrier rail at the bridges. Figure 5.33 shows the vertical joint in the barrier rail at the approach-to-bridge joint. The gap in the joint is larger at the top than at the bottom indicating the slab is tilting and creating a ramp effect. Long oil stain patches were found approximately 50 ft down traffic of the deteriorated pavement joint. Figure 5.34 shows the oil stain on the bridge.



a. Approach-to-bridge joint



b. Approach-to-pavement joint

Figure 5.32. Condition of bridge and approach joints



Figure 5.33. Barrier rail joint at approach slab showing settlement of approach slab



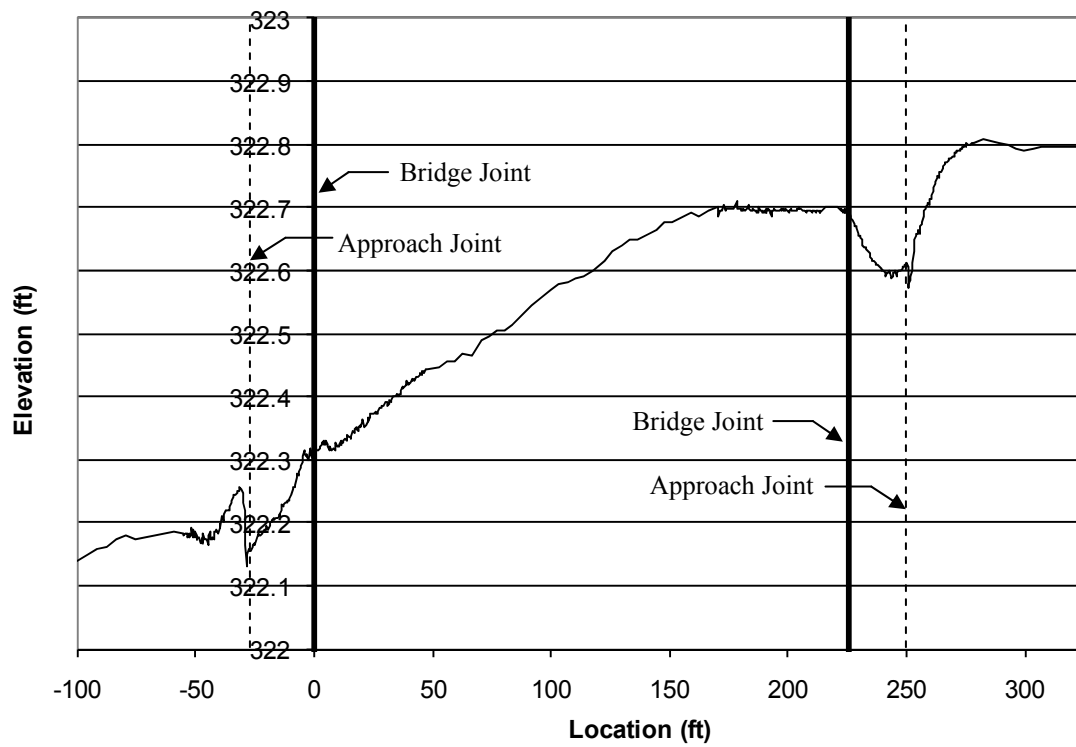
Figure 5.34. Oil staining on bridge surface caused by bump at pavement-to-approach joint

5.7.3. PRE 70-12.49 Geometric Bridge Testing and Support System Evaluation

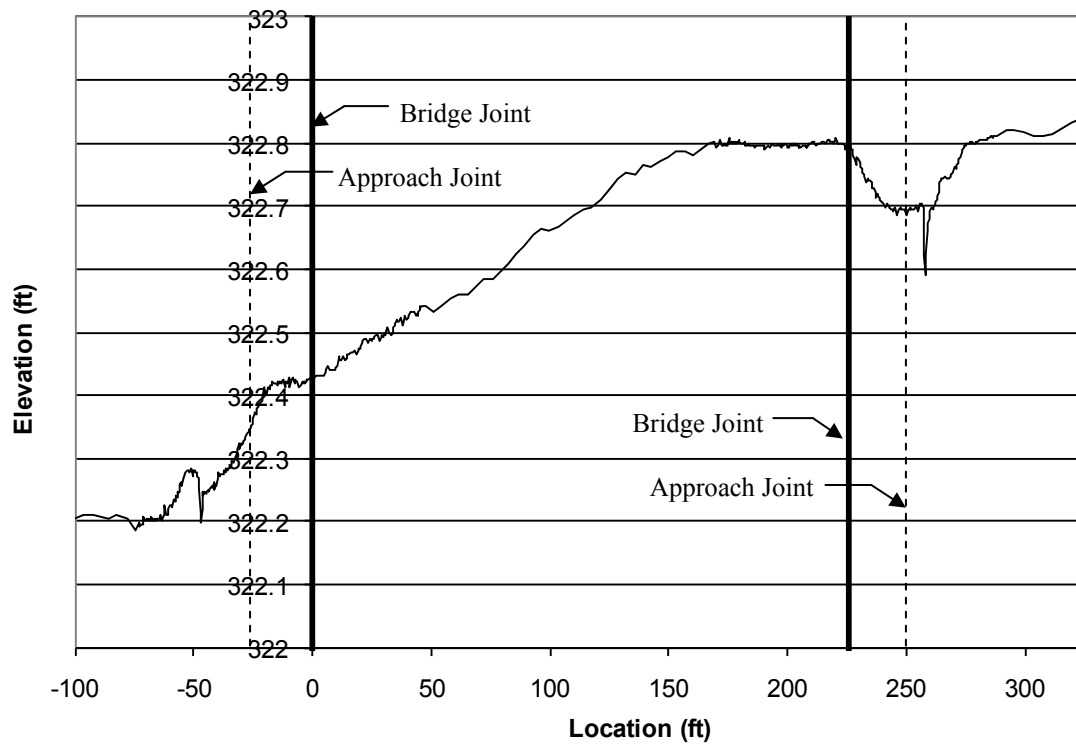
Four types of “geometric” data for the PRE 70-12.49 northbound bridge were collected and evaluated. Specifically, the global geometry was evaluated using a laser-based survey system, the local roadway condition was evaluated using IRI data collected by ODOT, the support system stiffness was evaluated from FWD test results, and the support system was further evaluated through a review of the overall support system depth.

Figures 5.35a and 5.35b show the results of the laser-based survey. These data show that the surface of the bridge itself is relatively consistent (a fact also shown in the IRI data). However, the regions around the approach slabs show global geometries that change rapidly with numerous dips and humps evident. Note the consistency in the measurements from the left and right wheel lines is relatively good.

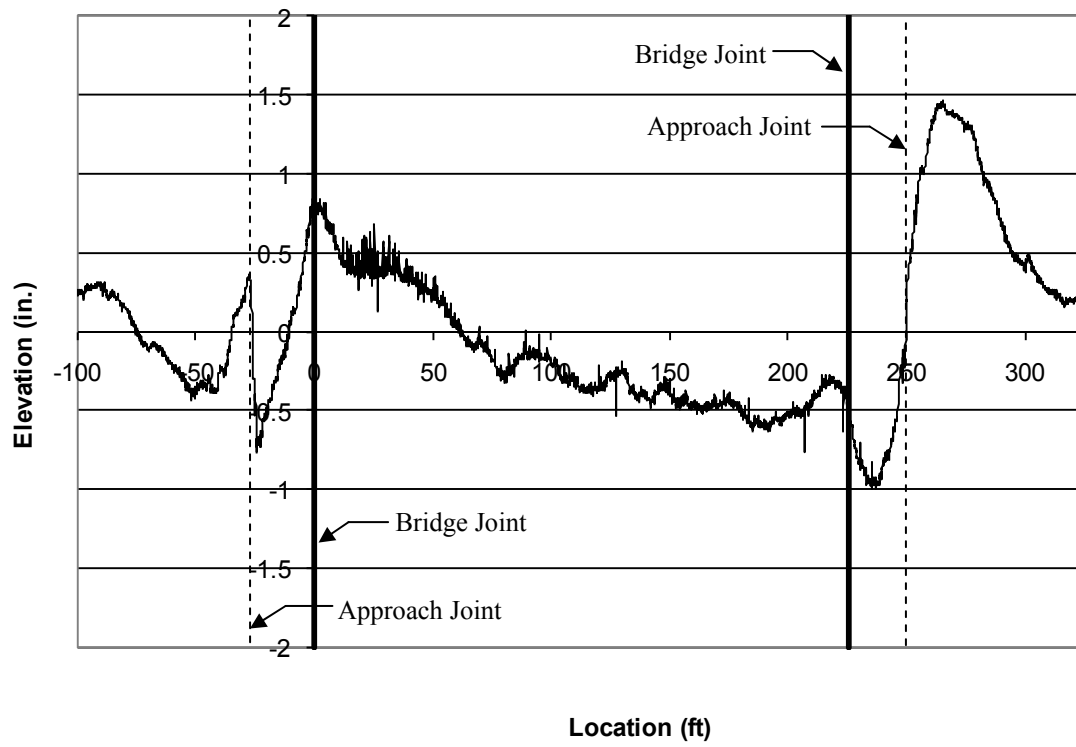
The profiler data (and the IRI data) show, as does the survey data, consistency in the profile of the bridge itself, with reduced consistency off the bridge. In fact, the IRI is more than 900in. per mile in one of these areas. When considering both the survey data and the profiler data, it seems as though some type of degradation in behavior/performance of the approach slab-to-bridge and/or approach slab-to-pavement connection may be contributing to the poor ride quality.



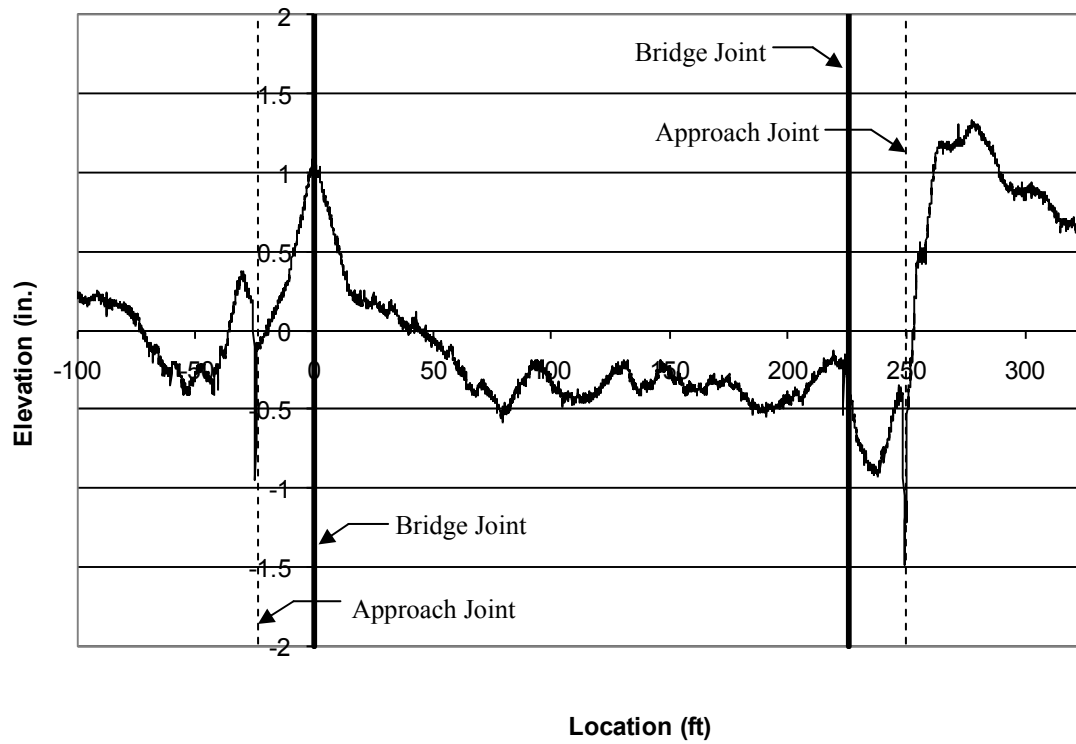
a. Absolute survey data: right wheel line



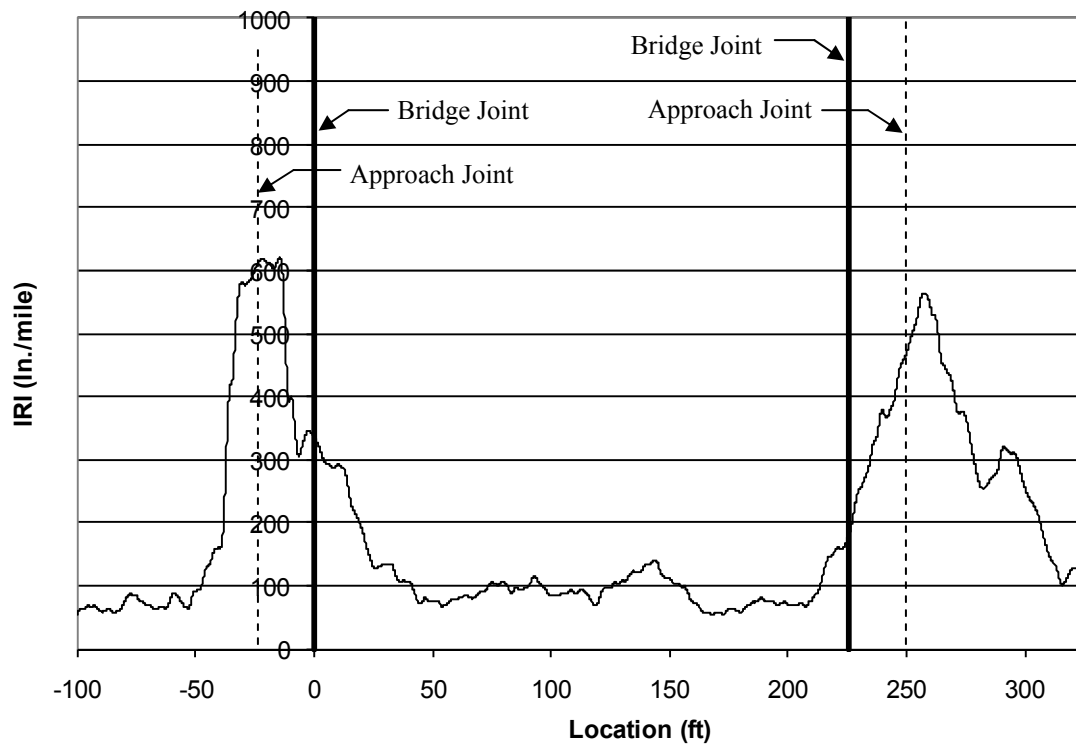
b. Absolute survey data: left wheel line



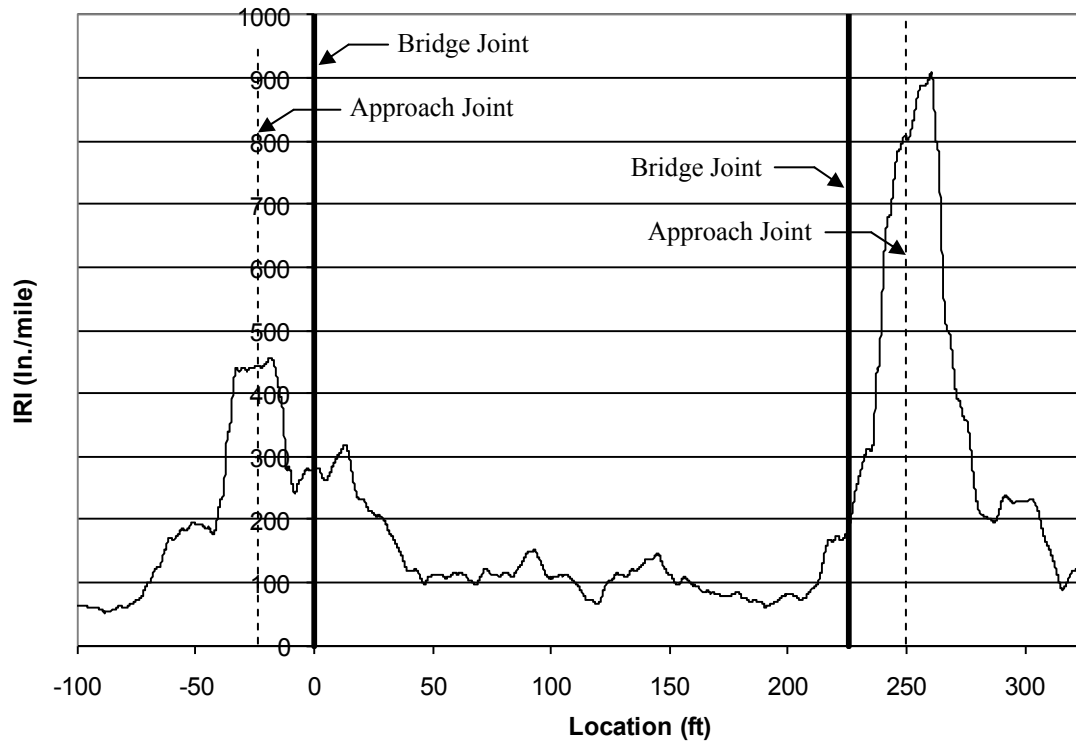
c. Raw profiler data: right wheel line



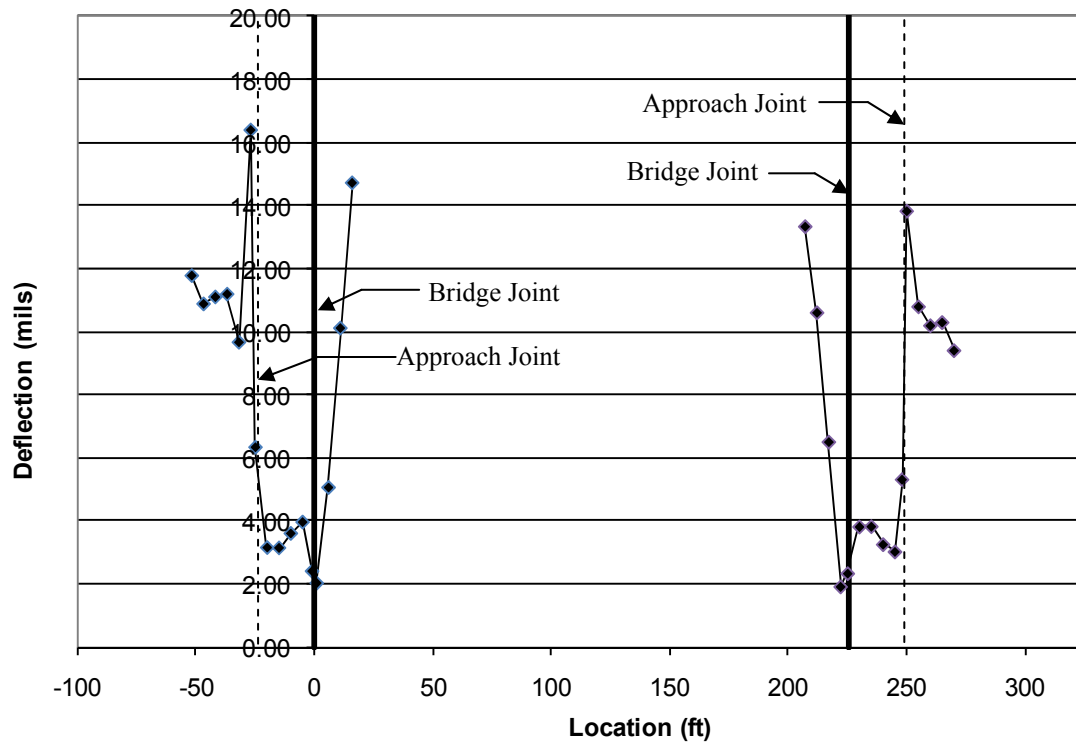
d. Raw profiler data: left wheel line



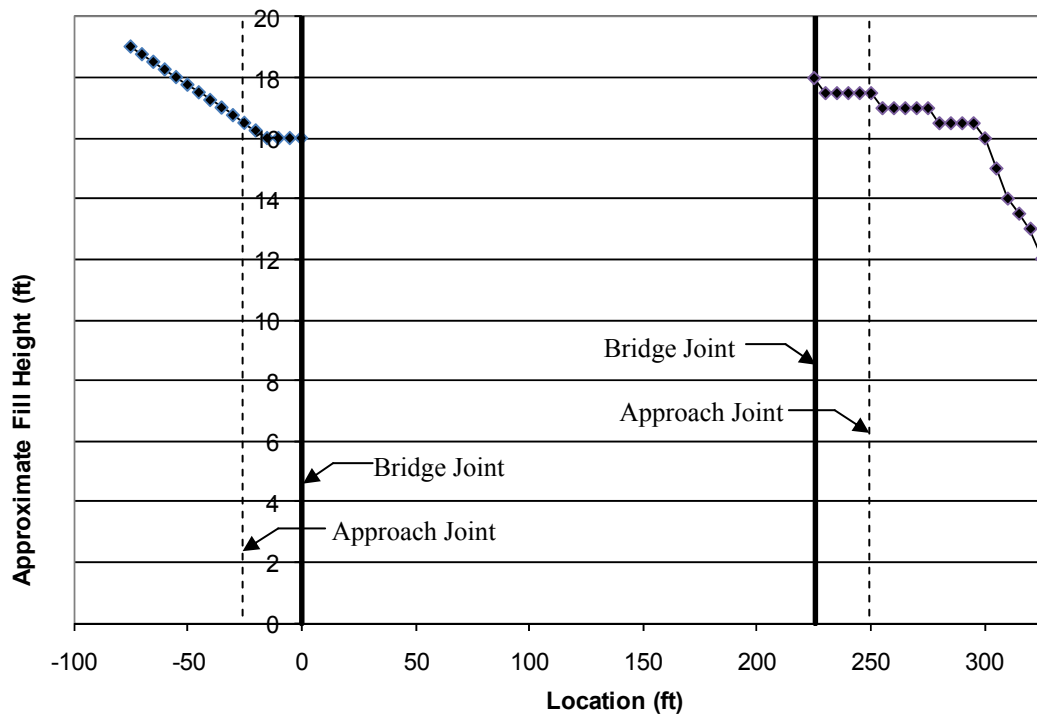
e. IRI: right wheel line



f. IRI: left wheel line



g. FWD test results



h. Approximate fill height

Figure 5.35. PRE 70-12.49 bridge geometric testing results

5.8. LIC 40-12.53

5.8.1. LIC 40-12.53 Bridge Description

The LIC 40-12.53 bridge was built in 1994 at an existing bridge site. The bridge carries two lanes of bi-directional traffic and has a turning lane. The bridge crosses South Fork Licking River with a three-span 21.5 in. thick continuous concrete slab. The total length of the bridge is 130 ft with a longest span of 49 ft. The bridge has a 10 degree skew with cast-in-place concrete columns and a concrete pile cap at the abutments. The approach slab is 25 ft long and bears on a 9 in. paving notch. The slab is attached to the pile cap with #8 diagonal bars. The bridge is shown in Figure 5.36. The joint at the pavement-to-approach interface did not allow for expansion, as shown in Figure 5.37.



Figure 5.36. LIC 40-12.53 profile view

5.8.2. LIC 40-12.53 Visual Bridge Evaluation

Due to the relative newness of the bridge and pavement, the rideability of the bridge was good at the time of the site investigation. Although the approach-to-pavement joint did not allow for expansion, the joint was in good condition and was only starting to show signs of deterioration as shown in Figure 5.37. No signs of the approach slabs settling or loss of fill was evident. No oil staining was present on or near the bridge as seen in Figure 5.38.



Figure 5.37. Pavement-to-approach joint condition at LIC 40-12.53



Figure 5.38. Condition of LIC 40-12.53 bridge surface

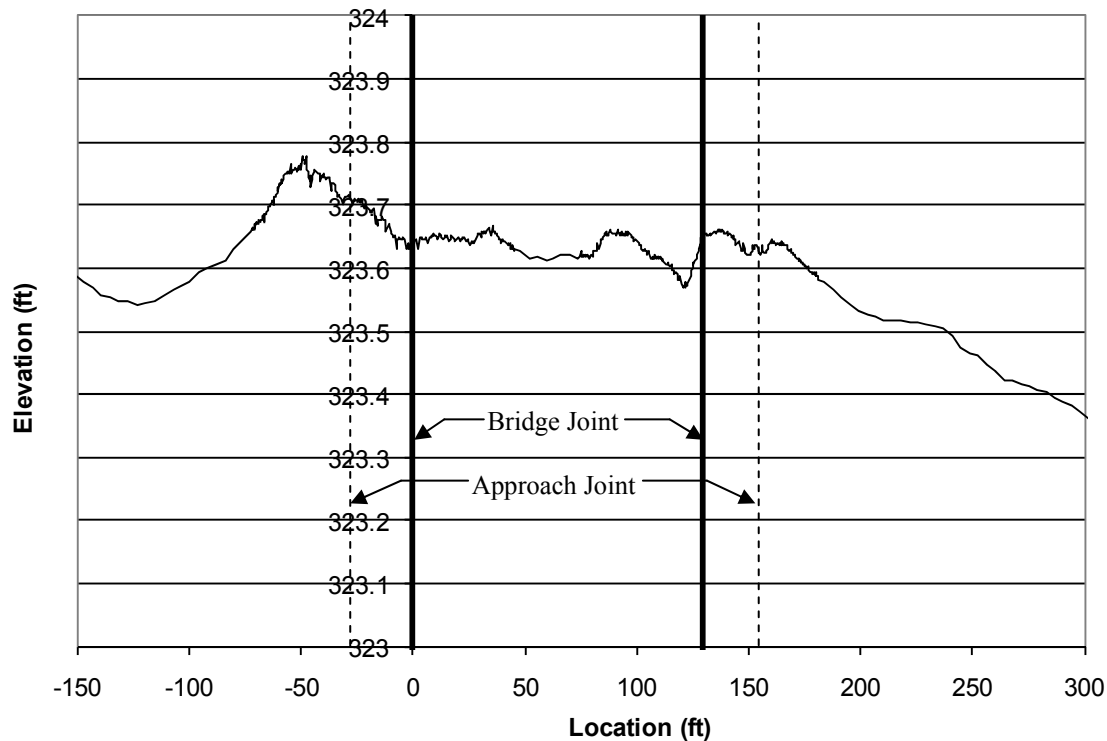
5.8.3. LIC 40-12.53 Geometric Bridge Testing and Support System Evaluation

Four types of “geometric” data for the LIC 40-12.53 bridge were collected and evaluated. Specifically, the global geometry was evaluated using a laser-based survey system, the local roadway condition was evaluated using IRI data collected by ODOT, the support system stiffness was evaluated from FWD test results, and the support system was further evaluated through a review of the overall support system depth.

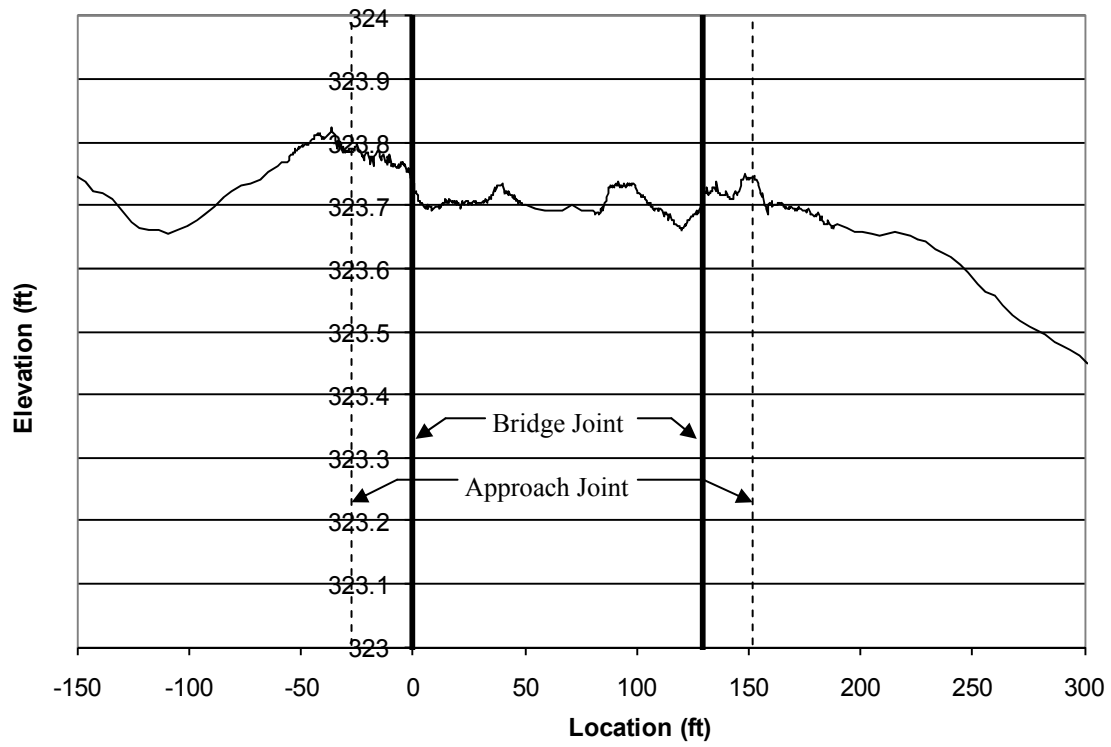
Figures 5.39a and 5.39b show the results of the laser-based survey. This bridge shows a consistent geometry from left to right. It further appears that the bridge consists of three distinct “regions,” both physically and in terms of data consistency. The entrance end of the bridge shows some variability that changes gradually. The bridge itself shows less overall variability in magnitude but that the variability changes quickly. The exit end of the bridge shows some instances of rapid, marked changes, indicating the potential for ride quality degradation.

In general, the above-mentioned observations are reified in the raw profiler and IRI data. Of greatest concern is the high spike in IRI data at the exit end on an otherwise acceptable bridge.

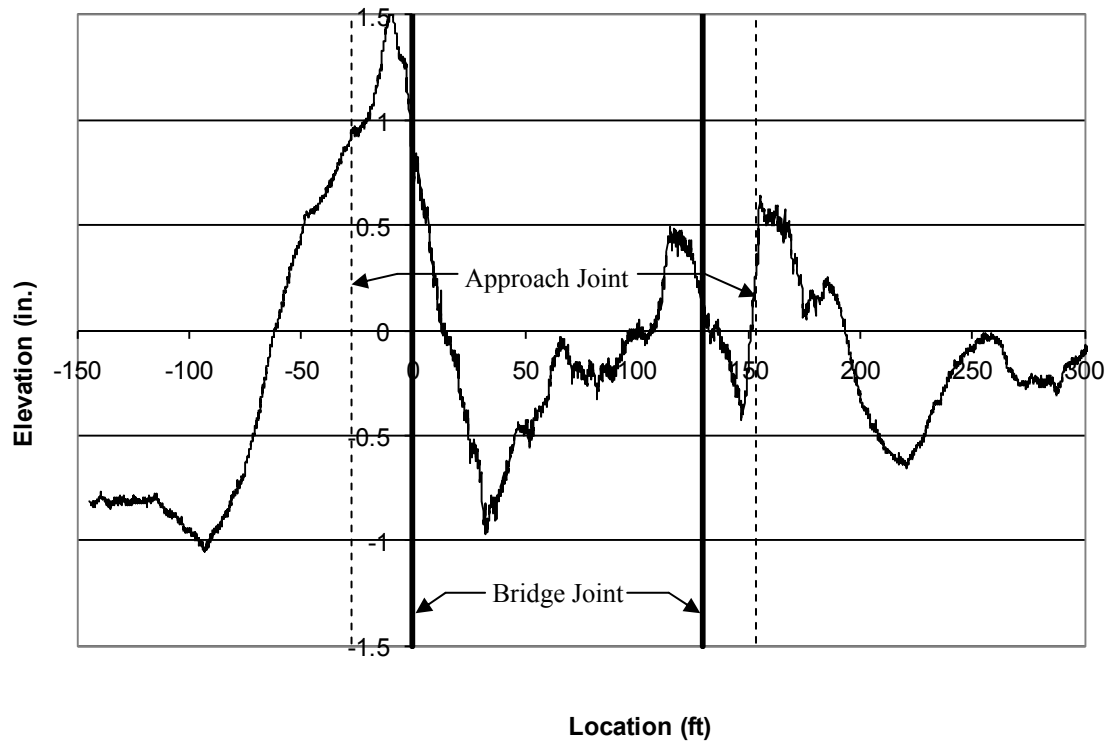
The FWD test results do not offer an explanation to the above-mentioned features. Specifically, the lowest stiffness is at the end of the bridge with the best ride quality. This may indicate approach slab connection problems.



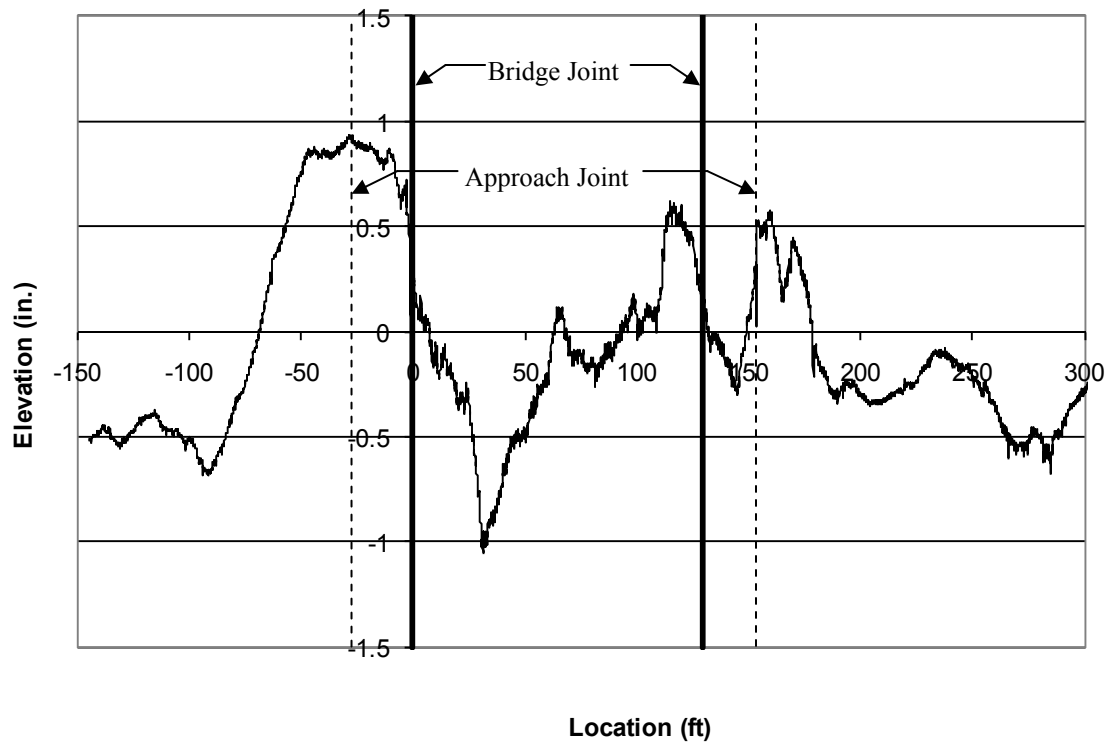
a. Absolute survey data: right wheel line



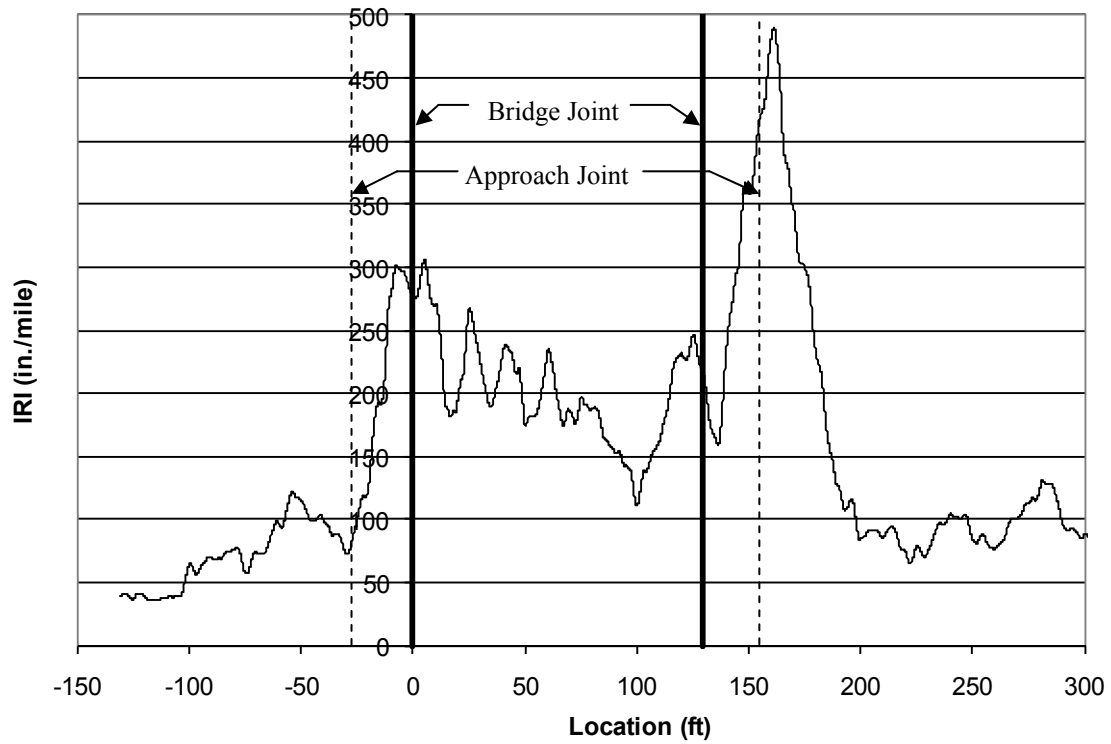
b. Absolute survey data: left wheel line



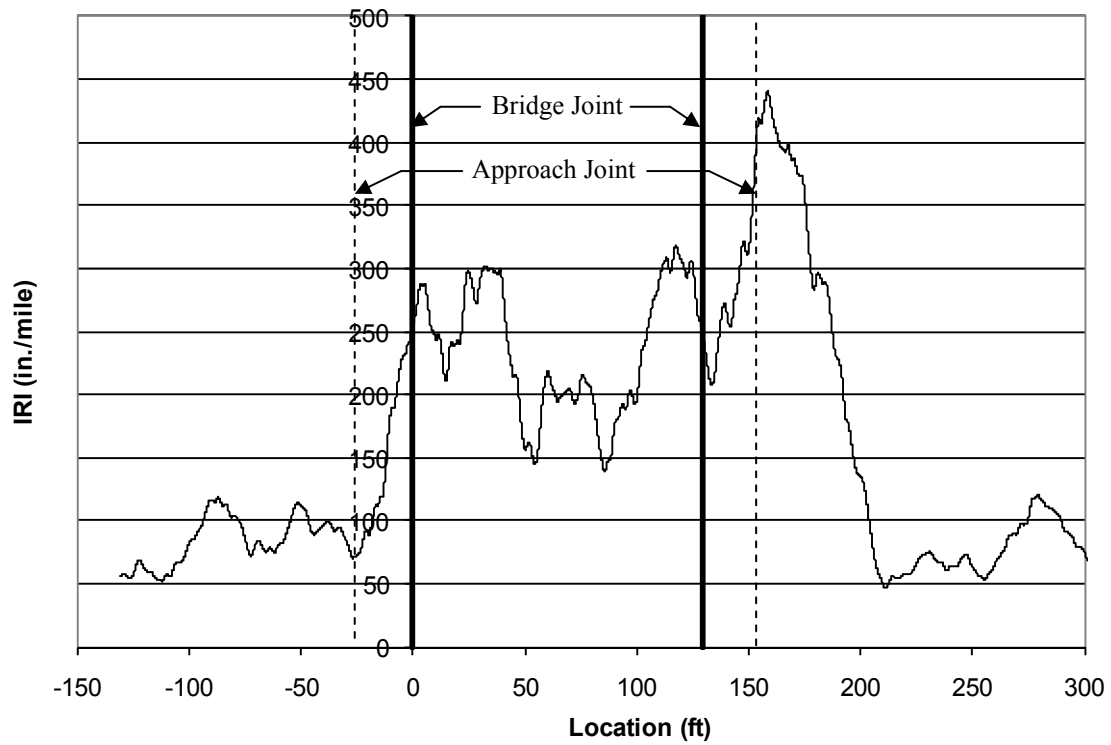
c. Raw profiler data: right wheel line



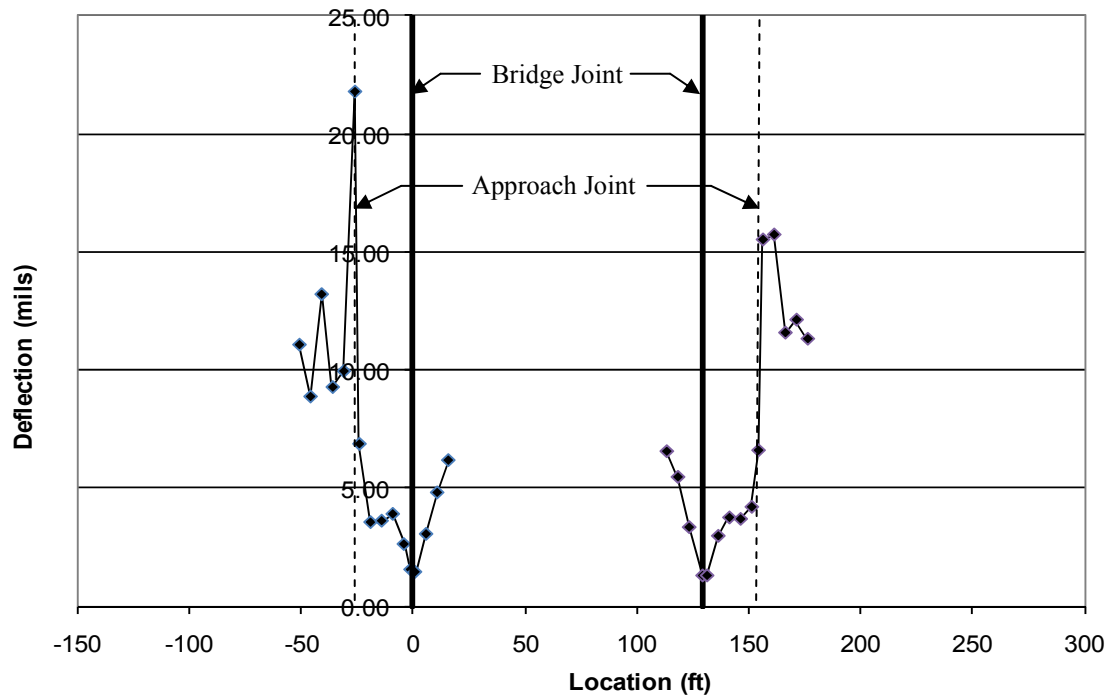
d. Raw profiler data: left wheel line



e. IRI: right wheel line



f. IRI: left wheel line



g. FWD test results

Figure 5.39. LIC 40-12.53 bridge geometric testing results

5.9. WYA 30-22.40

5.9.1. WYA 30-22.40 Bridge Description

The WYA 30-22.40 bridge, shown in Figure 5.40, was built circa 2000 over Broken Sword Creek. The bridge has a 20 degree skew and is two lanes wide carrying eastbound traffic. The bridge is three spans with a total length of 233 ft. Six 54 in. deep pre-stressed concrete I-beams, with a composite concrete deck, make up the three spans. The abutments consist of one row of H-piles oriented with their strong axis parallel to the direction of travel. The piles have a concrete cap that connects to the girders to create an ODOT integral abutment. The approach slabs are 30 ft long and rest on a 6 in. paving notch. The approach slab is connected to the abutment with diagonal # 8 bars. The joint between the concrete pavement and approach slab has a 50 in. asphalt pressure relief joint.



Figure 5.40. WYA 30-22.40 profile view

5.9.2. WYA 30-22.40 Visual Bridge Evaluation

The overall visual quality of the bridge rideability was good; however, the asphalt pressure relief joint was showing signs of deterioration and has been patched in some locations, as shown in Figure 5.41. The concrete pavement and approach slab on each side of the joint were in good condition. The approach slabs did not show signs of settlement. Good drainage of surface water was provided, as shown in Figure 5.42, which helps prevent erosion of fill material.



Figure 5.41. WYA 30-22.40 pressure relief joint deterioration



Figure 5.42. Bar grates and curb provide good water drainage and eliminates erosion at the edge of the shoulder

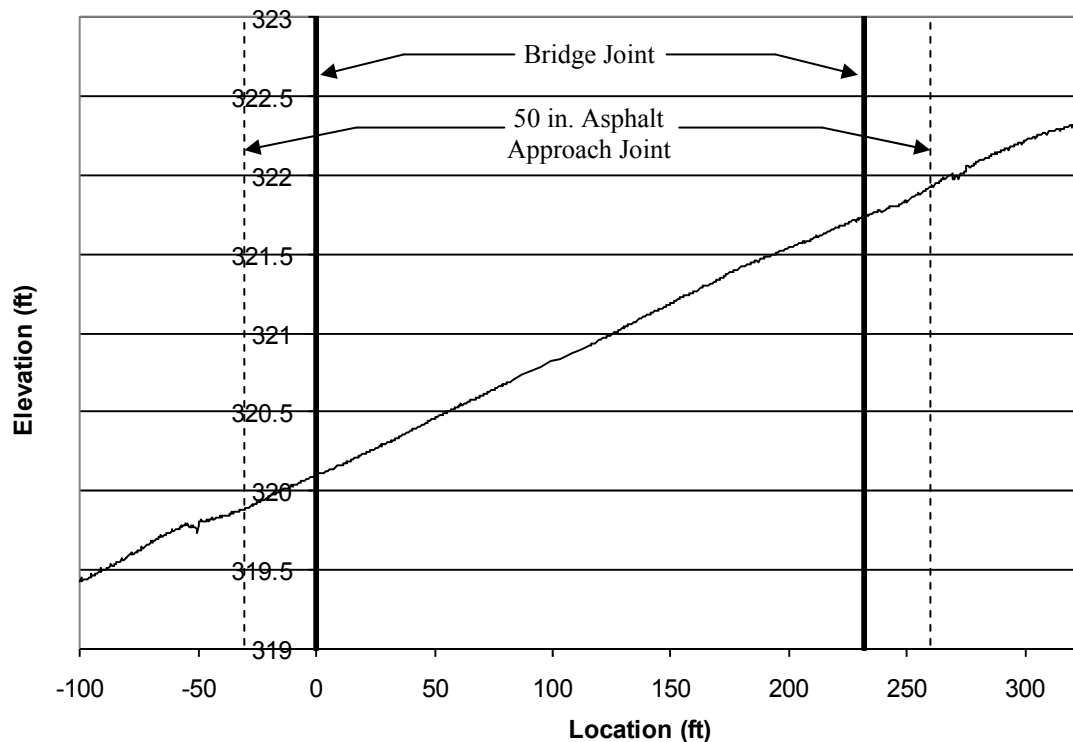
5.9.3 WYA 30-22.40 Geometric Bridge Testing and Support System Evaluation

Four types of “geometric” data for the WYA 30-22.40 eastbound bridge were collected and evaluated. Specifically, the global geometry was evaluated using a laser-based survey system, the local roadway condition was evaluated using IRI data collected by ODOT, the support system stiffness was evaluated from FWD test results, and the support system was further evaluated through a review of the overall support system depth.

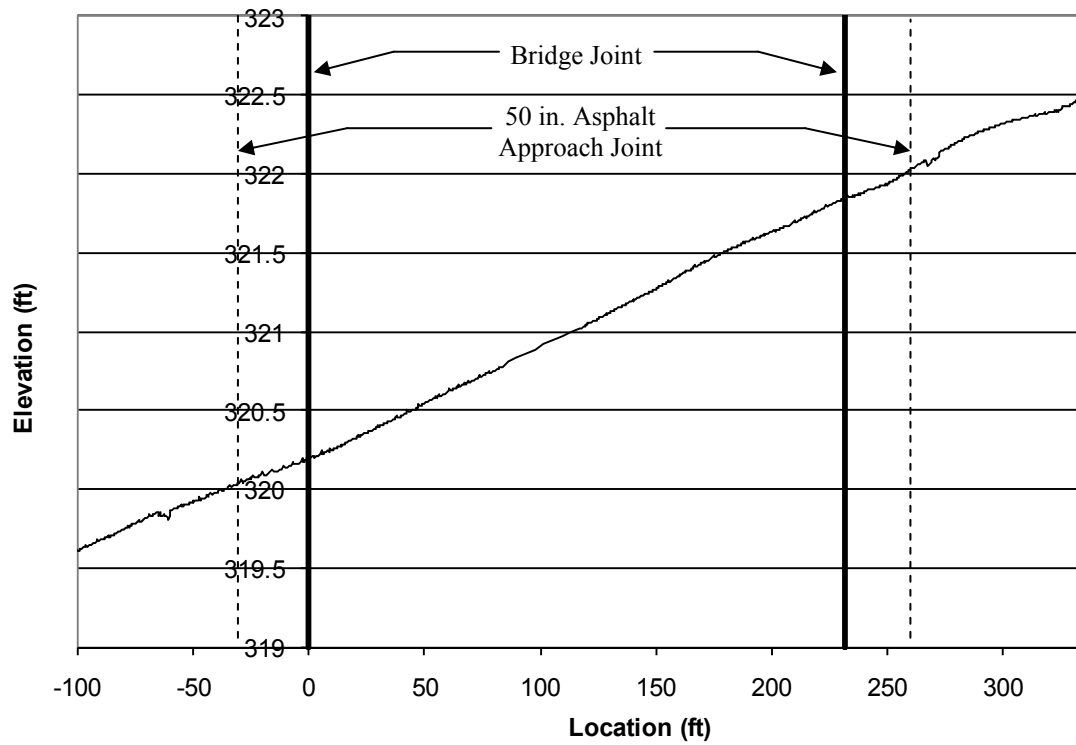
Figures 5.43a and 5.43b show the results of the laser-based survey. With only minor exceptions at both the entrance and exits of the bridge the global survey results indicate a bridge with few discontinuities. The discontinuities at both ends and in both wheel lines likely coincide with the approach slab-to-pavement joint. It would appear, then, that these areas would be the only locations where rideability may be less than desired.

The profiler data (both raw and IRI) further confirm the observations made with respect to the survey data. With the exceptions of the area in the vicinity of the approach slab-to-pavement connection, this appears to be a bridge with very good rideability characteristics.

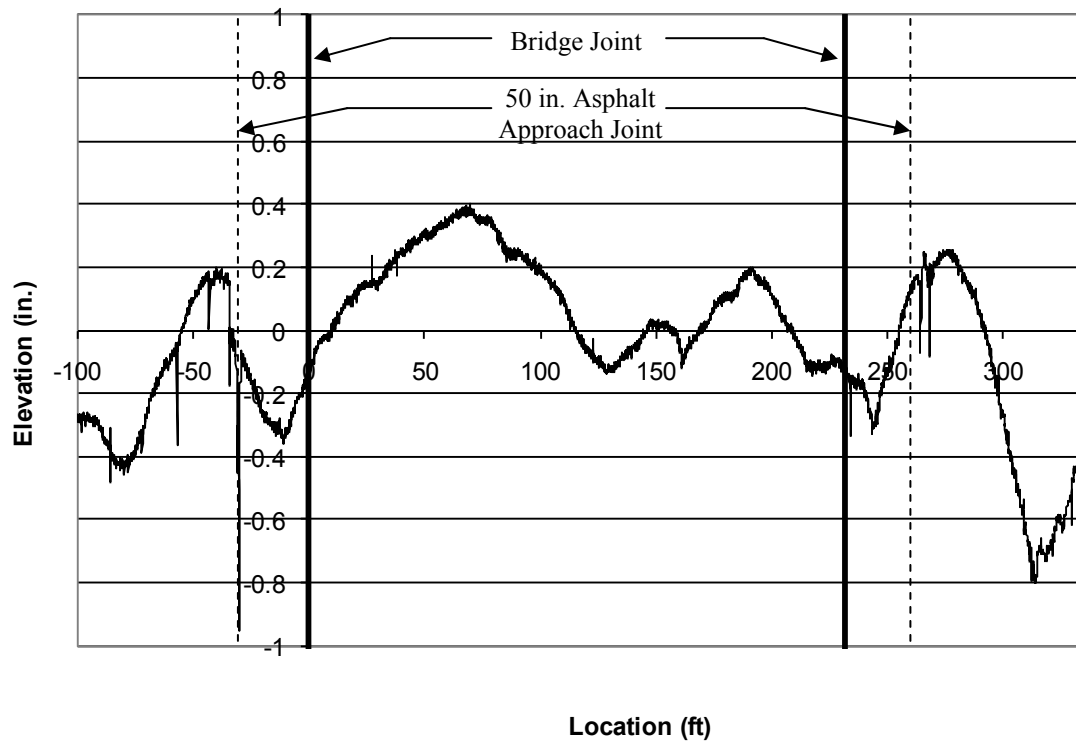
Unlike some of the previously-discussed FWD results, the WYA 30-22.40 bridge shows greater consistency in support system stiffness. Like the previously-discussed bridges, the greatest stiffness is at the abutment (as expected). However, the change in system stiffness with this bridge was less than that observed for other bridges.



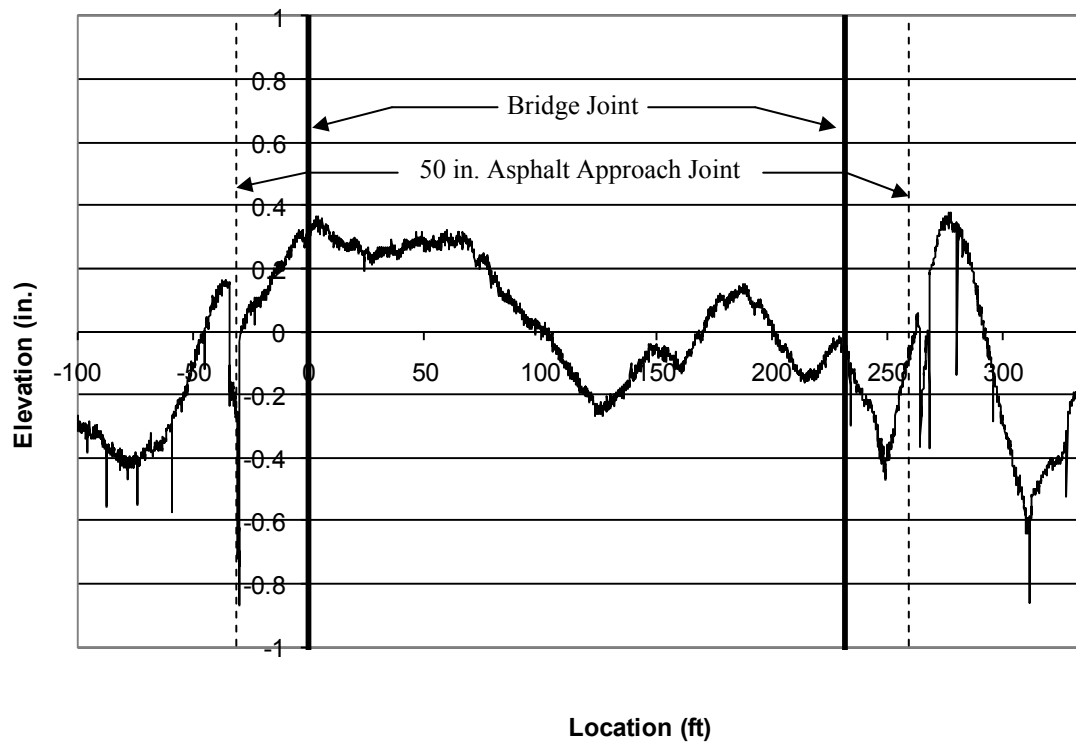
a. Absolute survey data: right wheel line



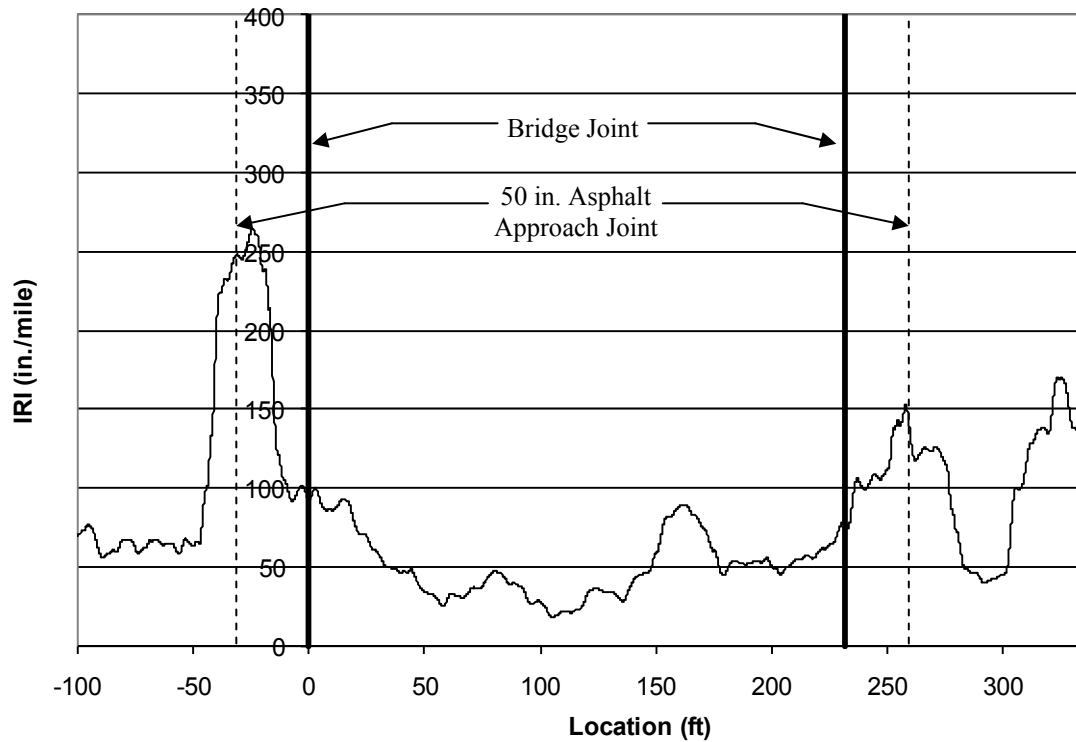
b. Absolute survey data: left wheel line



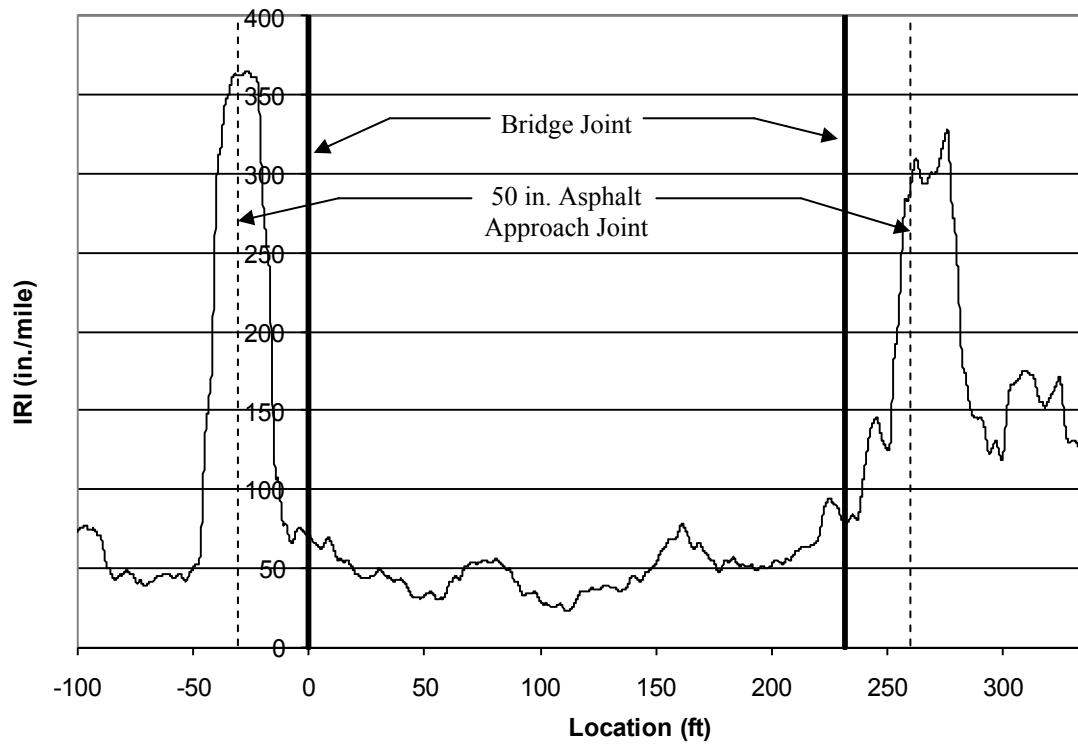
c. Raw profiler data: right wheel line



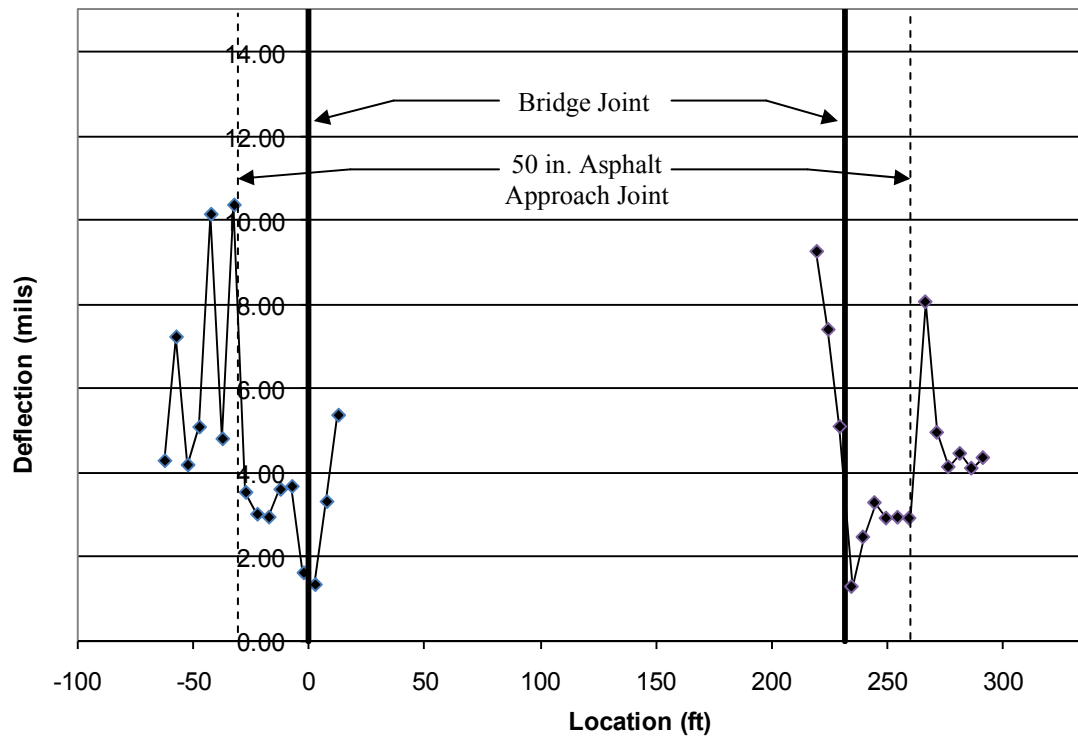
d. Raw profiler data: left wheel line



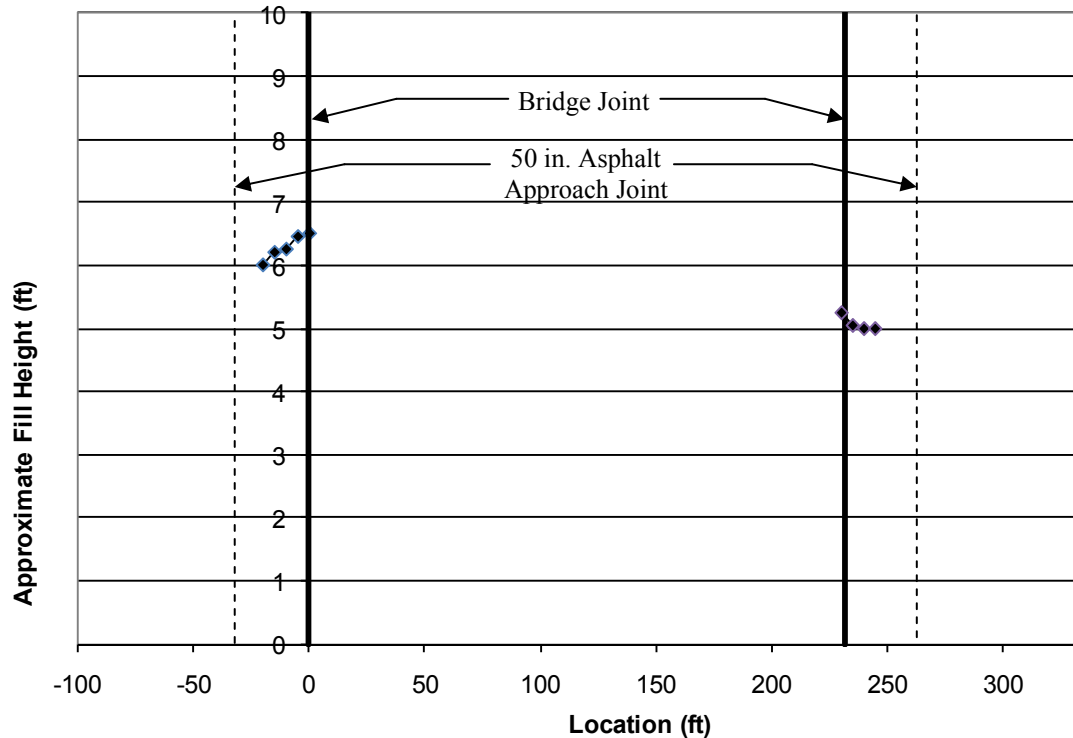
e. IRI: right wheel line



f. IRI: left wheel line



g. FWD test results



h. Approximate fill height

Figure 5.43. WYA 30-22.40 bridge geometric testing results

5.10. FRA 270-32.36

5.10.1. FRA 270-32.36 Bridge Description

The FRA 270-32.36 bridge was built circa 1995 and carries three southbound lanes of traffic over an on ramp to Interstate 270. The bridge is a 304 ft long two-span bridge with the longest span being 173 ft, as shown in Figure 5.44. The bridge has seven 84 in. deep continuous steel plate girders with composite concrete decking. The abutments are semi-integral, which bear pile caps of two rows of vertical steel H-piles with their weak axis oriented parallel to the travel direction. Each abutment has 20 ft of MSE wall retaining the bridge embankments. The bridge, during the time of testing, had 30 ft concrete approach slabs. The highway pavement is concrete; therefore, a 12 in. pressure relief joint was placed at the end of each approach slab. In the past this bridge had experienced a severe erosion problem at the entrance abutment. Specifically, water began infiltrating the MSE wall, creating a large void under the approach slab. Subsequently, the void was filled with grout.



Figure 5.44. FRA 270-32.36 profile view

5.10.2. FRA 270-32.36 Visual Bridge Evaluation

At the time of the field investigation, the bridge had excellent overall ridge quality; however, the west and center lane pavement and approach slabs had been replaced and/or ground within the previous year. The pressure relief joint was in good condition, as shown in Figure 5.45; however, it did cause a slight bump for traversing cars. Figure 5.46 shows soil deposits on top of the rock slope protection.



Figure 5.45. 12 in. pressure relief joint condition



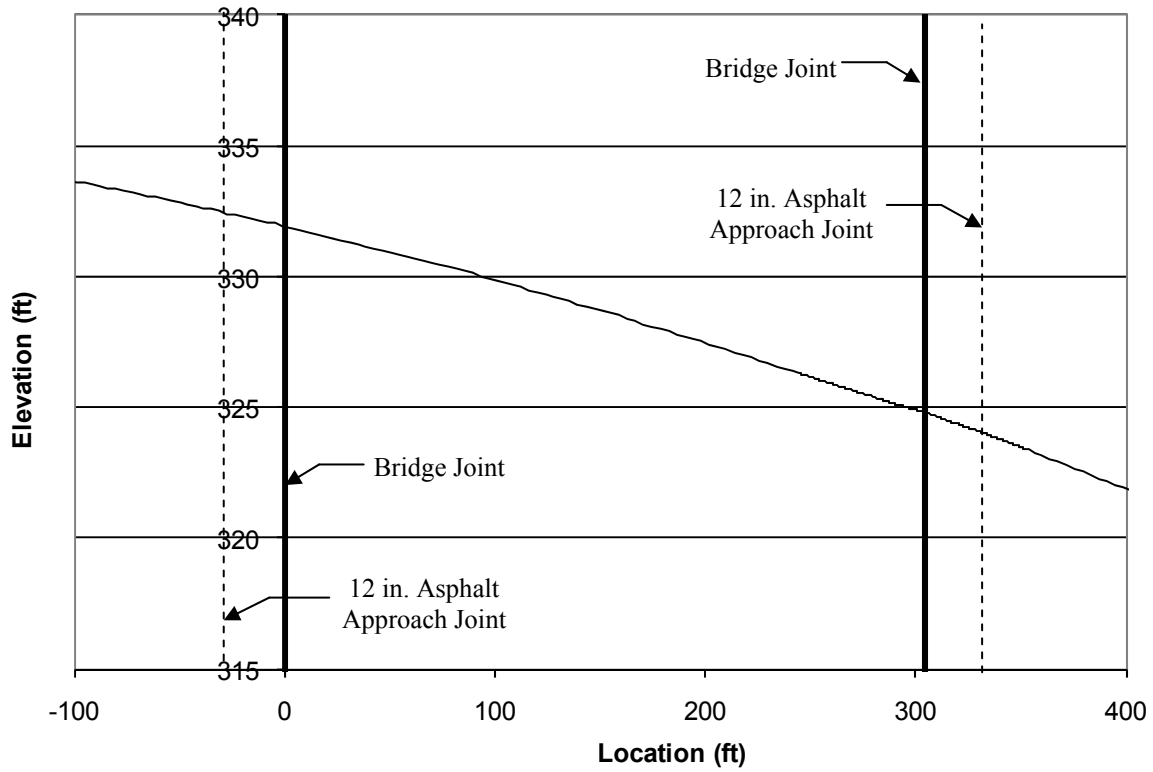
Figure 5.46. Erosion from bridge embankment

5.10.3. FRA 270-32.36 Geometric Bridge Testing and Support System Evaluation

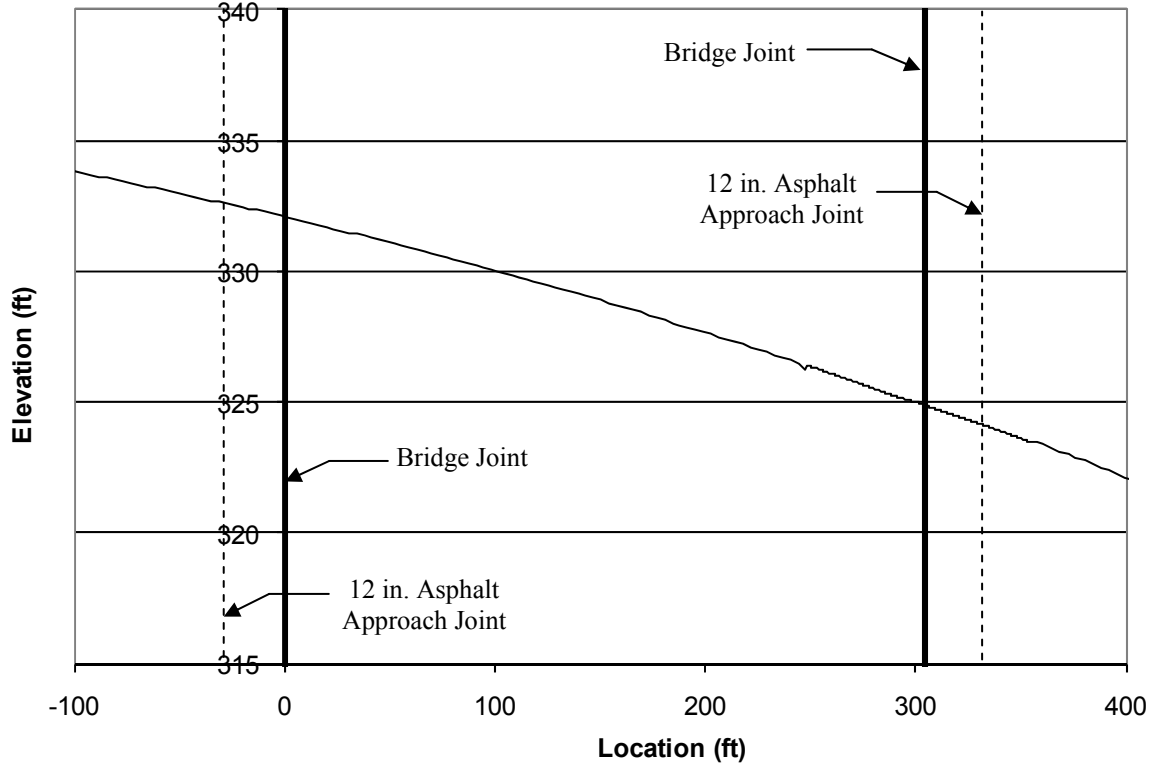
Four types of “geometric” data for the FRA 270-32.36 bridge were collected and evaluated. Specifically, the global geometry was evaluated using a laser-based survey system, the local roadway condition was evaluated using IRI data collected by ODOT, the support system stiffness was evaluated from FWD test results, and the support system was further evaluated through a review of the overall support system depth.

Figures 5.47a and 5.47b show the results of the laser-based survey. Generally, only one area showed a slight discontinuity in the survey results—approximately 75 ft from the bridge exit. The profiler data seems to substantiate the survey results, indicating this area has high IRI values (approaching 450). However, overall, the survey, raw profiler, and IRI data indicate a bridge with generally good rideability characteristics.

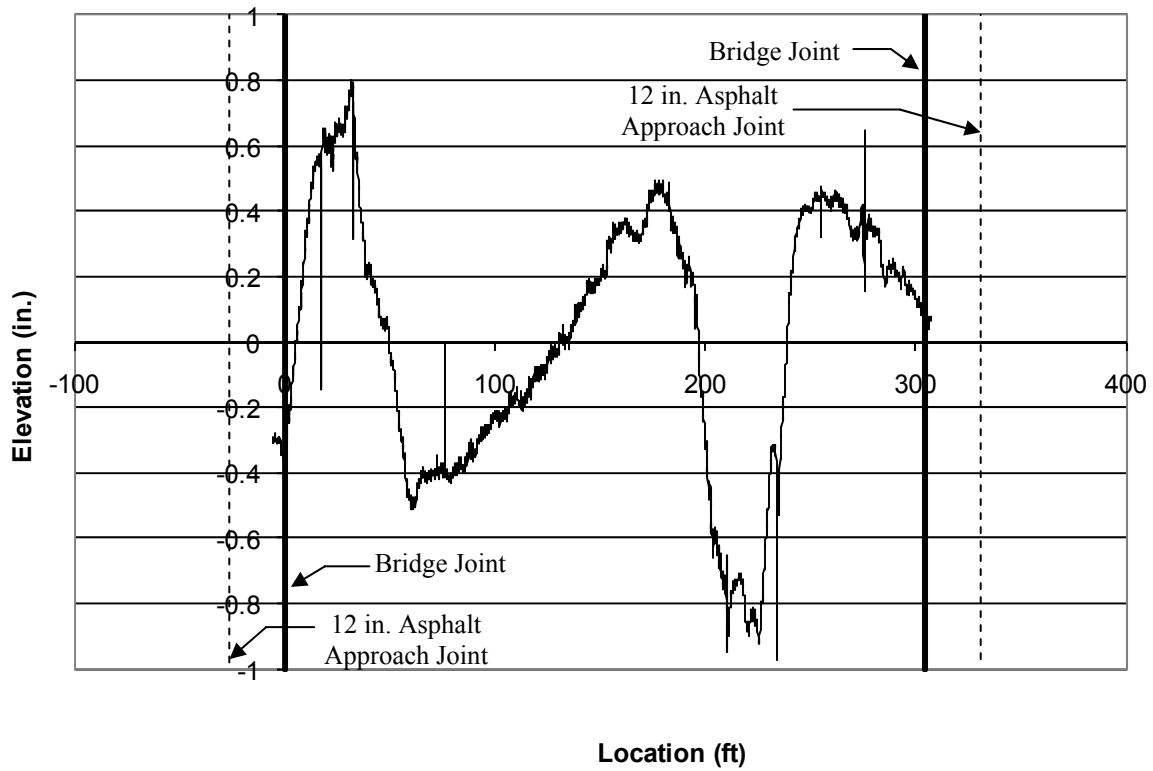
With only one exception, the FWD results indicate a support system that has very consistent stiffness characteristics. It is possible that this stiffness consistency contributed to the smoothness characteristics.



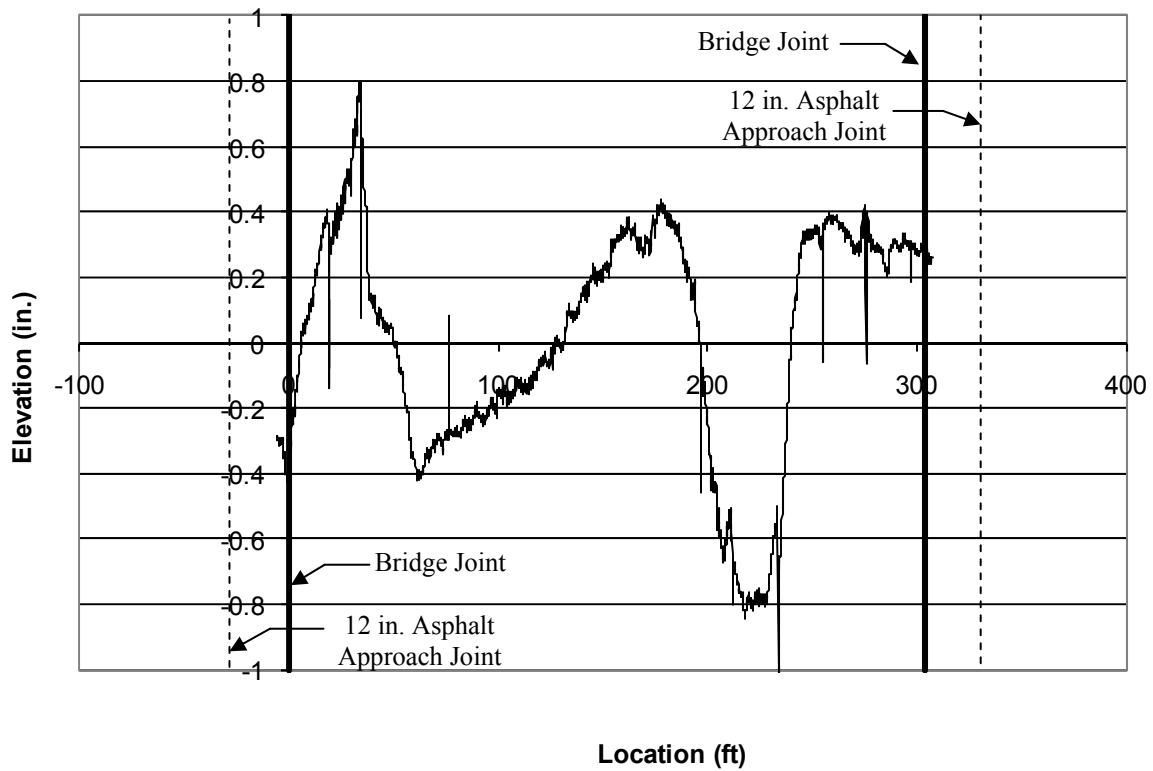
a. Absolute survey data: right wheel line



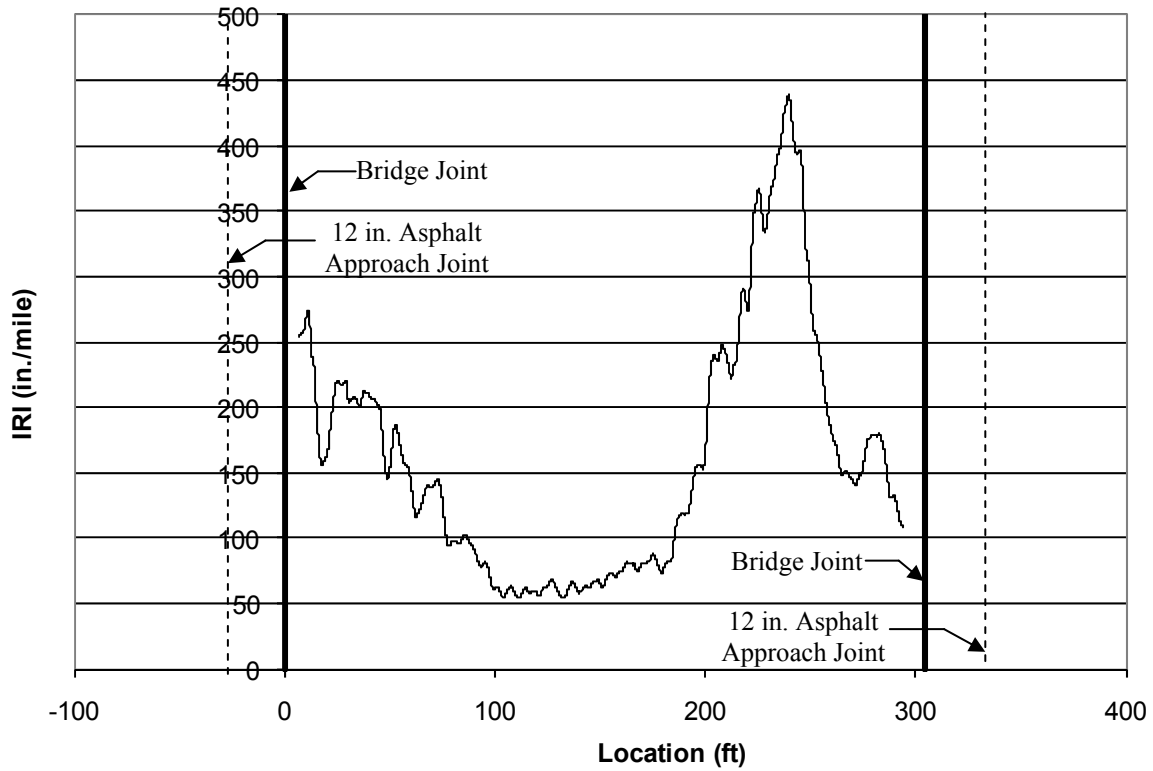
b. Absolute survey data: left wheel line



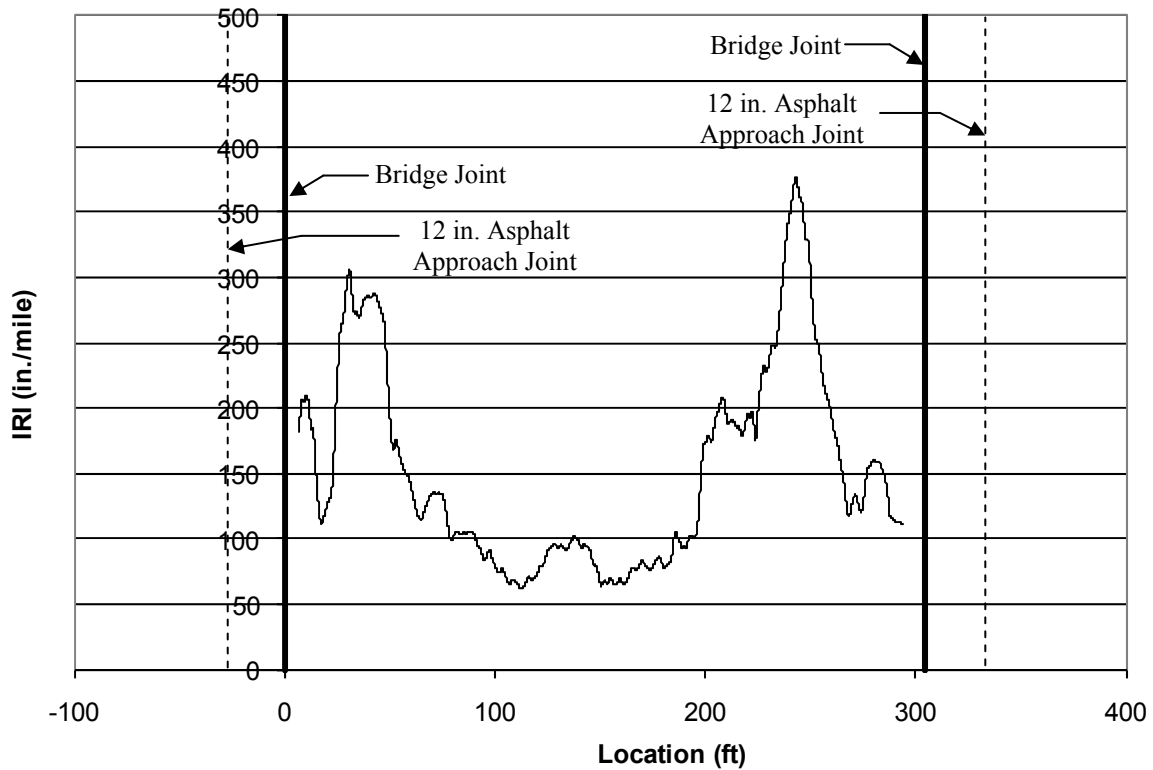
c. Raw profiler data: right wheel line



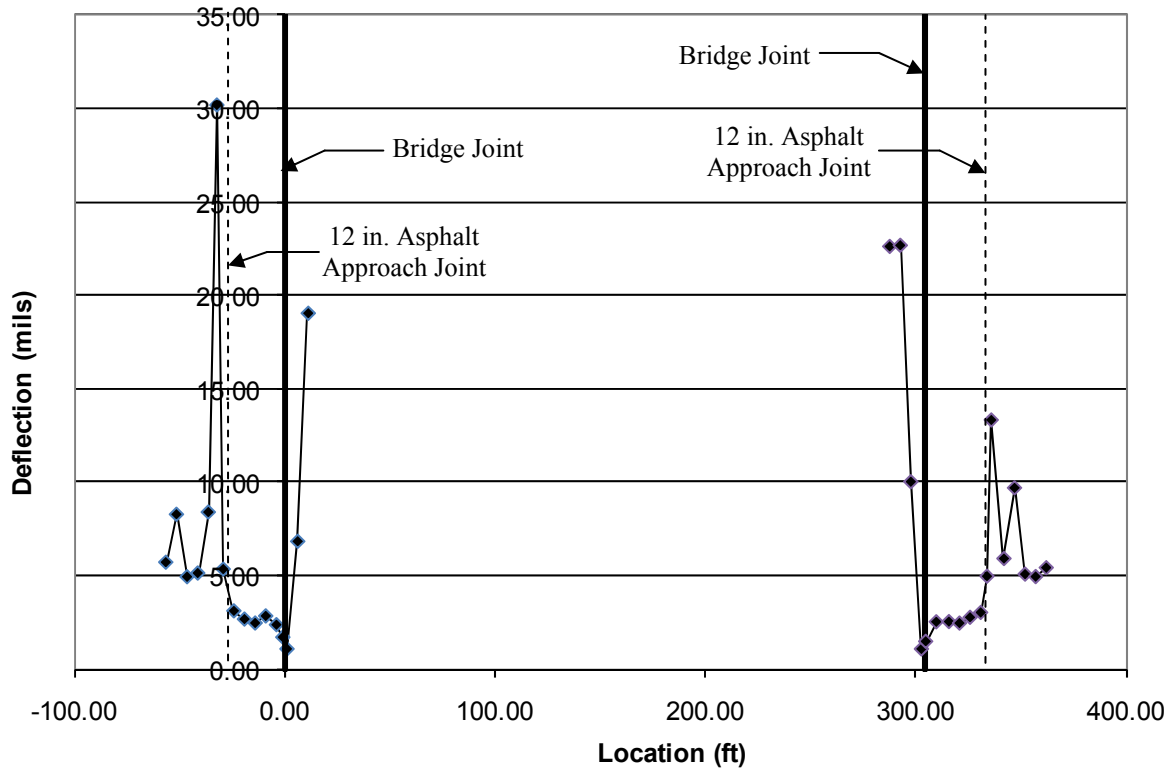
d. Raw profiler data: left wheel line



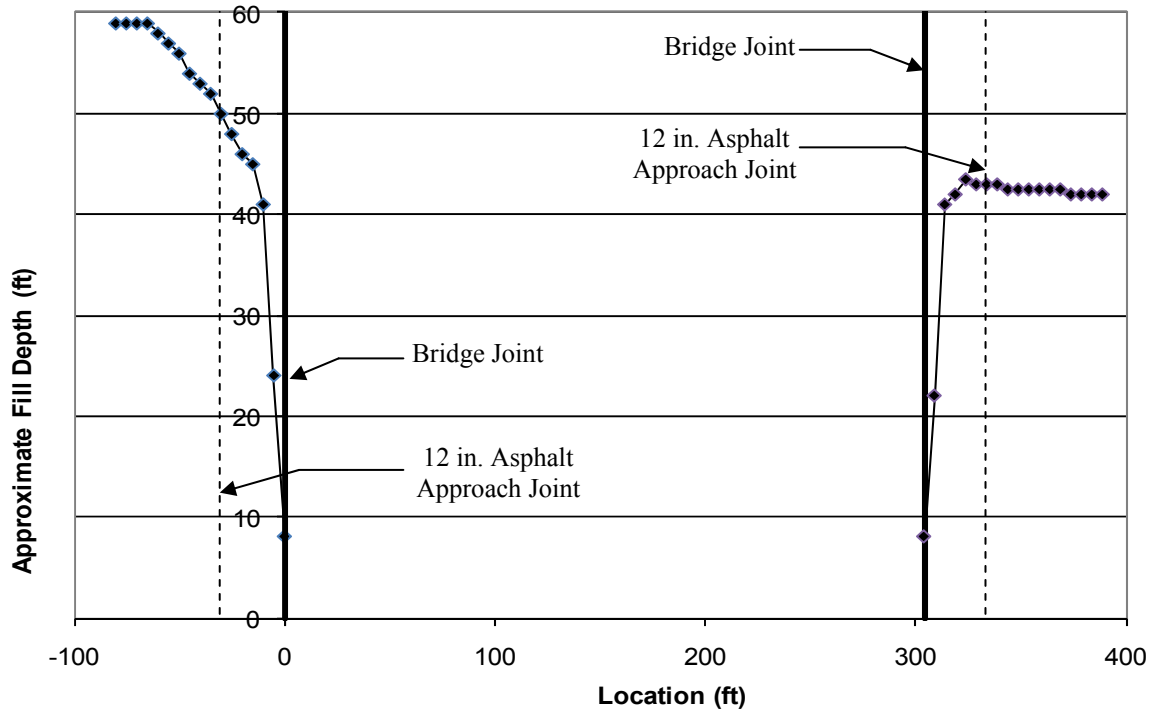
e. IRI: right wheel line



f. IRI: left wheel line



g. FWD test results



h. Approximate fill height

Figure 5.47. FRA 270-32.36 bridge geometric testing results

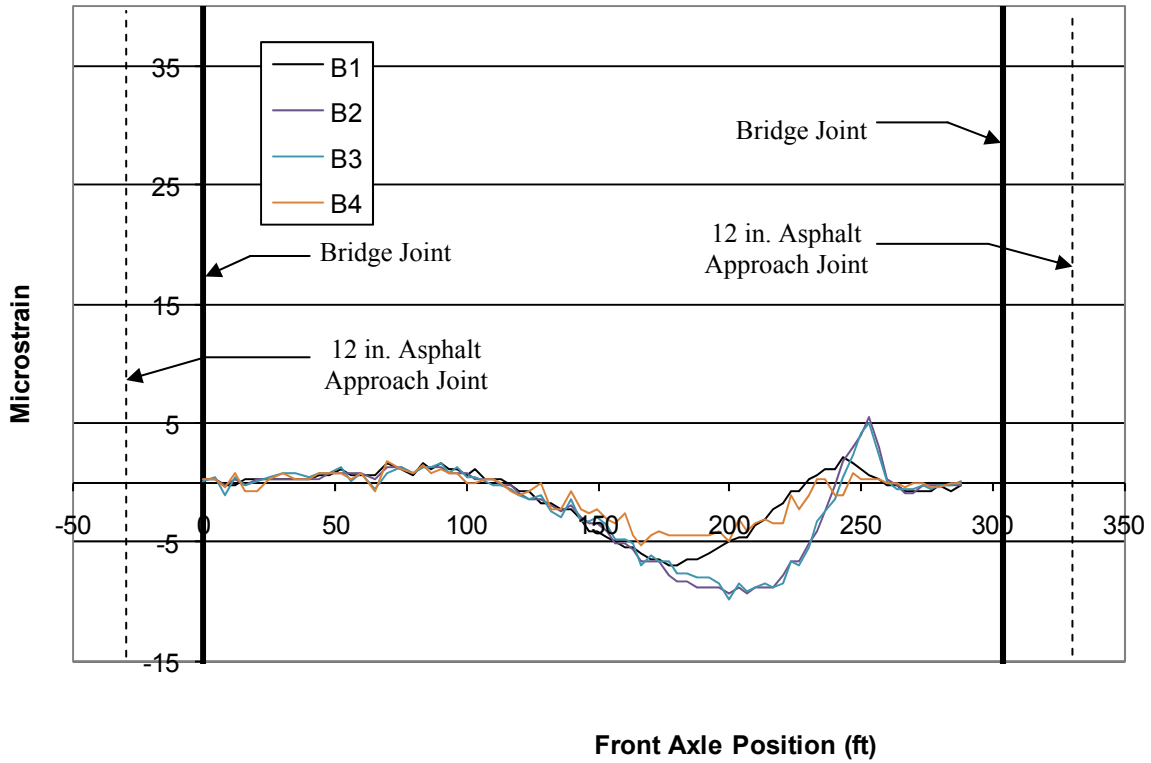
5.10.4. FRA 270-32.36 Live Load Testing

The FRA 270-32.36 bridge was load tested with a loaded legal truck to understand how the bridge behaves under typical live loads. As shown in Figure 5.48, the bridge was instrumented with strain gages on three girders (at two cross sections), with strain gages on the top of the approach slab, with deflection gages measuring relative movement of the approach slab corners, and with abutment rotation and translation sensors.

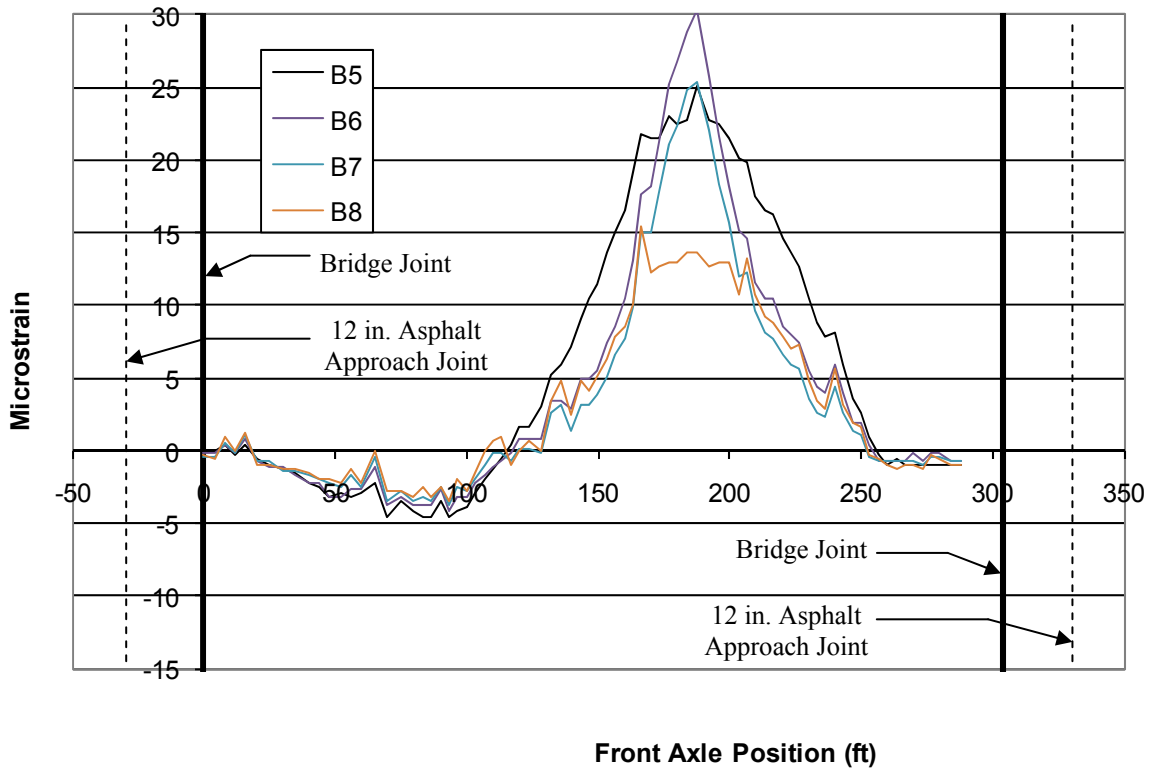
As shown in Figure 5.49, the near girder end strain gages indicated very little end restraint occurring at the bridge bearings. This finding is further observed in the near midspan strain gages. In general, the measured girder strain levels are low with measured stresses under 1 ksi.

As in previously-described testing, very little measured slab strain and relative displacement indicated that the slab is behaving as a “unit.” This type of unified behavior is probably most impacted by the high degree of consistency in the support system stiffness.

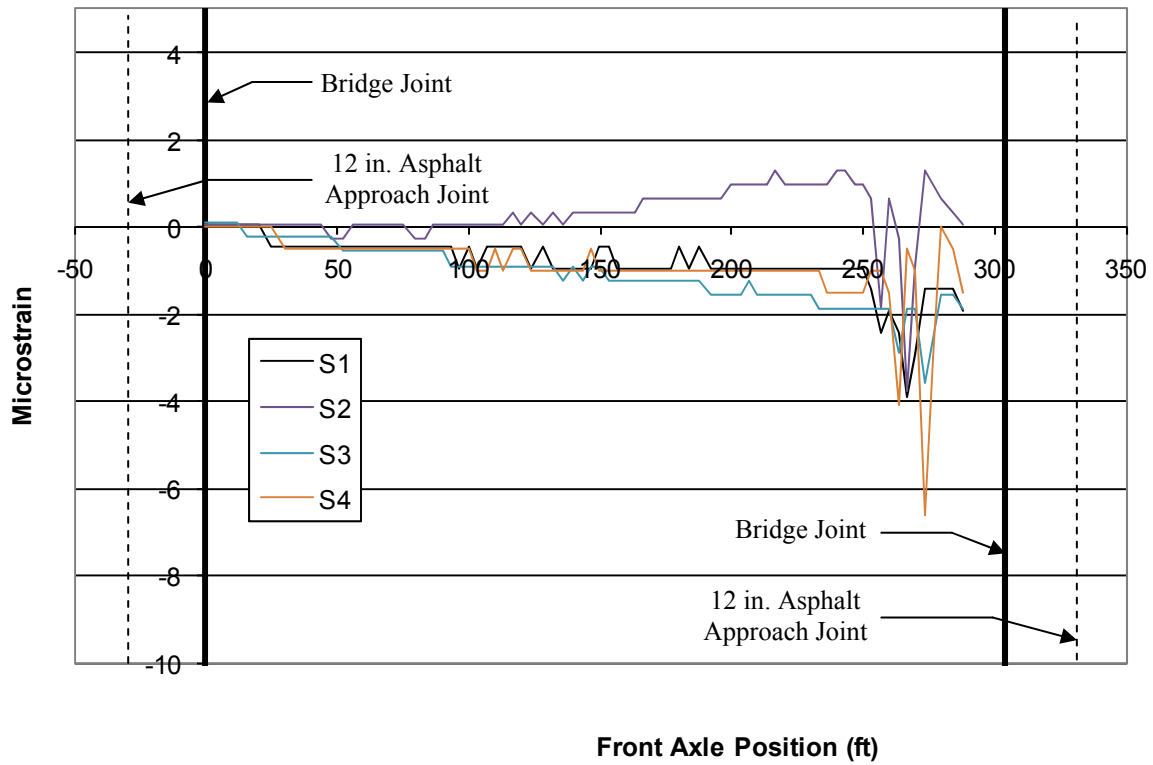
As before, it was found the abutment did not rotate, but that it did have observable live load induced movement. This movement again has signs of strain sign reversal, indicating that the abutment is moving into and out of the supporting soil.



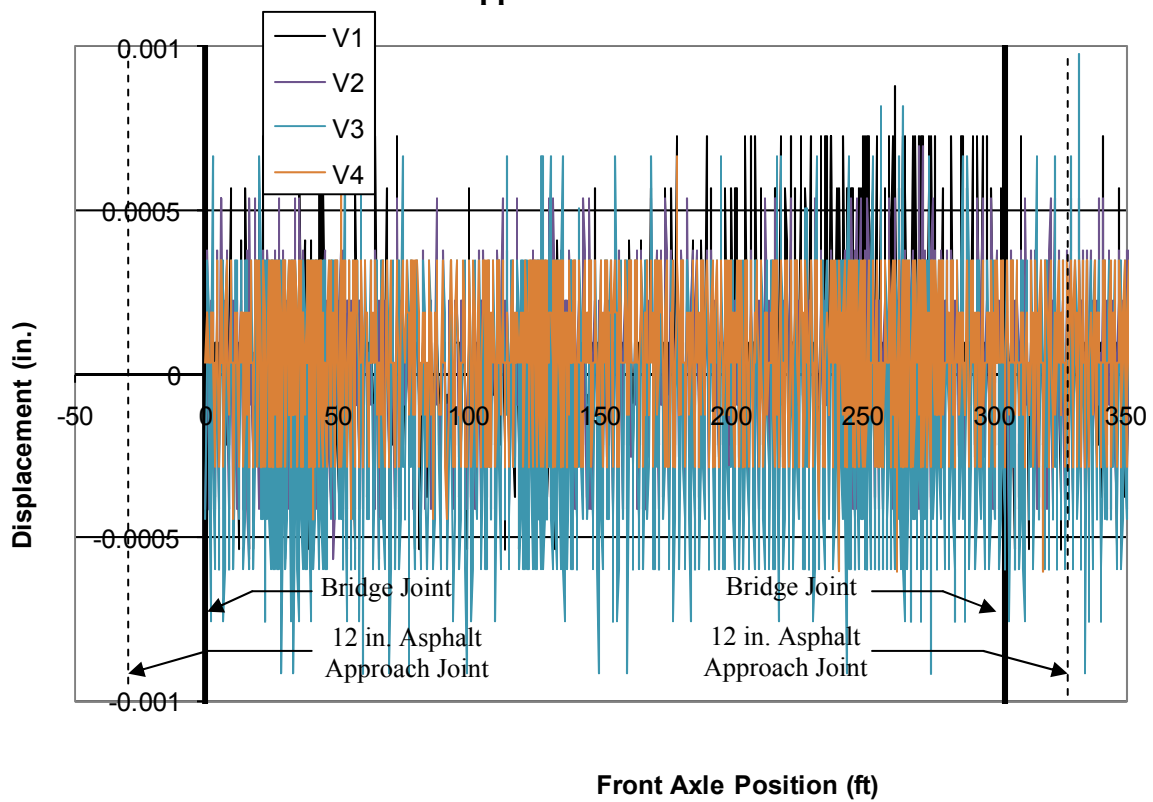
a. Girder strain: near end support



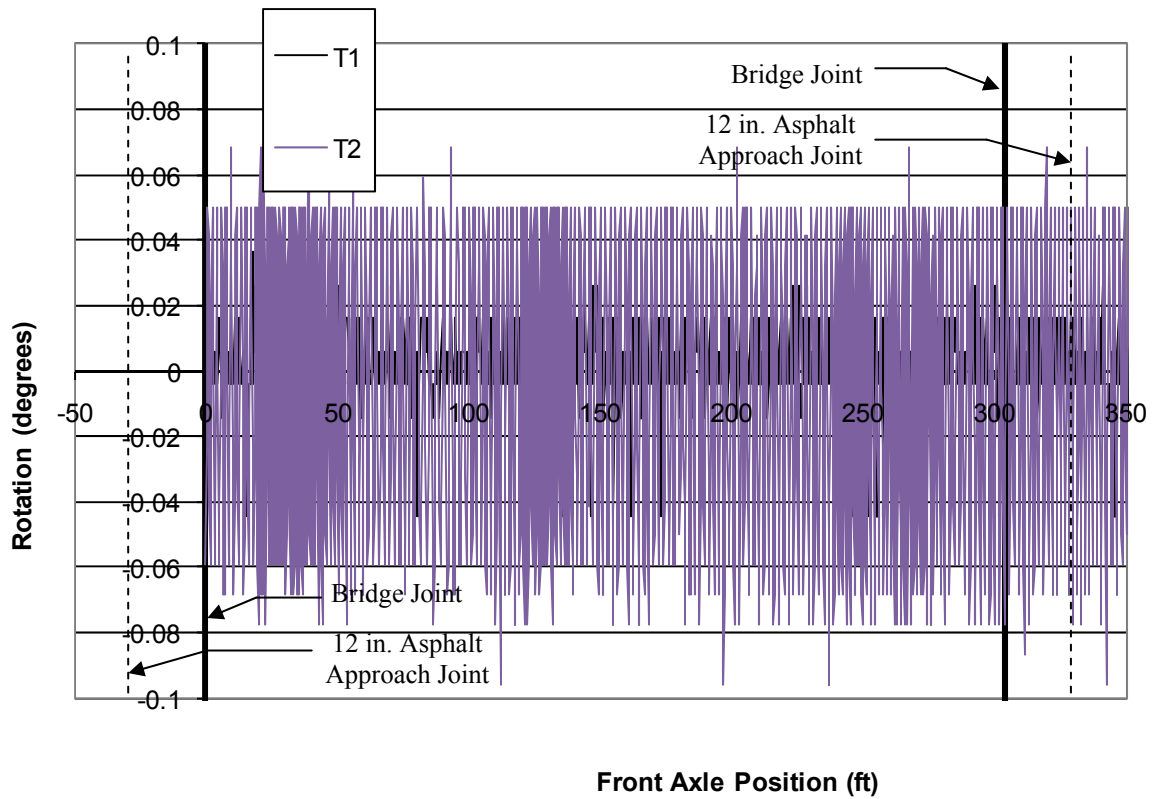
b. Girder strain: near mid-span



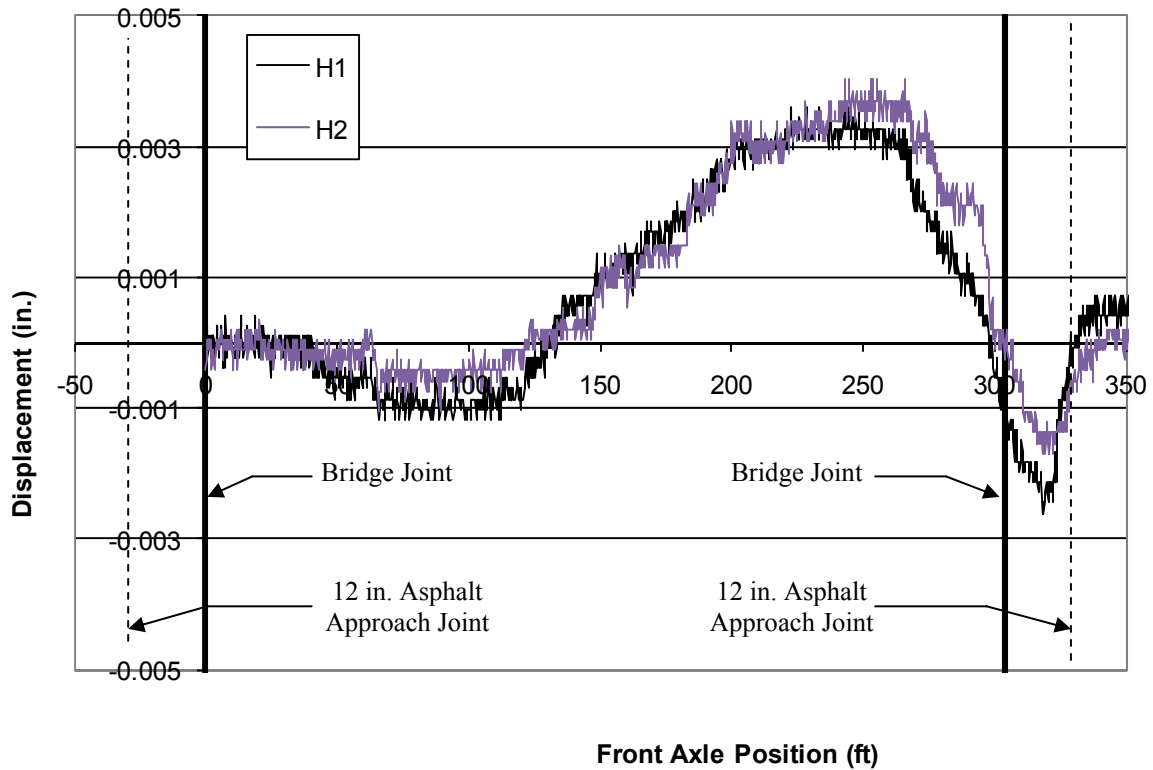
c. Approach slab strain



d. Approach slab vertical movement



e. Abutment rotation



f. Abutment translation

Figure 5.49. FRA 270-32.36 bridge live load testing results

5.11. ERI 60-2.39

5.11.1. ERI 60-2.39 Bridge Description

The ERI 60-2.39 bridge was built circa 1999 and carries two lanes of bi-directional traffic over the Vermilion River. The bridge has five spans with an overall length of about 1,128 ft. (See Figure 5.50). The bridge superstructure consists of five 60 in. deep continuous steel plate girders with a composite reinforced concrete deck. The girders are simply supported at the abutments as shown in Figure 5.51. The abutment has a 21 in. backwall extending up from the girder bearing to prevent the backfill from coming in contact with the superstructure. A strip seal expansion joint was placed between the girder and the backwall. The approach slab is rest on a 6 in. paving notch, which is part of the back wall. The approach slab is 25 ft long and butts directly against the asphalt pavement. The approach slab joint at the pavement and at the bridge is shown in Figure 5.52.



Figure 5.50. Five spans of ERI 60-2.39



Figure 5.51. Girder bearing at abutment



a. Pavement to approach joint



b. Approach to bridge joint

Figure 5.52. Approach slab joint at the pavement and bridge

5.11.2. ERI 60-2.39 Visual Bridge Evaluation

During the field investigation, the bridge overall rideability was good. However, only months prior to testing, new asphalt pavement was placed at both ends of the bridge, as shown in Figure 5.53, to correct previous poor ride issues. The only noticeable signs of deterioration on the bridge were holes in the strip seals as shown in Figure 5.54.



Figure 5.53. New asphalt pavement up to approach slab

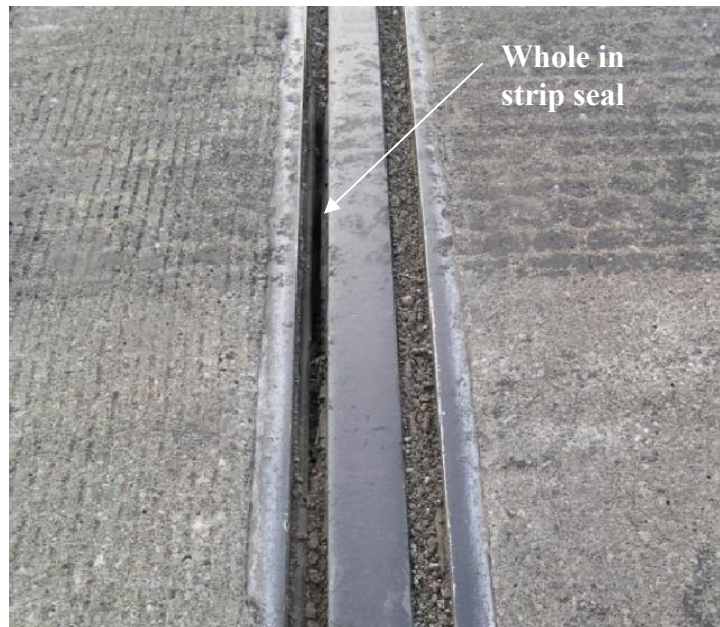


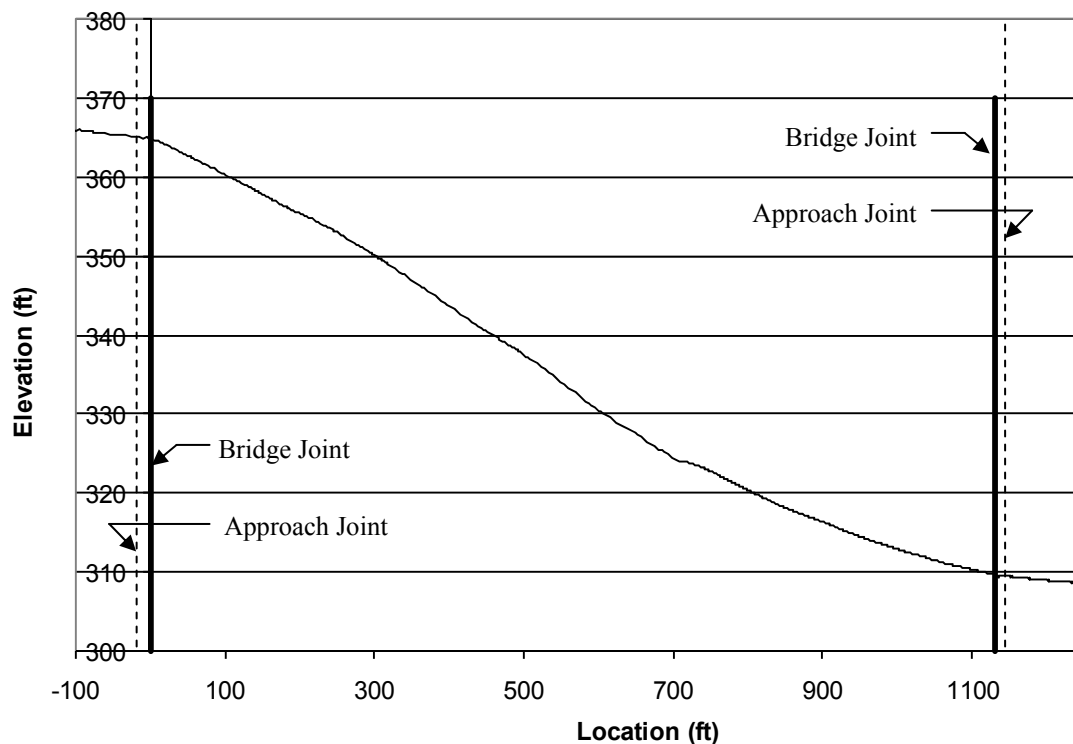
Figure 5.54. Deterioration of strip seal

5.11.3. ERI 60-2.39 Geometric Bridge Testing and Support System Evaluation

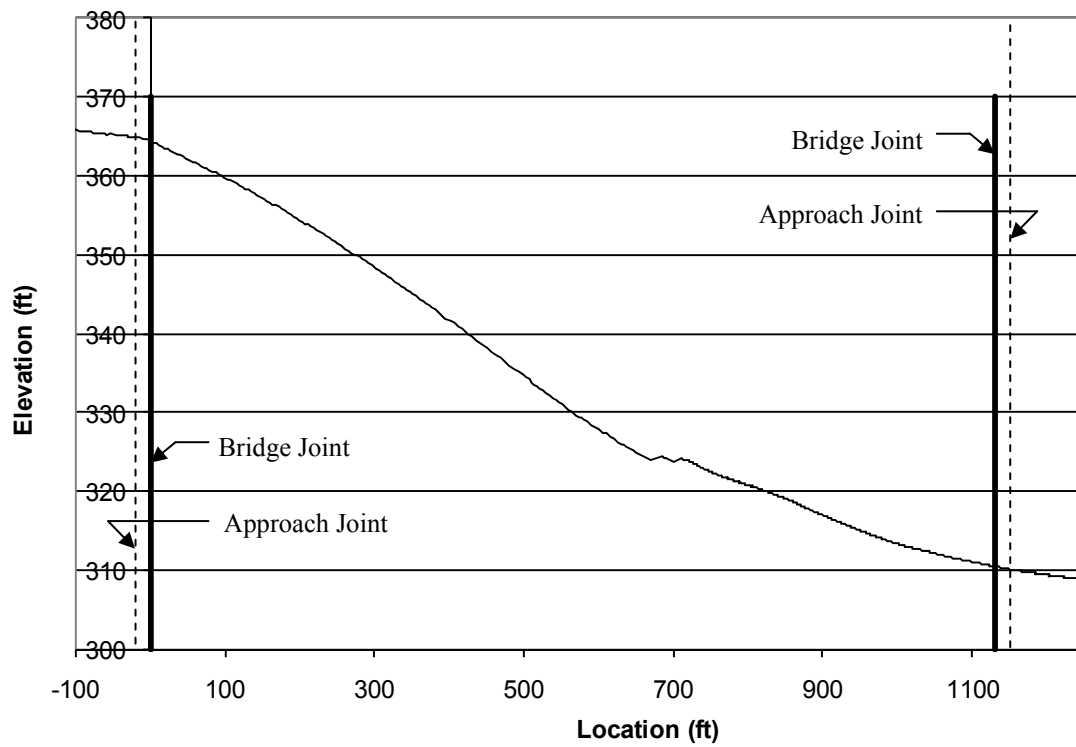
Four types of “geometric” data for the ERI 60-2.39 bridge were collected and evaluated. Specifically, the global geometry was evaluated using a laser-based survey system, the local roadway condition was evaluated using IRI data collected by ODOT, the support system stiffness was evaluated from FWD test results, and the support system was further evaluated through a review of the overall support system depth.

Figures 5.55a and 5.55b show the results of the laser-based survey. Generally, the survey results indicated a relatively consistent and gradually changing bridge profile. The only exceptions are at approximately 700 ft from the bridge entrance, as well as only slight discontinuities at each bridge end. (Note these are somewhat obscured due to the scale of the plots in the report).

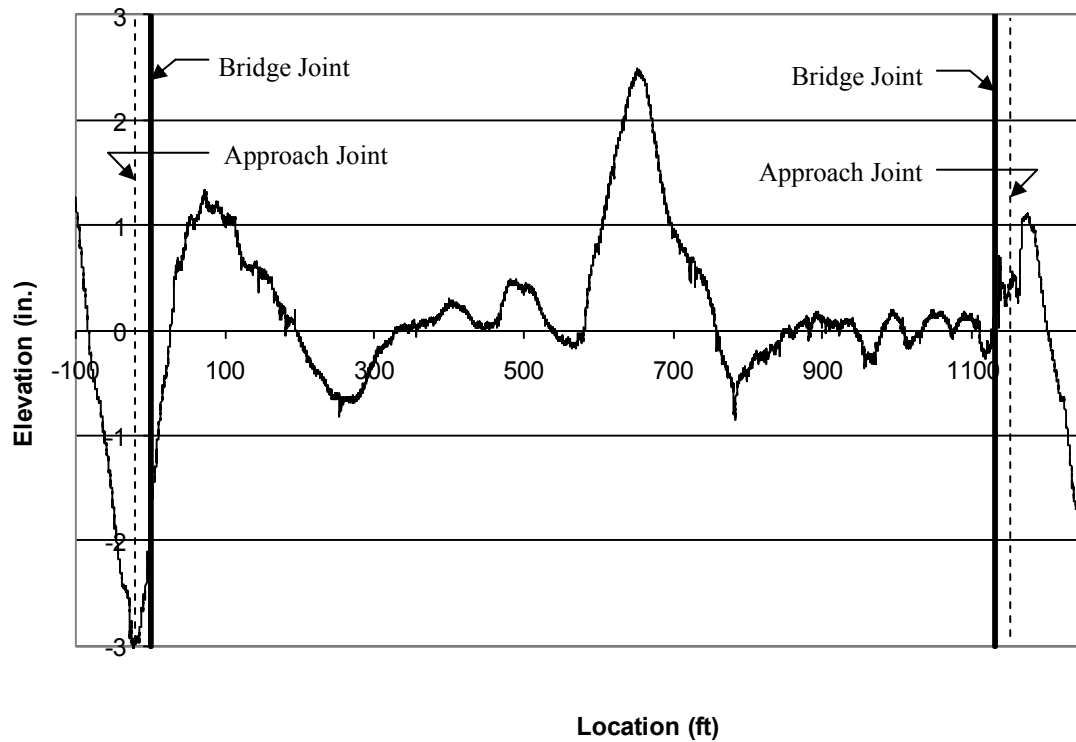
The profiler and IRI data also indicate a relatively consistent surface with slight discontinuities at each end and at the 700 ft location. The FWD test results indicate a region extending approximately 25 ft from the bridge end where the support system is quite stiff. From 25 ft on, the support is less stiff but very consistent.



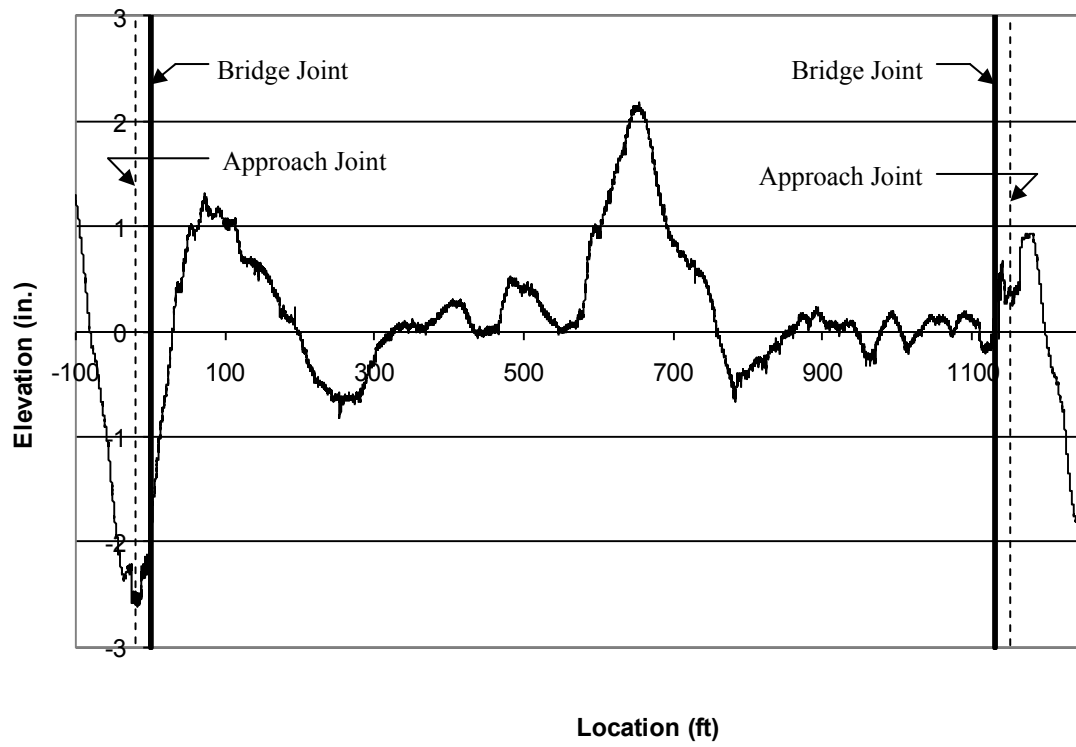
a. Absolute survey data: right wheel line



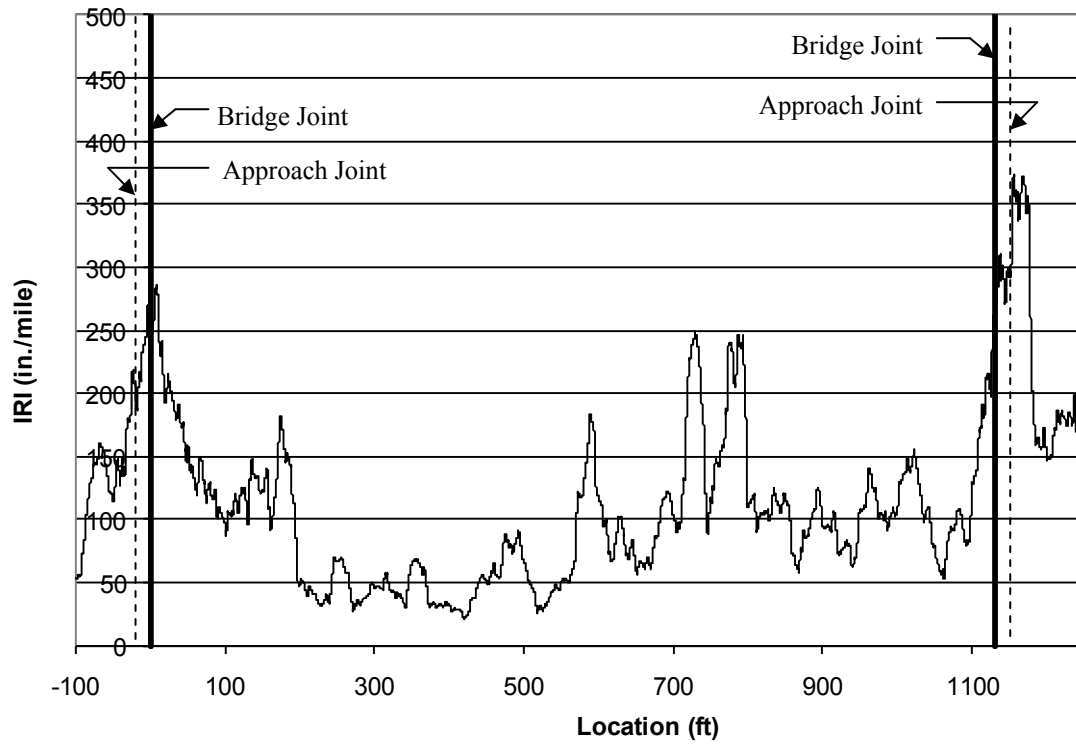
b. Absolute survey data: left wheel line



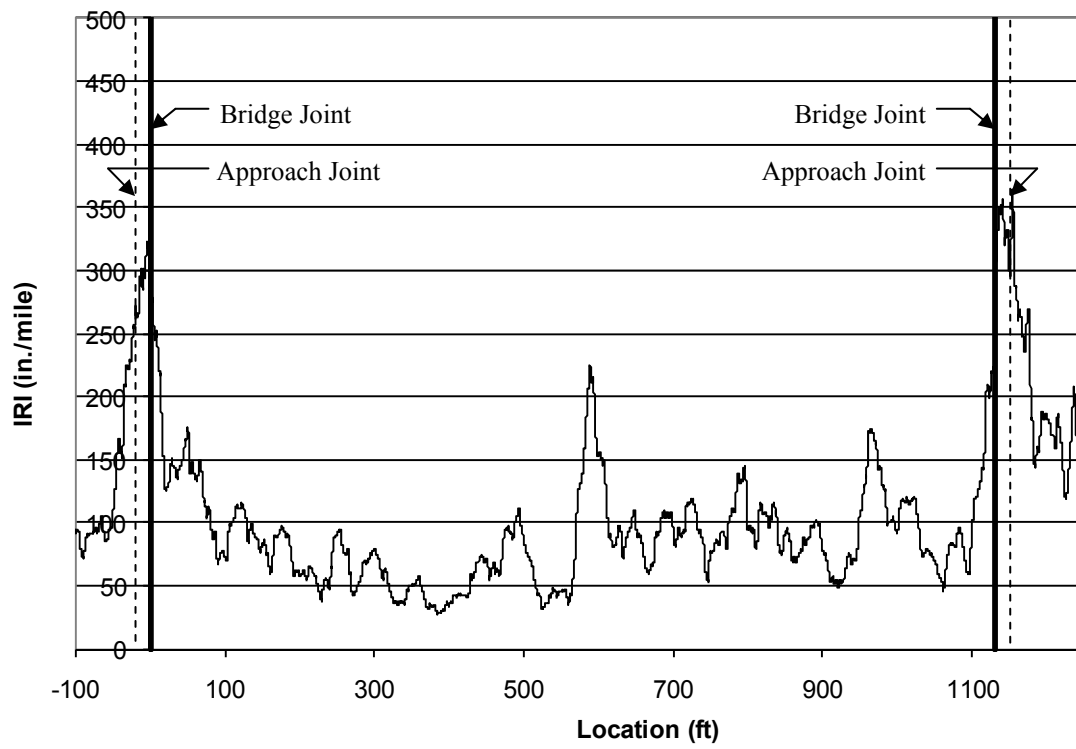
c. Raw profiler data: right wheel line



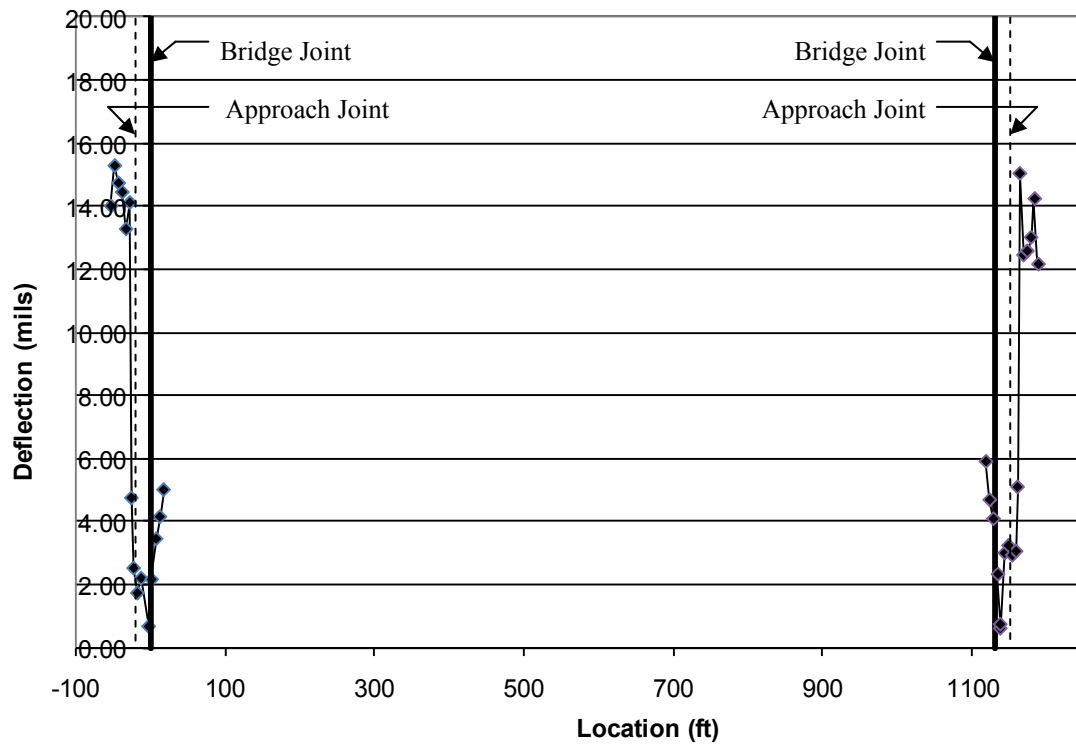
d. Raw profiler data: left wheel line



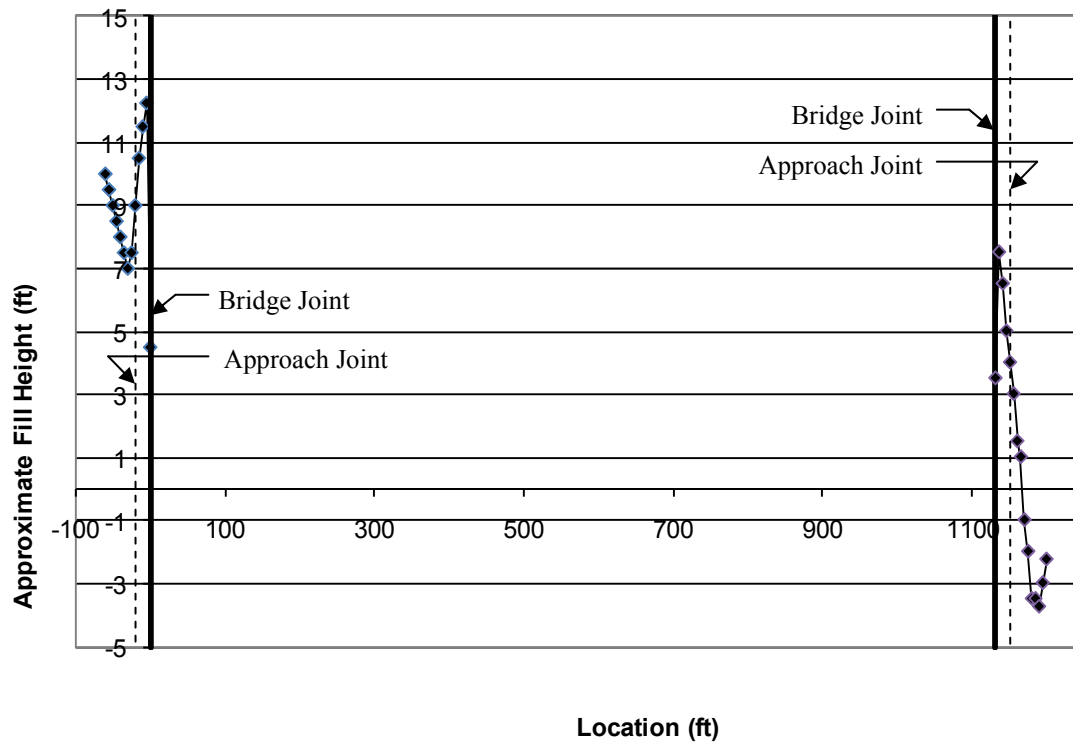
e. IRI: right wheel line



f. IRI: left wheel line



g. FWD test results



h. Approximate fill height

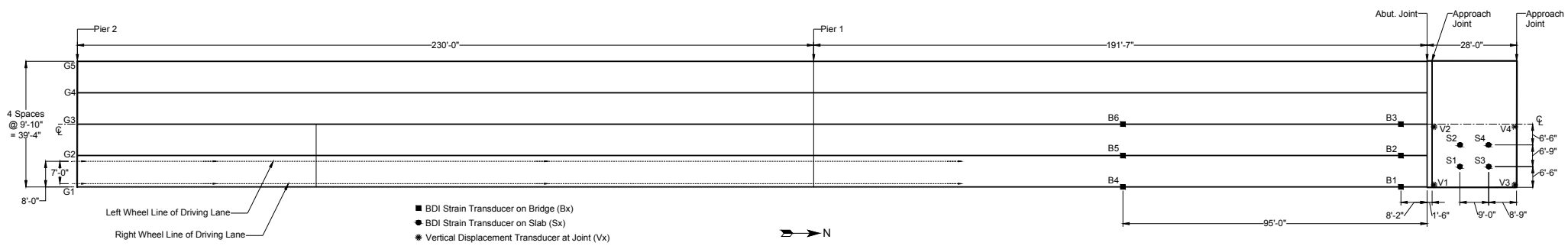
Figure 5.55. ERI 60-2.39 bridge geometric testing results

5.11.4. ERI 60-2.39 Live Load Testing

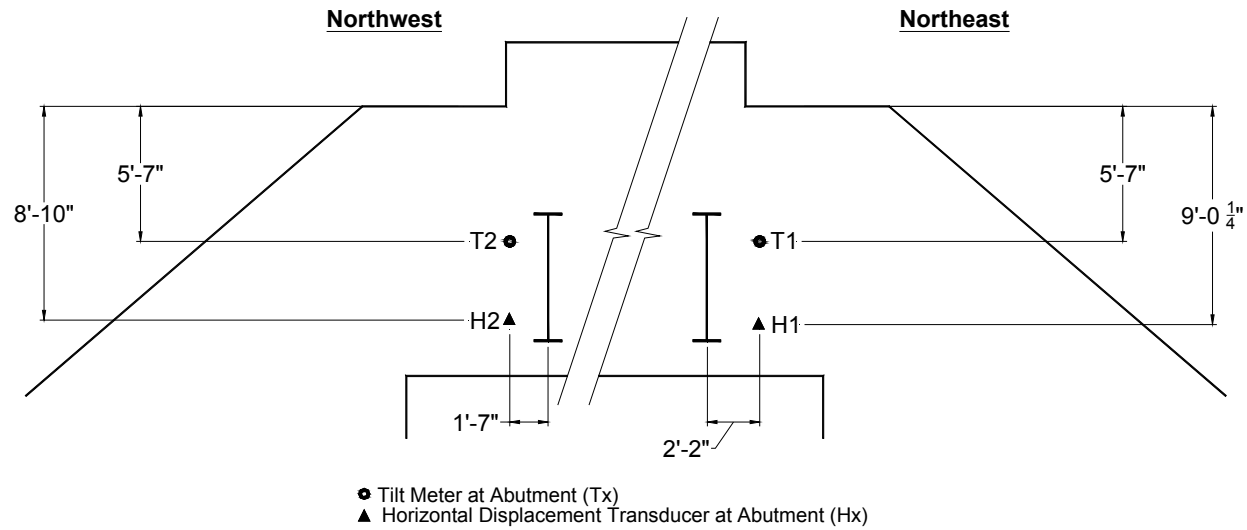
Bridge ERI 60-2.39 was load tested with a loaded legal truck to understand how the bridge behaves under typical live loads. As can be seen in Figure 5.56 the bridge was instrumented with strain gages on three girders (at two cross sections), with strain gages on the top of the approach slab, with deflection gages measuring relative movement of the approach slab corners, and with abutment rotation and translation sensors.

As seen in Figure 5.57, the near girder end strain gages indicate very little end restraint; this behavior is further confirmed by the near midspan strain gages. In general, the stresses at all locations are high, relative to others tested as part of this work. However, the maximum recorded live load stress was just over 1ksi. The approach slab had larger measured strains than other bridges tested during this work. However, the measured strains were still relatively low. Also, relatively vertical displacement of the slab was found to be negligible.

During live load testing, the ERI 60-2.39 bridge abutment was observed to rotate when loaded. However, the rotations are considered to be small. Furthermore, like all other bridges tested, the abutment was observed to translate under live loads. Unlike the other bridges, the movement was only “away” from the soil and back to “neutral.”

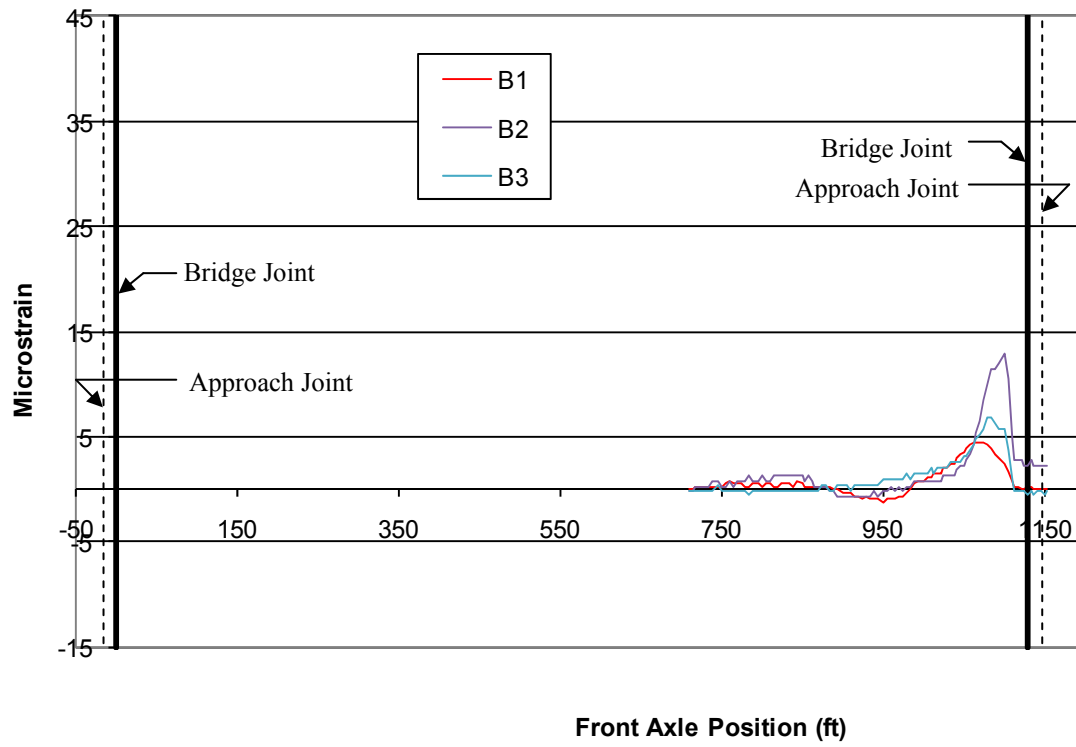


a. Bridge instrumentation plan

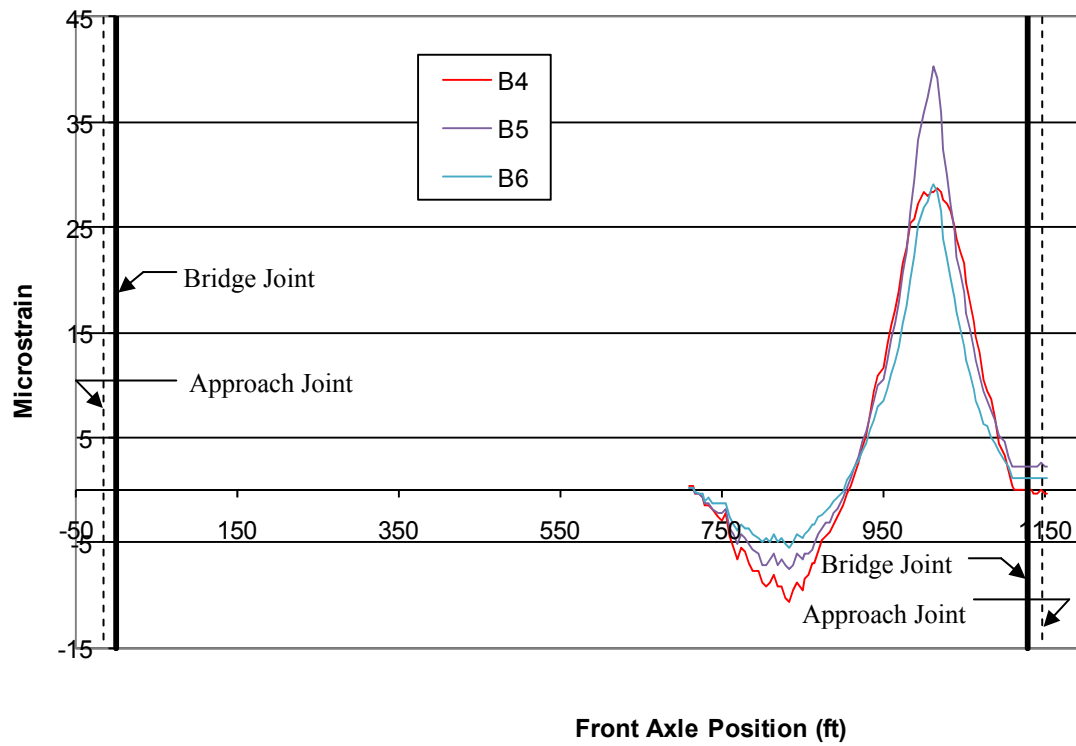


b. Abutment instrumentation

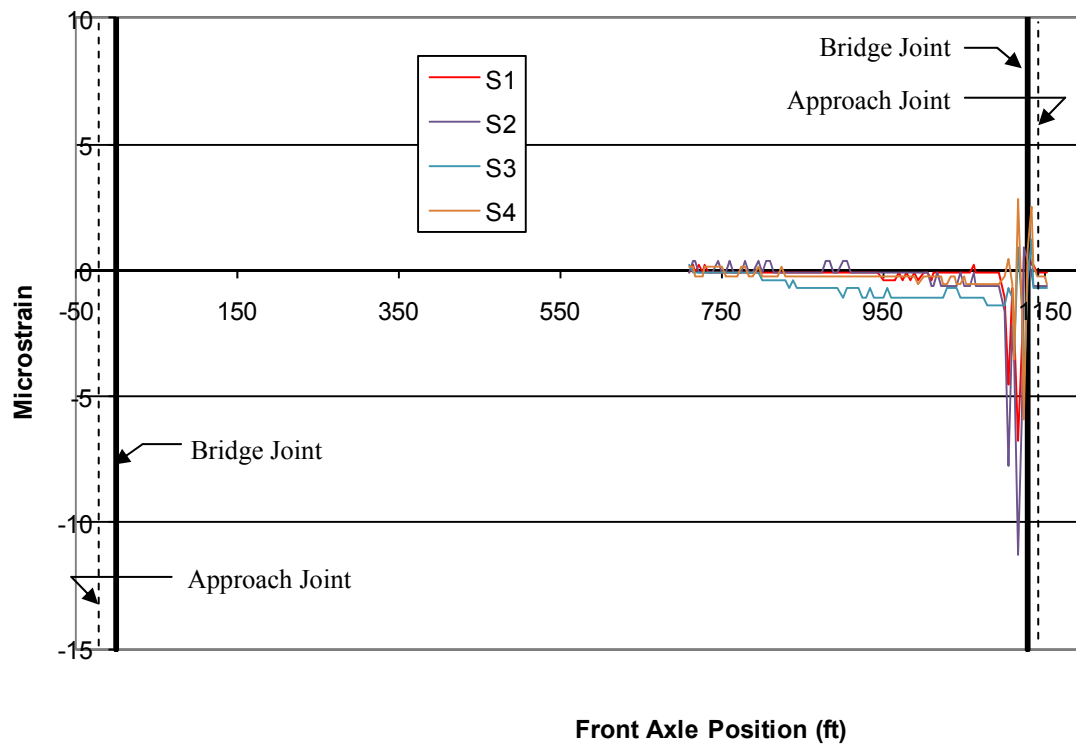
Figure 5.56. ERI 60-2.39 live load testing instrumentation



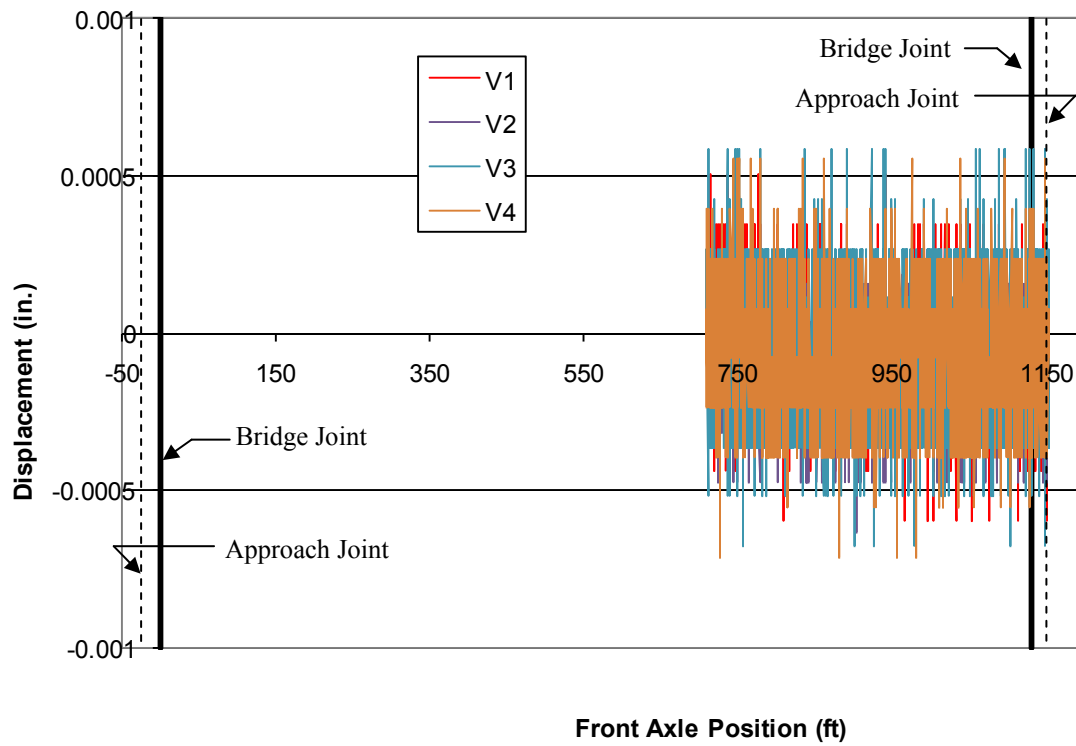
a. Girder strain: near end support



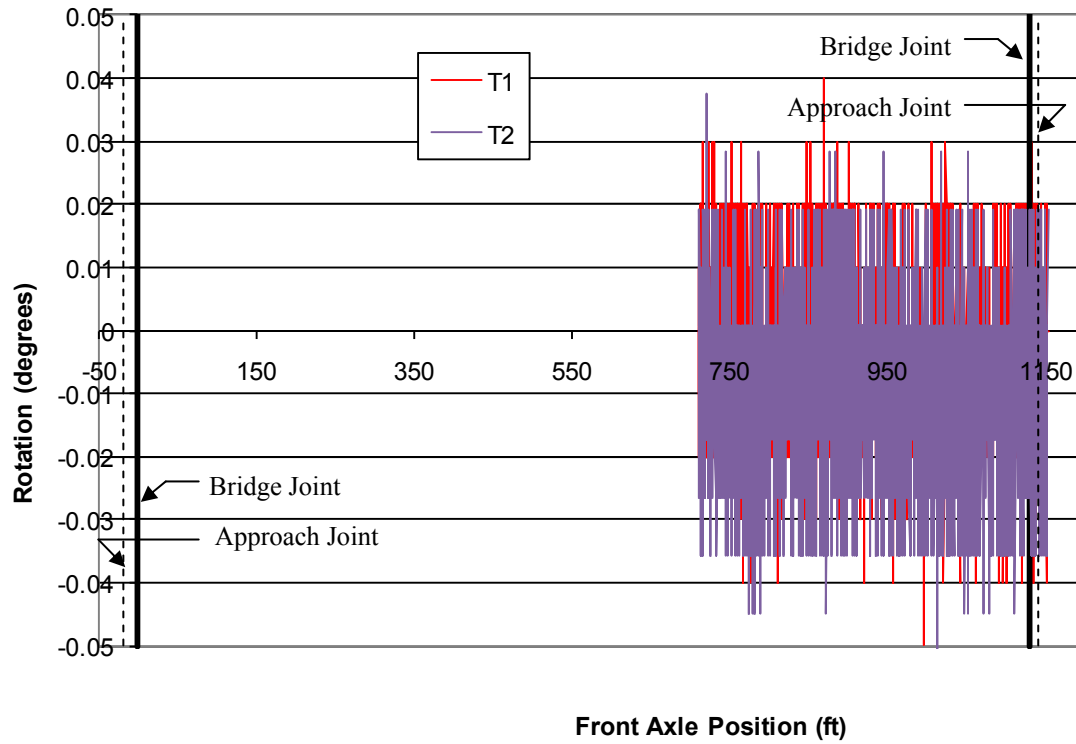
b. Girder strain: near mid-span



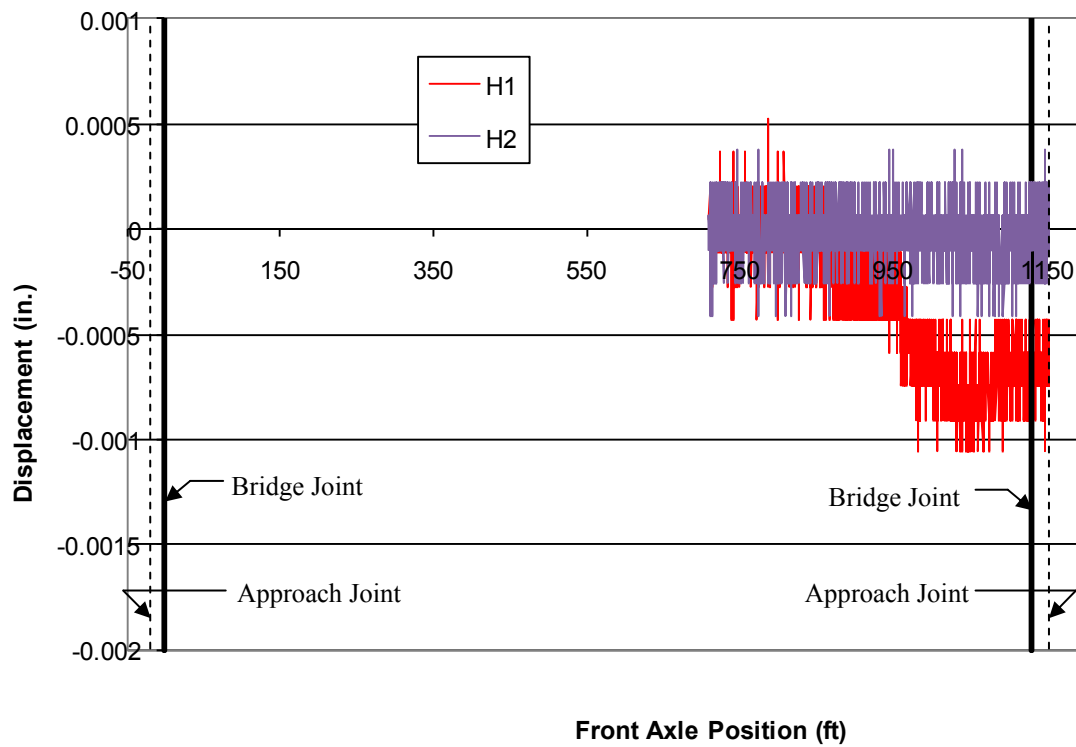
c. Approach slab strain



d. Approach slab vertical movement



e. Abutment rotation



f. Abutment translation

Figure 5.57. ERI 60-2.39 bridge live load testing results

6. IN SITU EVALUATION OF NEW BRIDGE APPROACH FILL MATERIALS

6.1. Introduction

Bridge approach backfill characteristics were studied at several new bridge sites that were under construction during the period of May 14-16, 2009. A summary of the project locations is provided in Table 6.1. The results of the investigation, including laboratory testing and in situ testing, are described in this chapter. The test results generally indicate that within about 6 ft of the abutment wall, the backfill compaction properties are more variable and have lower dry density, lower modulus, and lower strength compared to backfill outside of this area. According to the ODOT design specification, heavy compaction is not allowed within 6 ft of the bridge structures.

Table 6.1. Locations for evaluation of under-construction bridge approach backfill characteristics

Bridge number	Bridge ID (SFN)	Location
1	BUT-75-0660 (0901822)	I-75 & SR129 Interchange
2	CL1-73-0985 (1402293)	Wilmington
3	MOT-75-1393 (5708443)	Downtown Dayton
4	FRA-670-0904B (2517949)	Port Columbus Airport
5	LIC-37-1225L (4501691L)	Licking 161 Over Moots Run
6	MED-71-0729 (5202809)	I-71 & I-76 Interchange (I-71 over Greenwich RD)
7	MED-71-0750 (5204275)	I-71 & I-76 Interchange (Ramp over I-71)

6.2. Test Methods

Two categories of tests were conducted: laboratory testing of samples from each site and testing at each bridge site.

6.2.1. Laboratory Testing

Representative samples of the backfill material were collected by the research team from all the bridge sites and transported to the geotechnical laboratory at ISU to determine the soil index

properties. Grain-size analysis of the materials was conducted following the American Society for Testing and Materials (ASTM International) D422-63 *Standard Test Method for Particle-Size Analysis of Soils* standard procedures, and the materials were classified according to AASHTO and USCS.

Relative density tests were conducted on backfill material samples collected from all of the bridge sites except bridges #5 and #7; the sample from Bridge #5 was cohesive material and a sample was not collected from Bridge #7. Relative density tests were conducted following ASTM D4253, *Standard Test Methods for Maximum Index Density and Unit Weight of Soils Using a Vibratory Table*, and ASTM D4254, *Standard Test Methods for Minimum Index Density and Unit Weight of Soils and Calculation of Relative Density*.

Standard Proctor tests were performed in accordance with ASTM D698-00a, *Standard Test Methods for Laboratory Compaction Characteristics of Soil Using Standard Effort*. Further Atterberg limit tests were performed in accordance with ASTM D4318-05, *Standard Test Methods for Liquid Limits, Plastic Limit, and Plasticity Index of Soils*.

In addition to laboratory tests performed on samples collected from the seven bridge sites, collapse potential tests were performed on backfill material samples that were collected by ODOT and transported to the geotechnical laboratory at ISU. These samples were compacted using a vibratory table at a 50 Hz frequency for 8 minutes. After the material was compacted, a continuous load was applied. As the stress was increased to 14.5 psi, water was introduced at the surface to saturate the specimen while maintaining the applied stress; deflection was recorded during the entire procedure. When the deflection became stable after saturation, the applied load was increased until the specimen ultimate bearing capacity was reached. Figure 6.1 shows the collapse testing.



Figure 6.1. Laboratory evaluation of the collapse potential of the backfill materials

6.2.2. Field Testing

Backfill materials were evaluated in the field with the following:

- Humboldt 5001B NG to determine moisture content and density
- DCP to determine the California Bearing Ratio (CBR)
- Zorn LWD and static plate load test (PLT) device to determine the modulus of the approach fill materials

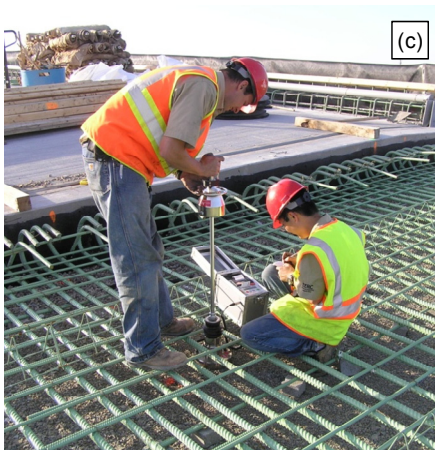
Figure 6.2 shows these devices in use and Table 6.2 summarizes which tests were conducted at each of the seven bridge sites.



a. Dynamic cone penetrometer (DCP)



b. Nuclear gauge (NG)



c. Zorn light weight deflectometer (LWD)



d. Static plate load test (PLT)

Figure 6.2. In situ testing methods/devices

Table 6.2. Summary of in situ testing at different bridge locations

Bridge number	Date	Bridge ID (SFN)	Location	In situ testing
1	5/14/2009	BUT-75-0660 (0901822)	I-75 & SR129 Interchange	DCP, NG
2	5/14/2009	CL1-73-0985 (1402293)	Wilmington	DCP
3	5/14/2009	MOT-75-1393 (5708443)	Downtown Dayton	DCP, LWD
4	5/15/2009	FRA-670-0904B (2517949)	Port Columbus Airport	DCP, LWD
5	5/15/2009	LIC-37-1225L (4501691L)	Licking 161 Over Moots Run	DCP, LWD, NG
6	5/16/2009	MED-71-0729 (5202809)	I-71 & I-76 Interchange (I-71 over Greenwich RD)	DCP, LWD, PLT
7	5/16/2009	MED-71-0750 (5204275)	I-71 & I-76 Interchange (Ramp over I-71)	DCP, LWD

Note: SFN = Structural file number; DCP = Dynamic cone penetrometer; NG = Nuclear moisture density gauge; LWD = 200 mm plate diameter Zorn light weight deflectometer; PLT = 300 mm diameter static plate load test

DCP tests were conducted in general accordance with ASTM D6951-03, *Standard Test Method for Use of the Dynamic Cone Penetrometer in Shallow Pavement Applications*, to measure the dynamic cone penetration resistance or DCP index (DCPI) values in units of mm/blow. The DCPI values were used to estimate the CBR, using equation 6.1:

$$CBR = \frac{292}{DCPI^{1.12}} \quad (6.1)$$

The LWD device was setup with a 0.2 in. diameter plate and a 21.7 in. drop height. Tests were conducted in accordance with manufacturer recommendations (See Zorn, 2003) to determine the elastic modulus using equation (2):

$$E = \frac{(1 - \nu^2)\sigma_0 r}{d_0} \times f \quad (6.2)$$

where: E = elastic modulus (psi), d_0 = measured deflection (in.), ν = Poisson's ratio, σ_0 = applied stress (psi), r = radius of the plate (in.), and f = shape factor (assumed as 8/3; see Vennapusa and White, 2008). The elastic modulus determined from the LWD device is denoted as E_{LWD-Z2} in the following discussions.

Displacement-controlled static PLTs were conducted by applying a static load on an 11.8 in. diameter plate against a 3,133 kilo-pound capacity reaction force. The applied load was measured using a 4,549 kilo-pound load cell and deformations were measured using three 2.0 in. linear voltage displacement transducers (LVDTs). The average of the three deflection measurements was used for calculations. The load and deformation readings were continuously recorded during the test using a data logger. Equation 6.2 was used to determine the initial modulus (E_{V1}) and the re-load (E_{V2}) modulus with stress and deformation readings taken from the 4,206-8,412 psi stress range as shown in Figure 6.3.

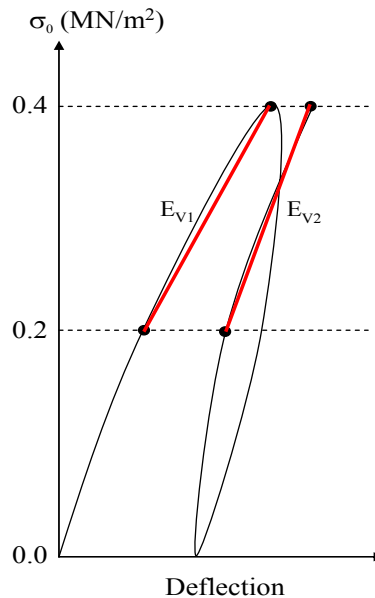


Figure 6.3. E_{V1} and E_{V2} determination from static plate load test

6.3. Laboratory Test Results

Table 6.3 summarizes the soil index properties for the backfill materials collected by ODOT personnel and transported to the geotechnical laboratory at ISU. Figure 6.4 and Figure 6.5 show the grain-size distribution curves for these materials. Table 6.4 summarizes the laboratory test results for the material sampled during this field study, and Figure 6.6 to Figure 6.11 provide the grain-size distribution curves of those materials. The moisture content and dry unit weight relationships of the backfill materials collected by ODOT personnel are shown in Figure 6.12. The laboratory collapse potential tests were performed on the compacted specimens; the results of these tests are shown in Figure 6.13 through Figure 6.21. The pre-saturation modulus were the slope of the curve when the applied stress increases from 1.1 psi to 2.1 psi, and the post-saturation modulus were the slope of the curve after the samples were saturated as shown in Figure 6.18. Figure 6.19 indicates the collapse potential of the backfill materials. Collapse potential is herein defined as the ratio of the amount of settlement due to inundating the sample with water to the height of the compacted specimens. Figure 6.22 shows the relationships between moisture content and collapse potential of Marzane at Perryville sand type 2 at 1% moisture increments from 1 to 12%. Collapse potential measurements ranged from 0 to 14%.

Table 6.3. Soil index properties of bridge approach fill materials tested in situ

Description	Location				
	Marzane at Perryville			Shelly at Newark	West Mill Grove
Material ID	Natural Sand Type 1	Natural Sand Type 2	Crushed Gravel	Natural Sand	MF Sand
Gravel Content (%) (>0.187 in.)	0.1	—	49.4	—	2.9
Sand Content (%) (0.187 in. – 0.002952 in.)	95.9	94.6	33.6	91.7	93.3
Fine Content (%) (<0.002952 in.)	4.0	5.4	17.0	8.3	3.8
Coefficient of Uniformity (C_u)	5.47	5.40	—	6.09	8.31
Coefficient of Curvature (C_c)	0.77	0.90	—	1.19	1.31
AASHTO	A-1-b	A-1-b	A-1-b	A-1-b	A-1-b
USCS	SP	SP-SM	GM	SW-SM	SW

Table 6.4. Summary of index properties of bridge approach fill materials tested in situ

Description	Bridge ID				
	BUT-75-0660	CL1-73-0985		MOT-75-1393	
Material ID	BUT Select Fill	CL1 Sand	MOT Sand	MOT Pea Gravel	MOT Subbase
Gravel Content (%) (>4.75mm)	—	—	3	97	68
Sand Content (%) (4.75mm - 75 µm)	99	95	88	—	32
Fine Content (%) (< 75 µm)	1	5	9	3	—
Coefficient of Uniformity (C_u)	3.15	7.09	6.83	1.76	23.06
Coefficient of Curvature (C_c)	0.96	1.18	1.74	1.14	2.52
Maximum Dry Unit Weight (kN/m ³)	18.7	18.8	17.9	16.2	21.1
Minimum Dry Unit Weight (kN/m ³)	15.6	14.5	13.7	14.7	16.3
AASHTO	A-1-b	A-1-b	A-1-b	A-1-a	A-1-a
USCS	SP	SW-SM	SW-SM	GP	GW

Description	Bridge ID				
	FRA-670-0904B		LIC-37-1225L		MED-71-0729
Material ID	FRA Porous Backfill	FRA Subbase	LIC EB- Till	LIC WB-Till	MED SB Gravel
Gravel Content (%) (>4.75mm)	95	43	24	39	49
Sand Content (%) (4.75mm - 75 µm)	1	33	35	34	44
Fine Content (%) (< 75 µm)	4	14	41	27	7
Coefficient of Uniformity (C_u)	1.82	—	—	—	36.74
Coefficient of Curvature (C_c)	1.16	—	—	—	0.84
Maximum Dry Density (kg/m ³)	15.8	18.2	—	—	20.8
Minimum Dry Density (kg/m ³)	14.2	15.1	—	—	16.4
Liquid Limit, LL (%)	NP	NP	23	24	NP
Plasticity Index, PI	NP	NP	8	7	NP
AASHTO	A-1-a	A-1-a	A-4	A-2-4	A-1-a
USCS	GP	GM	SM	GM	GP-GM

Note: No material was collected from MED-71-0750

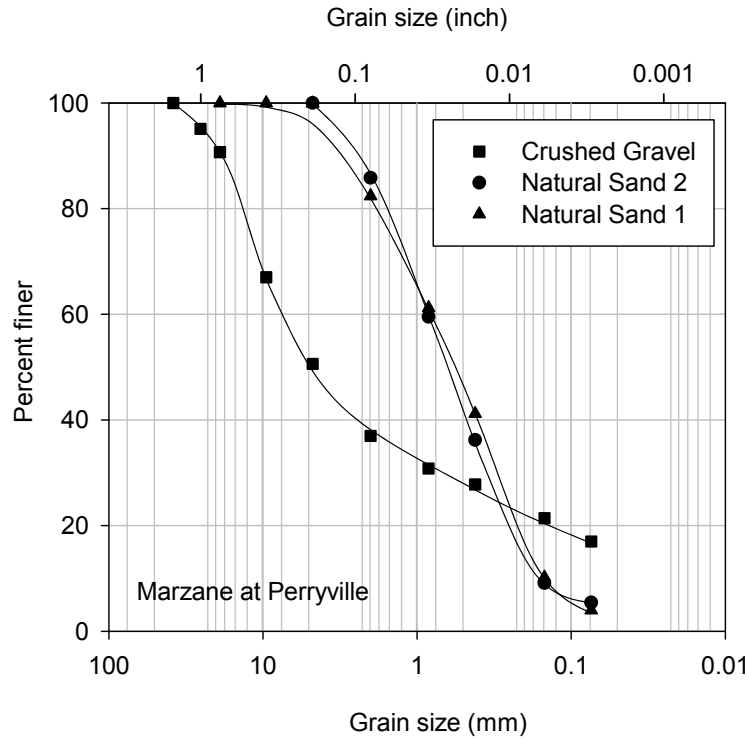


Figure 6.4. Grain-size distribution curves for materials sampled from Marzane at Perryville

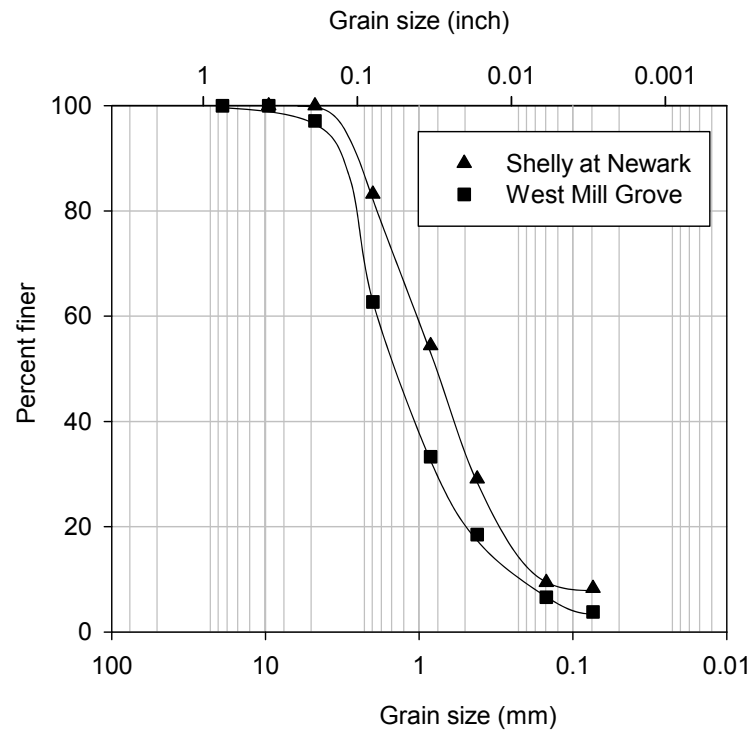


Figure 6.5. Grain-size distribution curves for materials sampled from Shelly at Newark and West Mill Grove

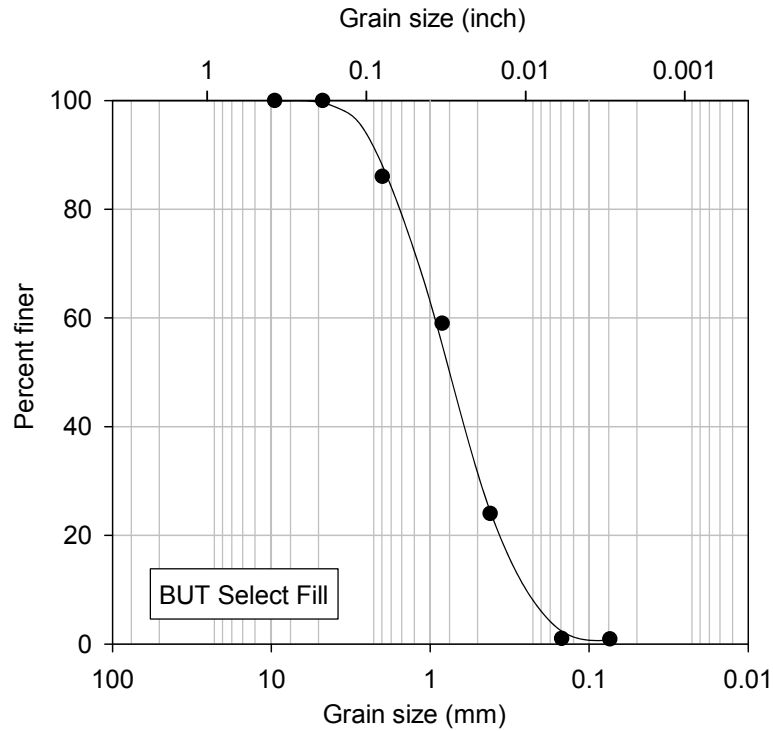


Figure 6.6. Grain-size distribution curves for material sampled from Bridge #1

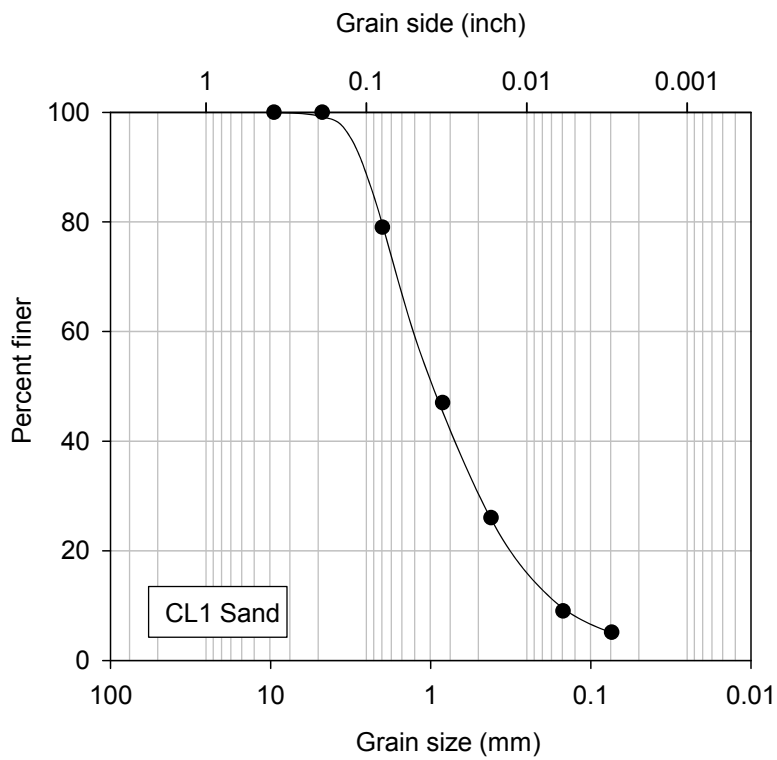


Figure 6.7. Grain-size distribution curve for material sampled from Bridge #2

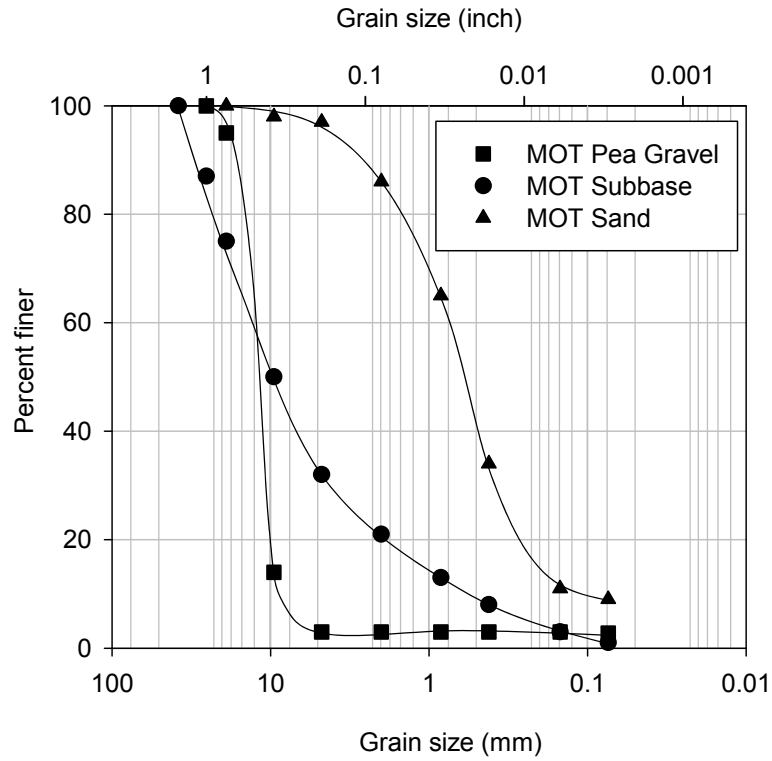


Figure 6.8. Grain-size distribution curves for materials sampled from Bridge #3

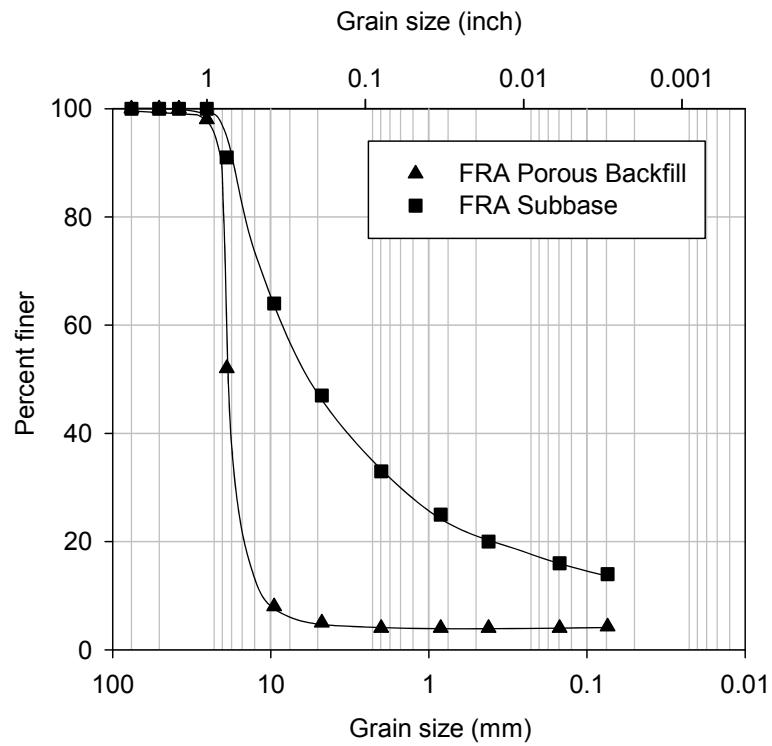


Figure 6.9. Grain-size distribution curves for materials sampled from Bridge #4

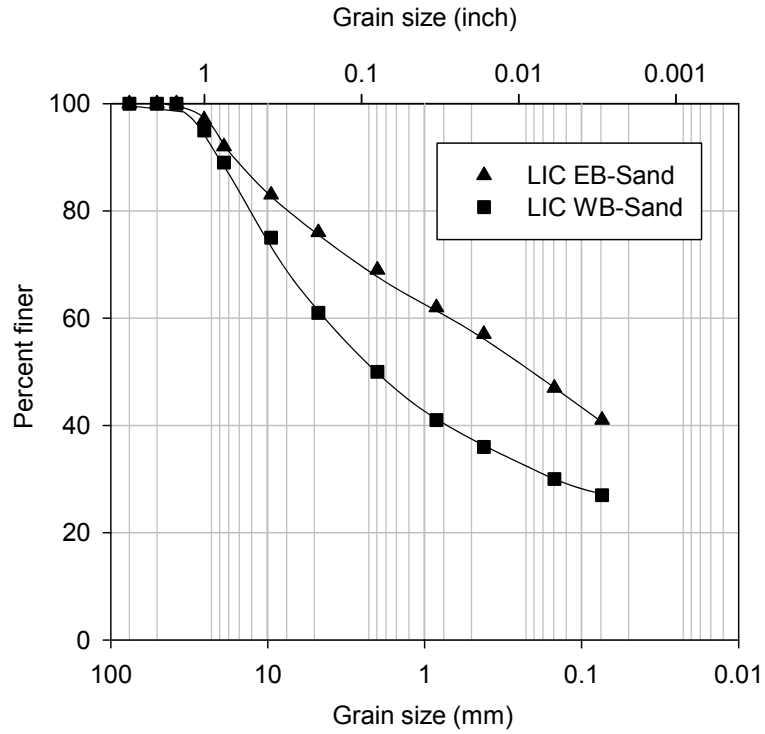


Figure 6.10. Grain-size distribution curves for materials sampled from Bridge #5

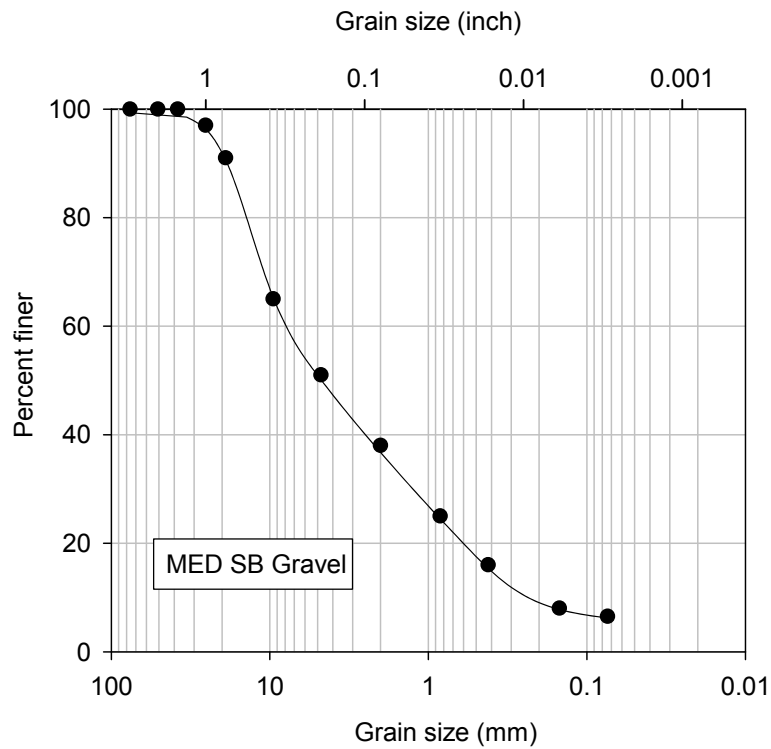


Figure 6.11. Grain-size distribution curve for material sampled from Bridge #6

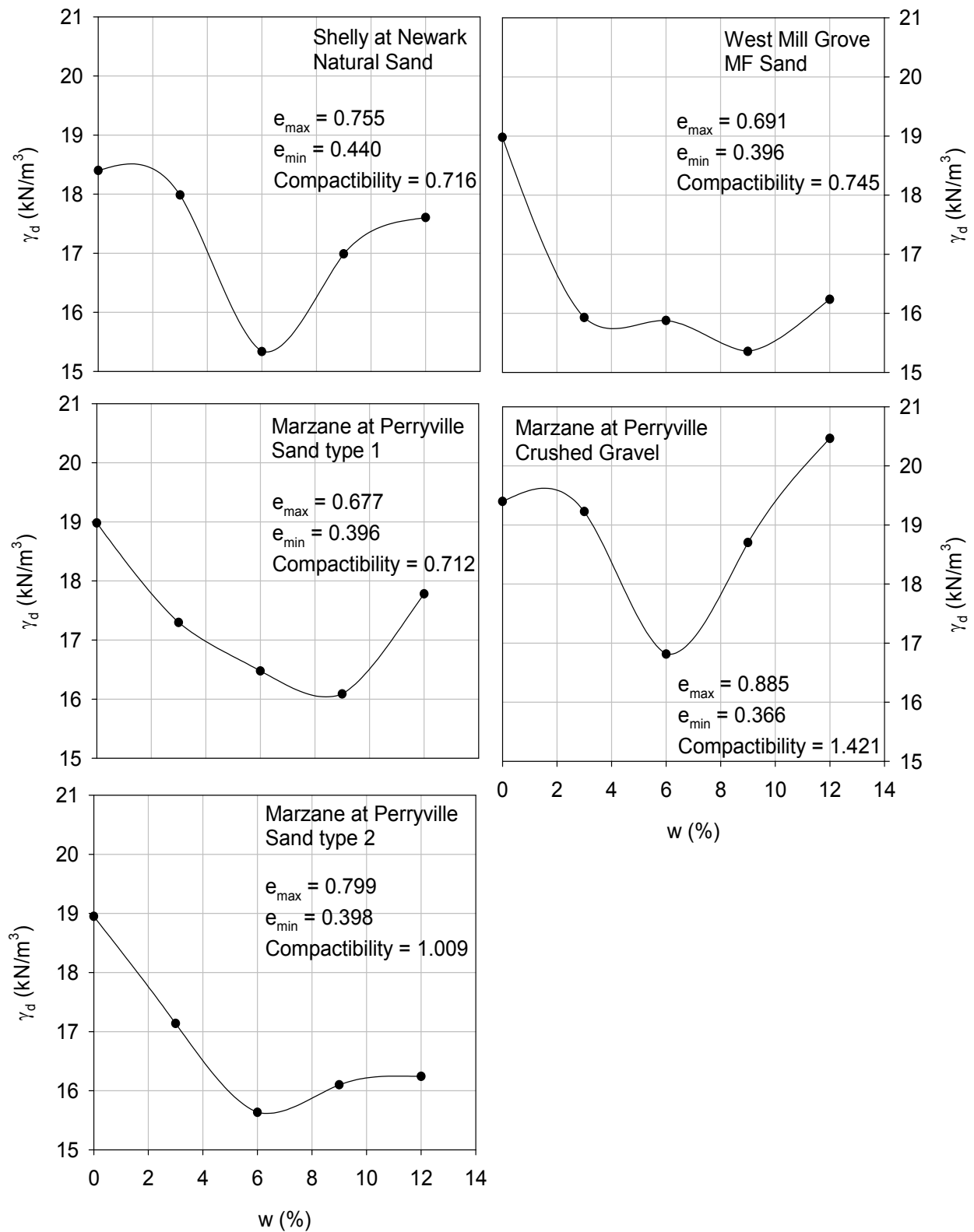


Figure 6.12. Moisture and dry unit weight relationships developed by using vibratory compaction (bulking moisture contents in the range of about 6%)

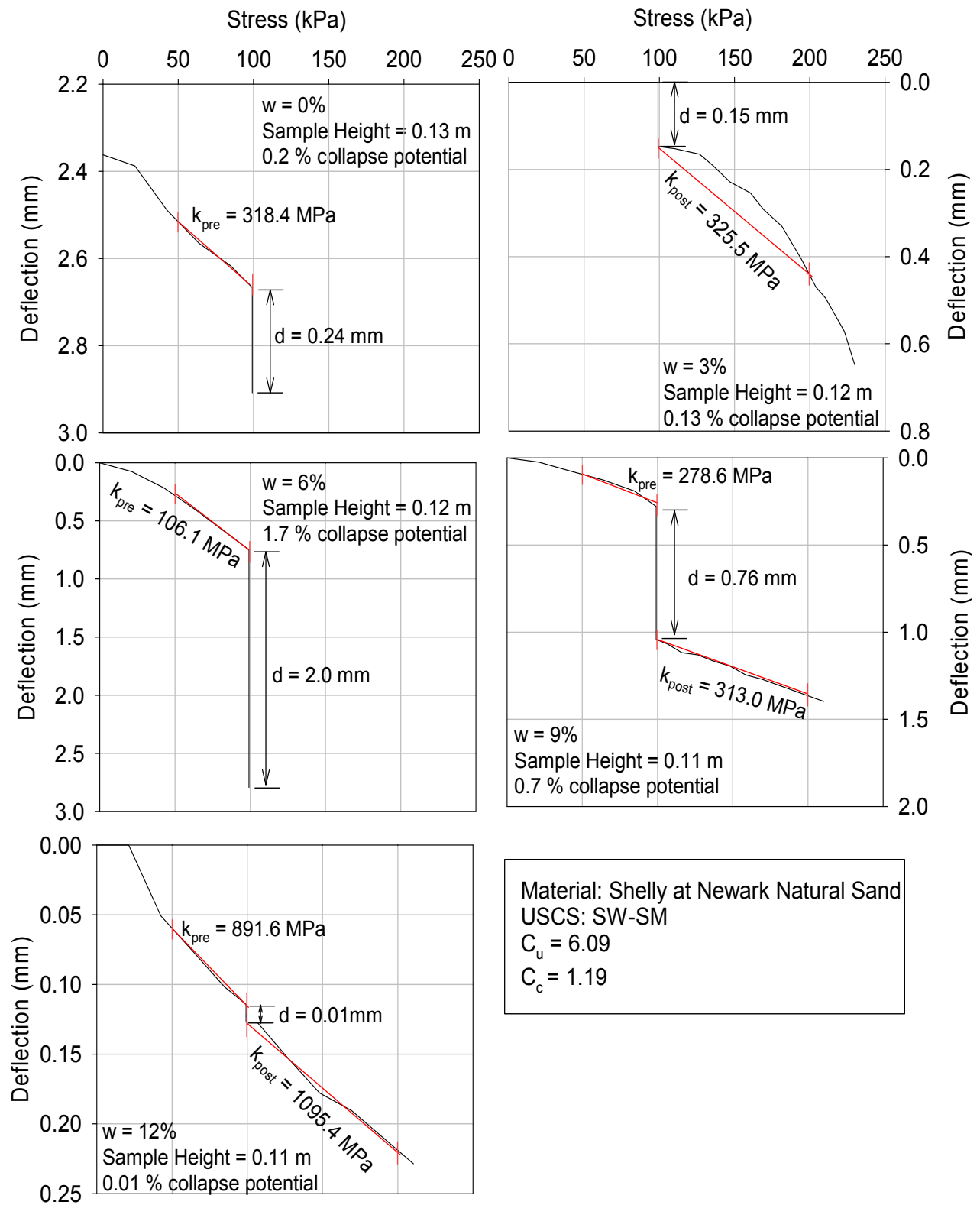


Figure 6.13. Collapse potential tests results for material - Shelly at Newark natural sand

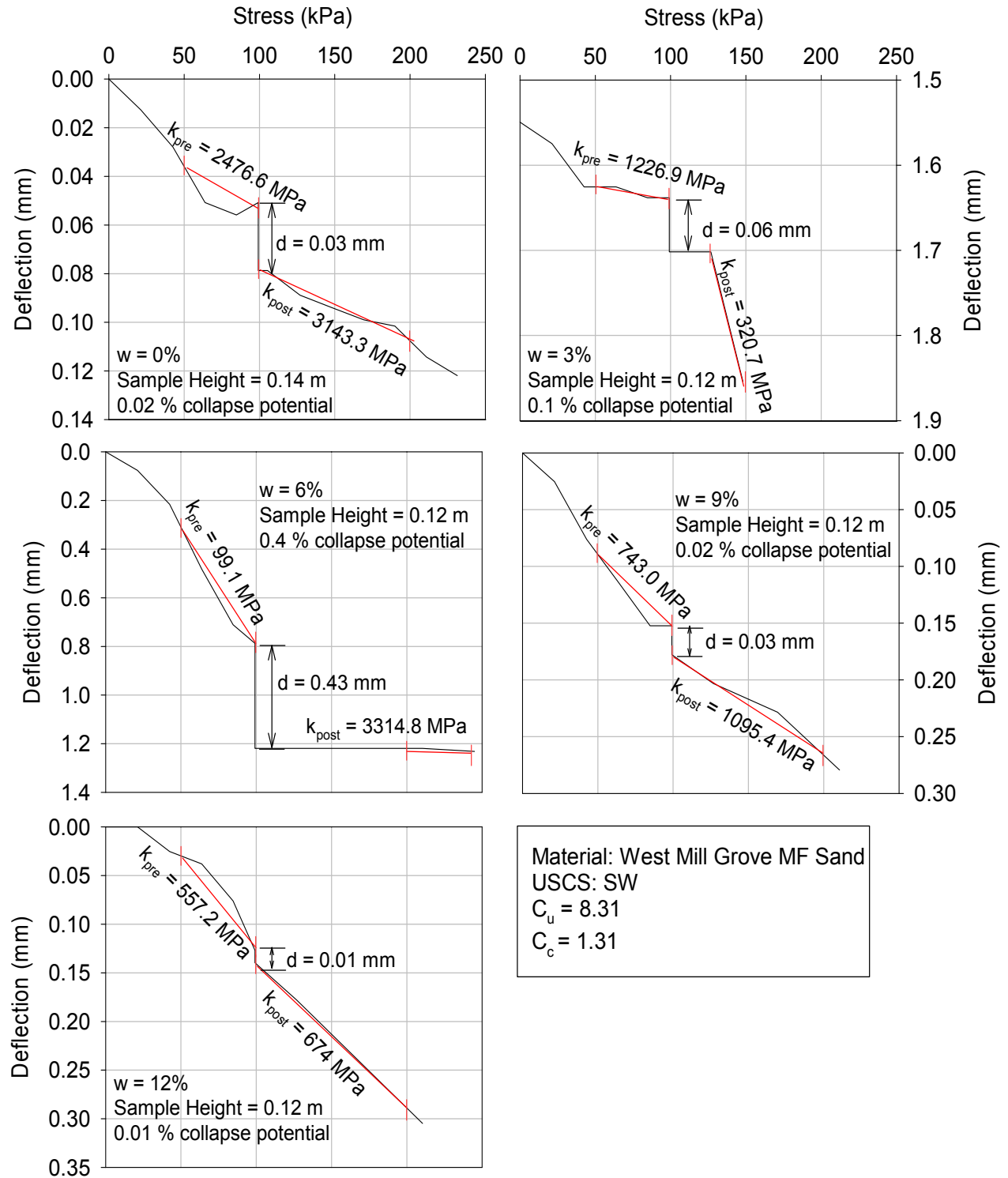


Figure 6.14. Collapse potential tests results for material – West Mill Grove MF sand

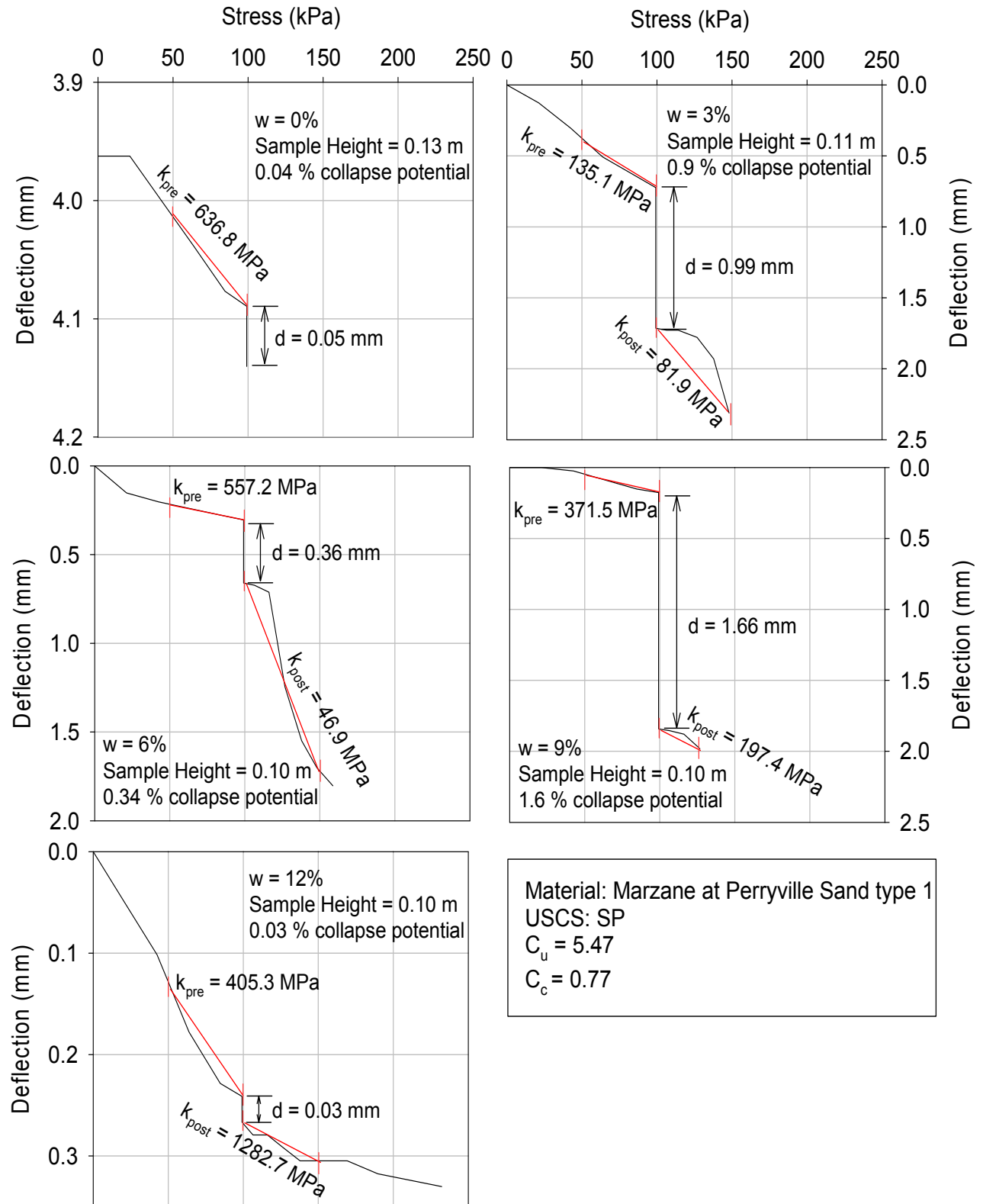


Figure 6.15. Collapse potential tests results for material – Marzane at Perryville Sand 1

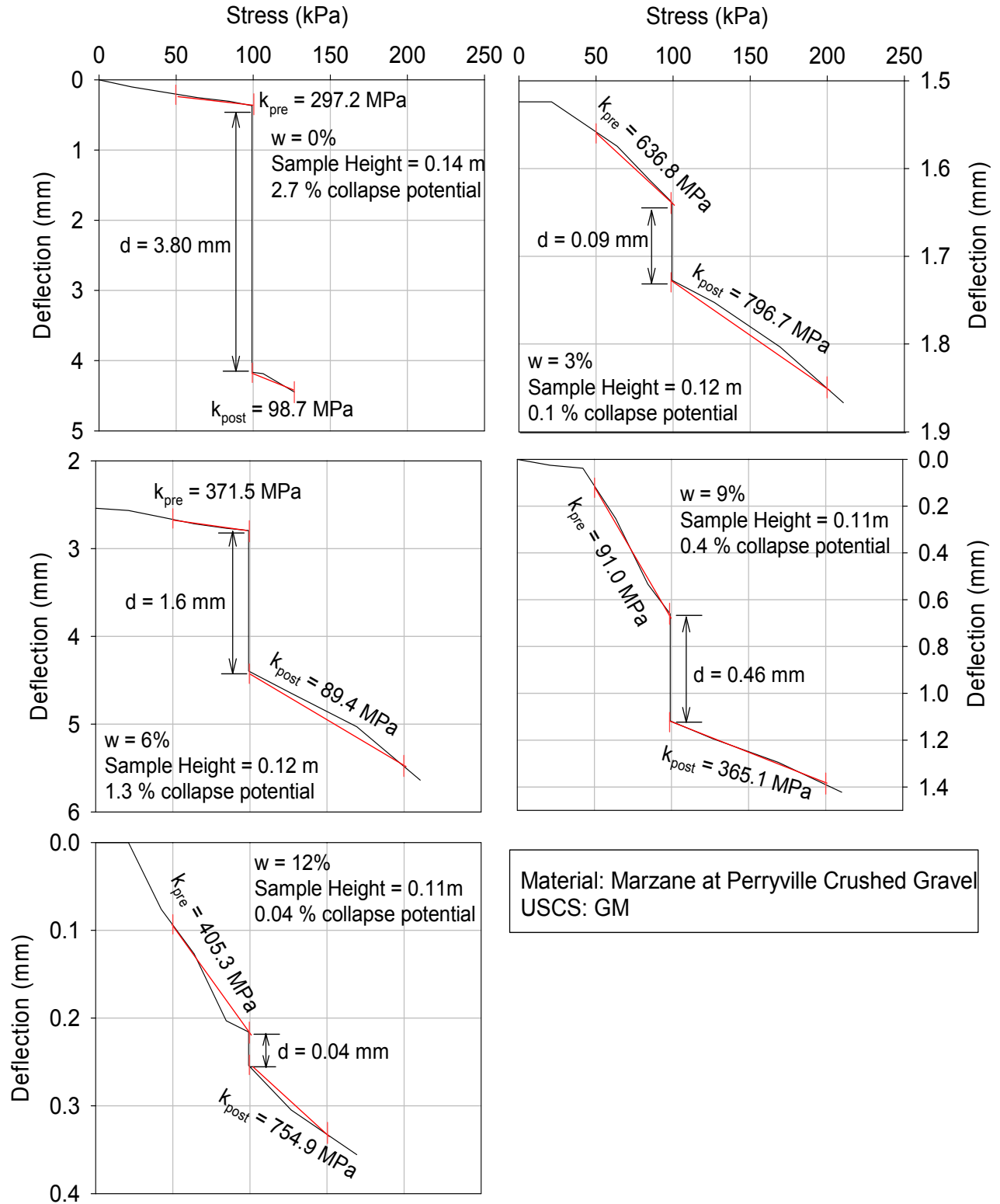


Figure 6.16. Collapse potential tests results for material – Marzane at Perryville gravel

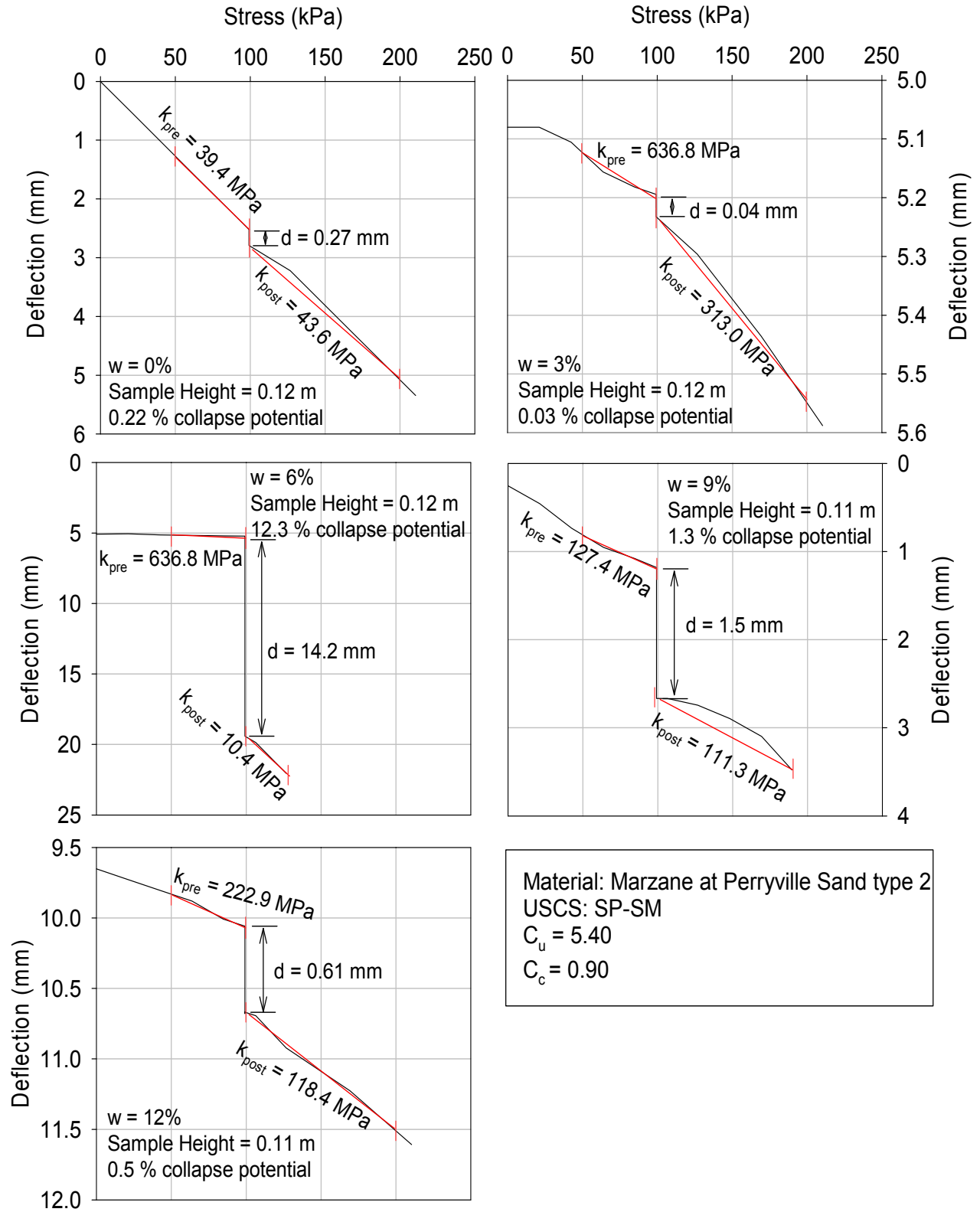


Figure 6.17. Collapse potential tests results for material – Marzane at Perryville Sand 2

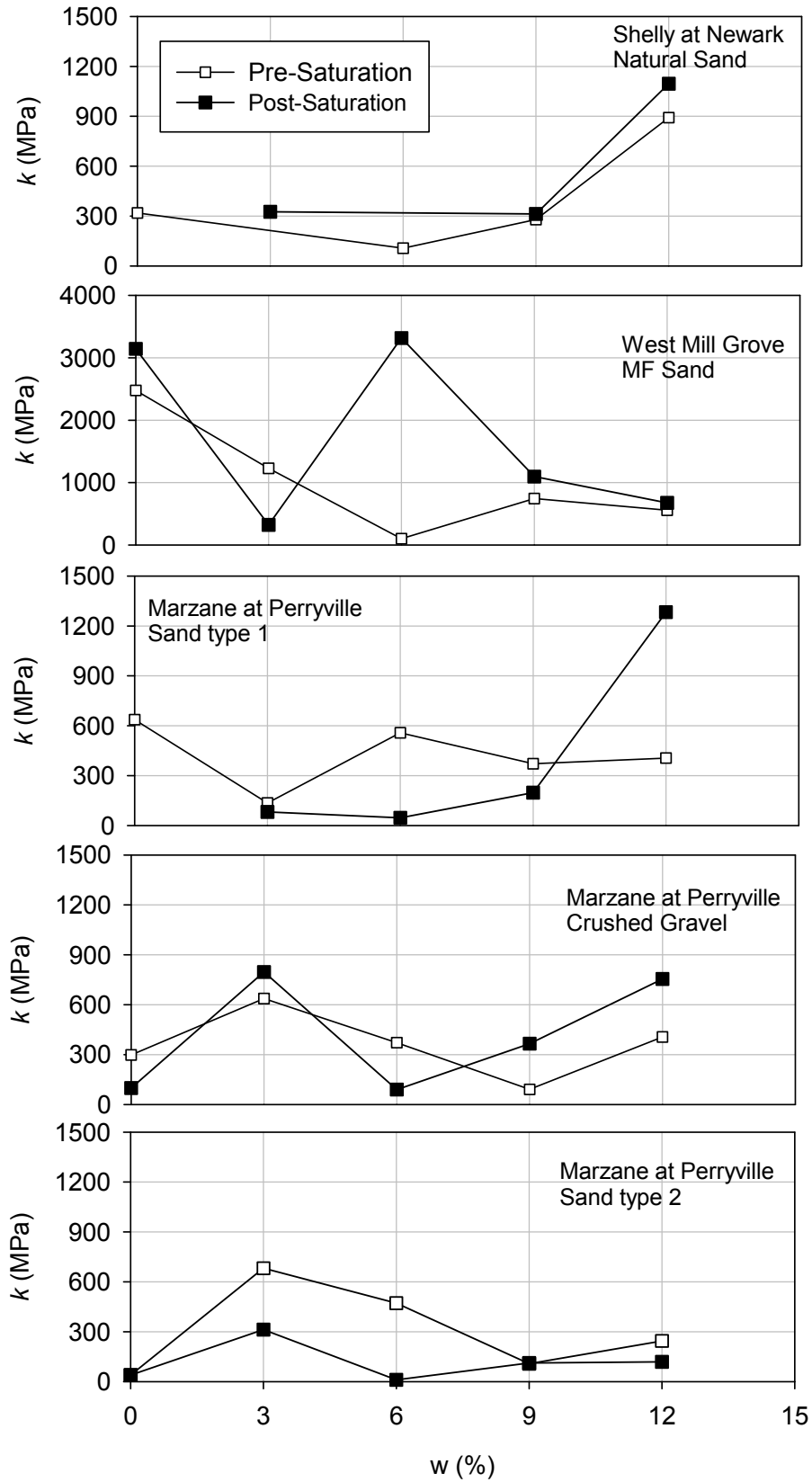


Figure 6.18. Pre-saturation and post-saturation modulus versus moisture content

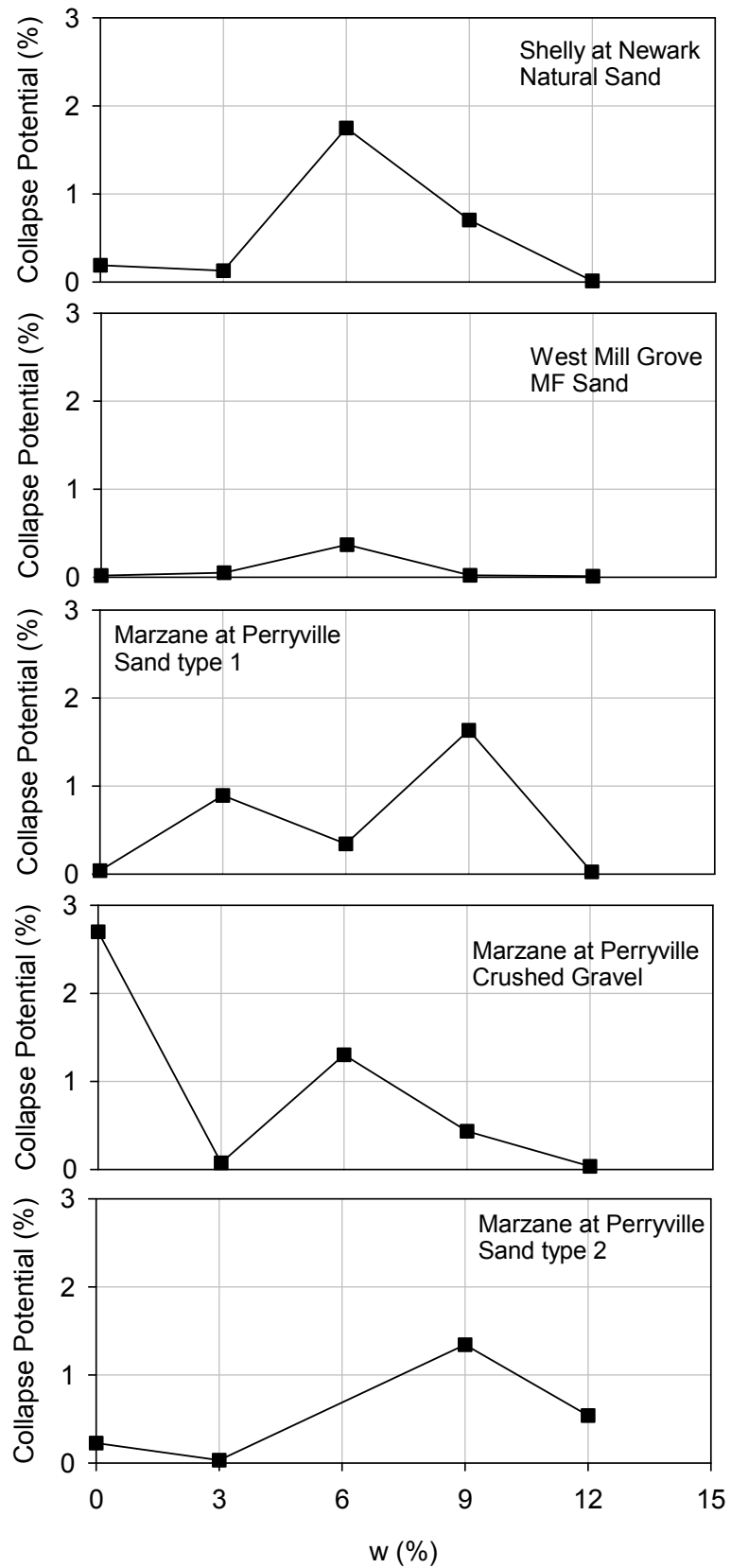


Figure 6.19. Collapse potentials versus moisture content

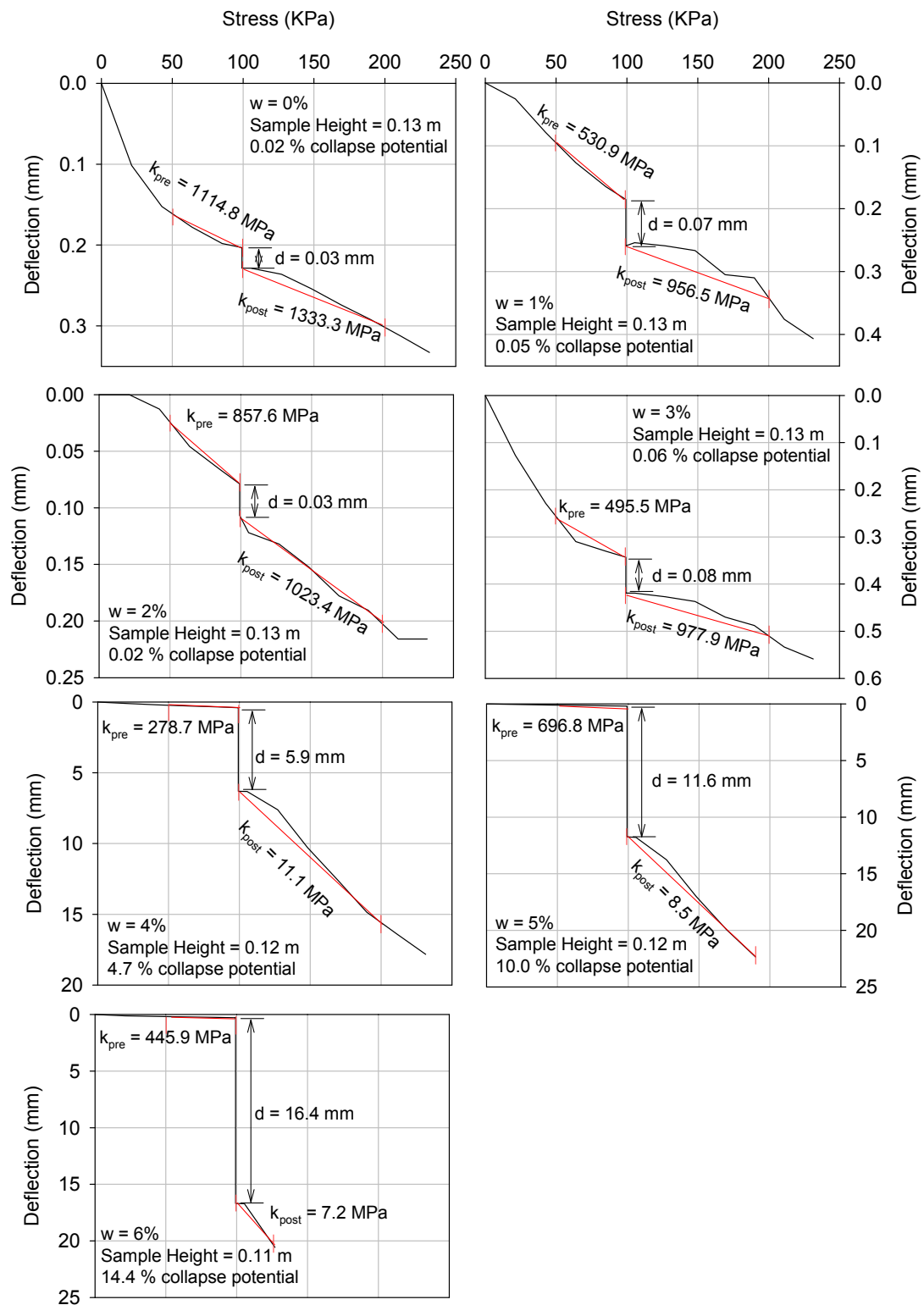


Figure 6.20. Collapse potential versus moisture contents – Marzane at Perryville Sand 2

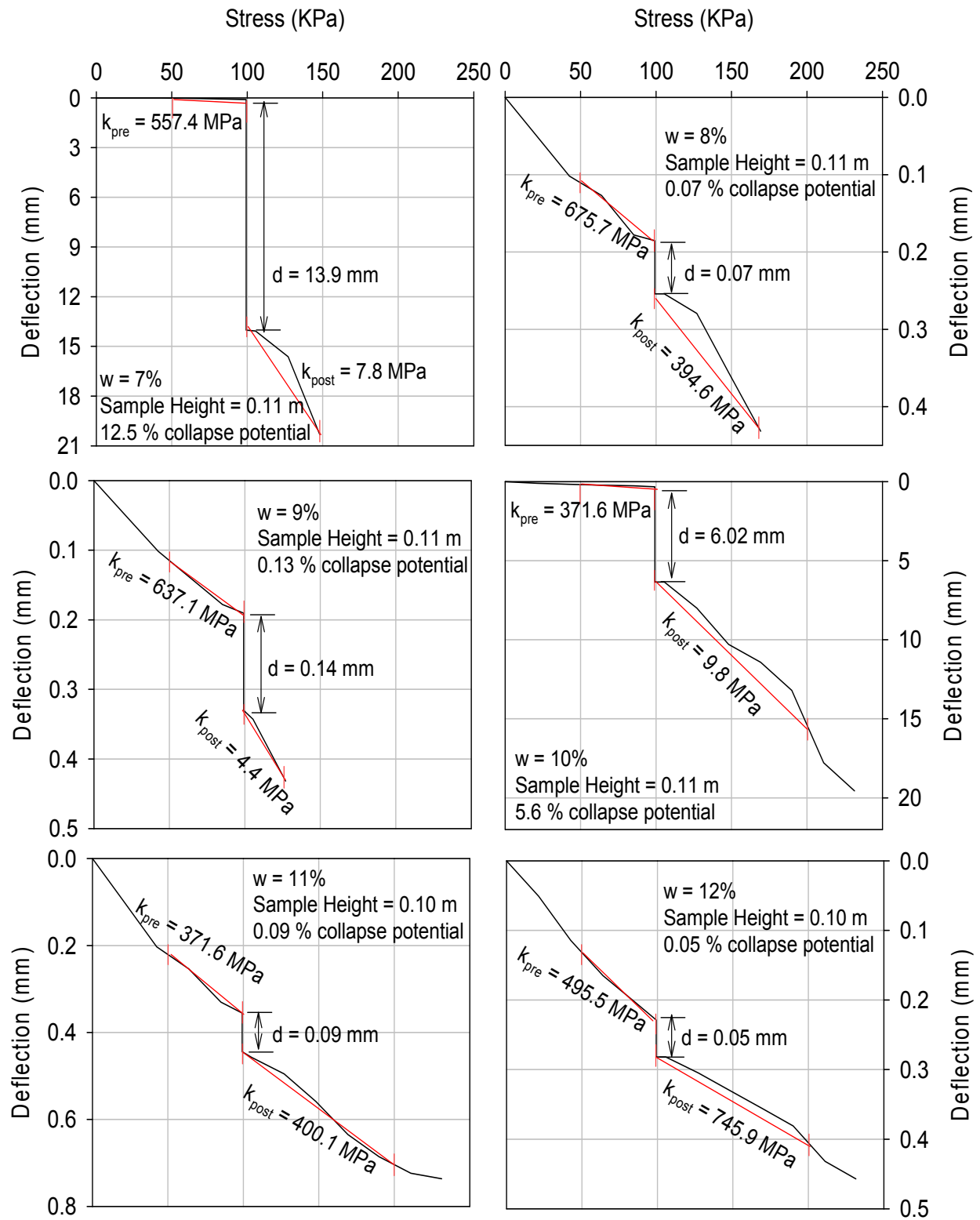


Figure 6.21. Collapse potential versus moisture contents – Marzane at Perryville Sand 2

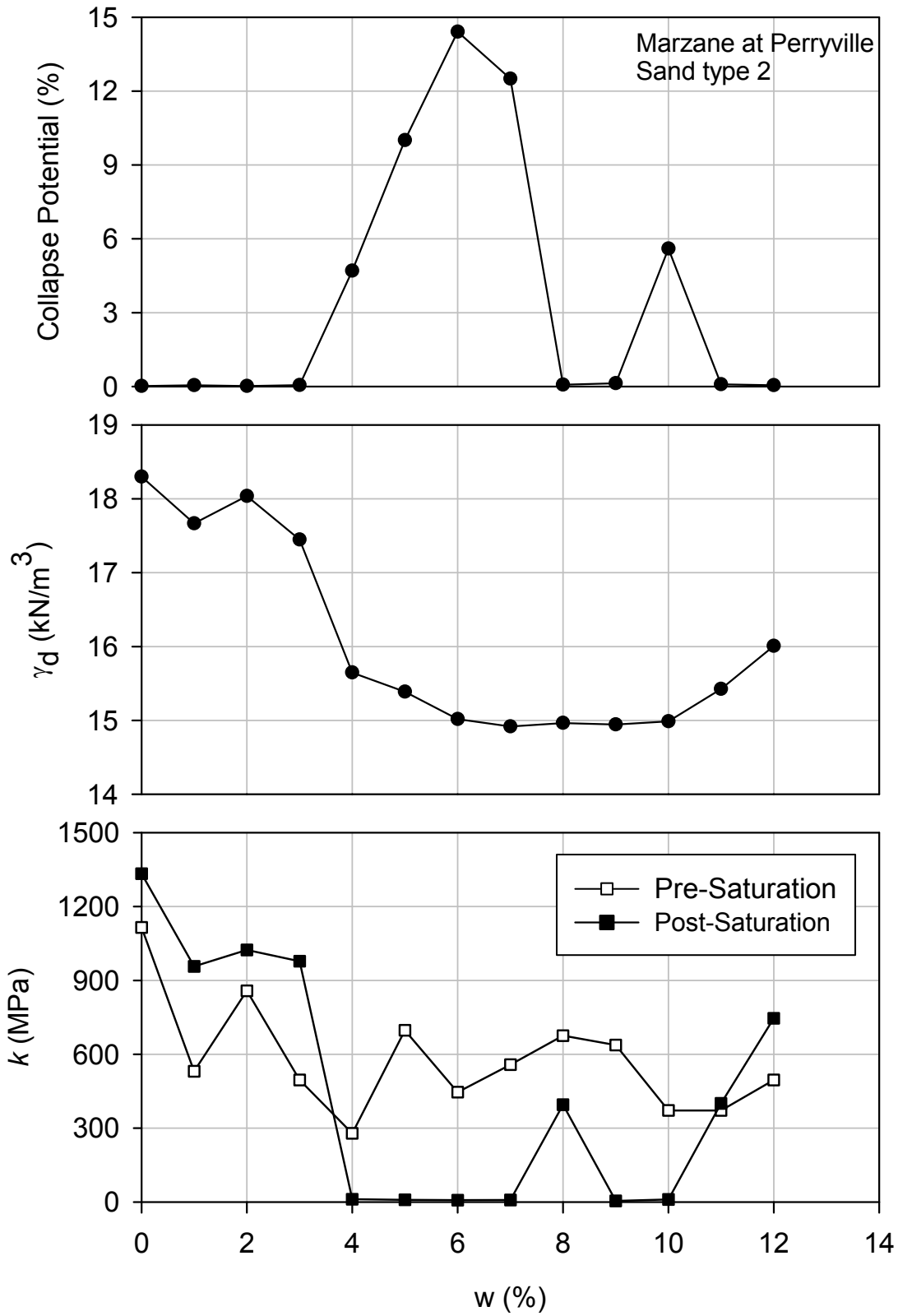


Figure 6.22. Collapse potential and dry unit weight versus moisture content for Marzane at Perryville Sand 2

6.4. Field Study Results

Seven new bridge sites were studied as part of the backfill investigation phase of this work. A brief description of site conditions at each bridge location and results of in situ testing at each bridge site are included in the following sections.

6.4.1. Bridge # 1: BUT-75-0660

Bridge # 1 is located at the I-75 and SR129 interchange in West Chester, Ohio. MSE walls were built on spread footing foundations on the northeast (NE) and southwest (SW) sides of the interchange to support the completed bridge, which will be a 116 ft single-span bridge constructed of pre-stressed concrete I-beams with semi-integral abutments (See Figure 6.23). Select granular material (USCS classification: SP) was used as backfill material for the MSE walls, and it was loosely placed, watered, and compacted using a hand-operated vibratory plate compactor within 6 ft of the MSE wall (See Figure 6.23d). The moisture content of the backfill material was reported to be about 4% before watering, about 8 to 11% after watering, and about 5% at about 10 minutes after watering.

In situ testing was conducted at four test locations. The testing included DCPs to depths of up to 6.6 ft from the ground surface and NG tests with a probe penetration depth of about 8 in. DCP tests were performed at distances of 0.5 ft, 1.5 ft, 3 ft, and 6 ft away from the SE and NW MSE walls (See Figure 6.23b). NG tests were performed at distances of about 1.5 ft, 6 ft, 12 ft and 18 ft away from the SE and NW walls.

The CBR values ranged from 0 to 10% at 0.5 ft to 3 ft away from the MSE walls for the upper 6.6 ft of the backfill, which indicates variability and relatively low strength of the backfill material (See Figure 6.24). The tests performed at 6 ft away from the MSE showed higher CBR values at depths greater than about 4.9 ft. Because the DCP tests were conducted at the fill stage, no compaction had occurred in the upper 1.6 ft of loose lift. Figure 6.25 shows CBR values with distance away from the MSE for selected depths. Results show an increase in CBR value with distance from the MSE with depth.

The dry density measurements of the backfill material at the SW wall and NE wall ranged from 95 to 105 pcf. The moisture content measurements were relatively constant for the measured locations, except for the tests conducted at 1.5 ft away the NE wall, which could be the result of that location having been watered just before the measurements were taken. Figure 6.26 shows the moisture contents, which ranged from 5 to 10% for the test locations.



a. location of south and north MSE walls



b. DCP test locations



c. watering of backfill prior to compaction



d. compaction of backfill next to the wall

Figure 6.23. Bridge #1 (BUT-75-0660) at I-75 and SR 129 interchange

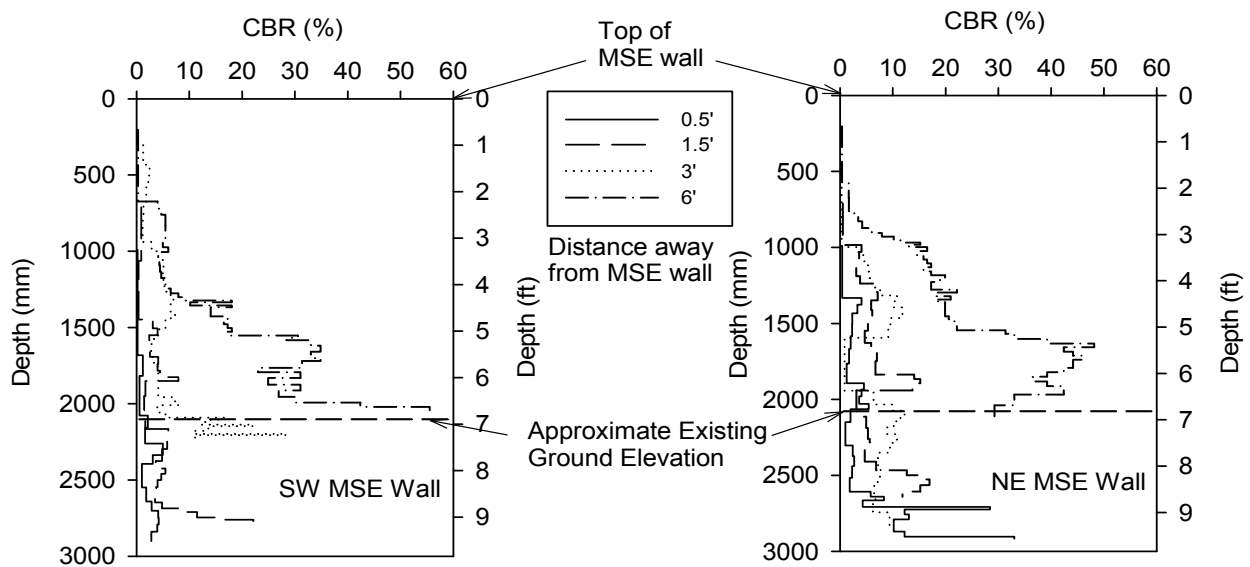


Figure 6.24. DCP-CBR profiles at selected distances away from the NE and SW MSE walls – Bridge #1

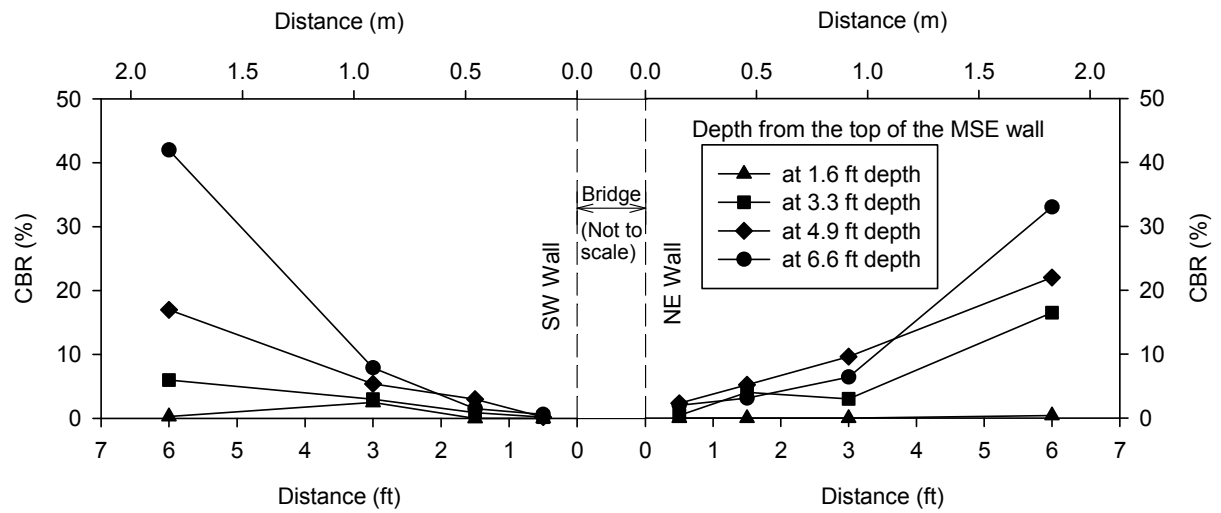


Figure 6.25. CBR at different depths from the top of the MSE wall at various distances away from the walls – Bridge #1

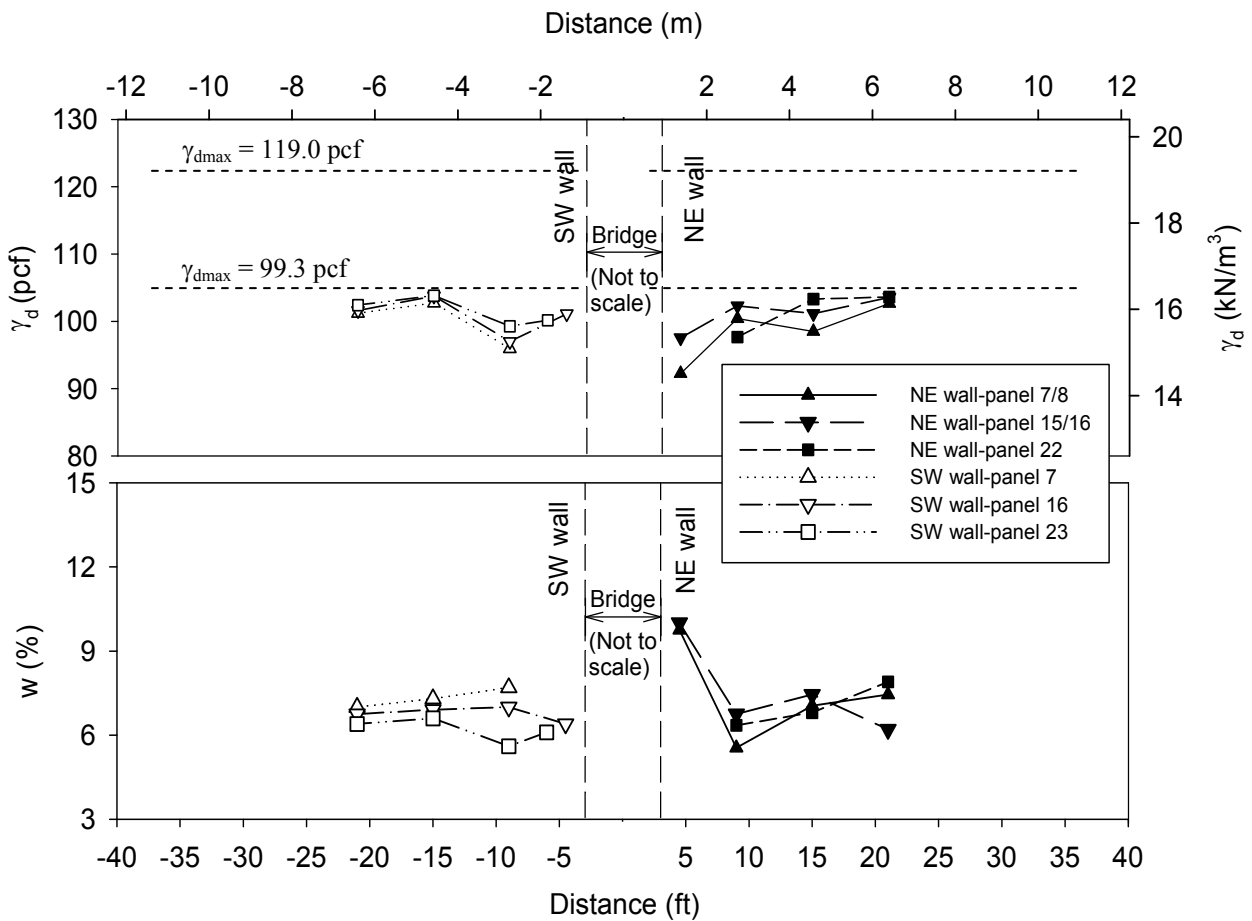


Figure 6.26. Moisture and dry density measurements at selected distances away from the MSE walls – Bridge #1

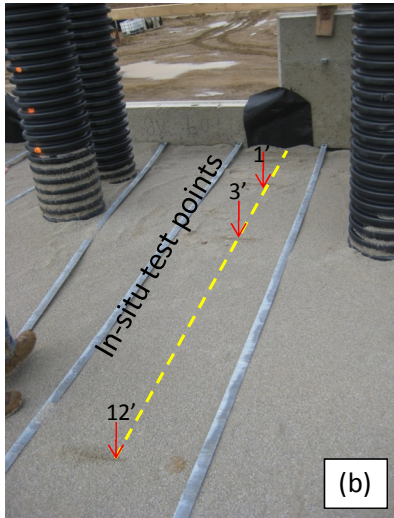
6.4.2. Bridge # 2: CL1-73-0985

Bridge # 2 is located near Wilmington, Ohio. The bridge structures are two semi-integral stub abutments, MSE walls with caps, column piers, and a 30 ft long approach slab. The abutments will support a 90.79 ft long span constructed of continuous pre-stressed concrete I-beams with a reinforced concrete deck. Because of the construction when testing was completed, only the south abutment was investigated. A CAT 5636 compactor was used to compact the area away from the piles, and a vibrating plate sled compactor was used within 6 ft around the piles. The backfill material used in this bridge was classified as SW-SM (USCS). Figure 6.27 shows an overview of the site, the in situ testing locations, and the compaction device used around piles of this bridge site.

DCP tests were performed at distances of 1 ft, 3 ft, and 12 ft away from the MSE wall. Between 1 and 3 ft away from the MSE wall, the CBR values did not change significantly; the test conducted at 12 ft away from the MSE wall showed a higher CBR value. The CBR values for the tests conducted at 1 ft and 3 ft away from the MSE wall ranged from 0.7 to 23% from the surface to 6.6 ft below the ground, and the DCP test conducted at 12 ft away from the MSE wall showed the CBR value ranged from 1.7 to 50%, which indicates a significant increase. DCP tests results are presented in Figure 6.28



a. Overview of the south MSE wall



b. DCP test locations



c. Vibratory plate compactor used for compaction of wall backfill

Figure 6.27. Bridge #2 (CL1-73-0985) at Wilmington

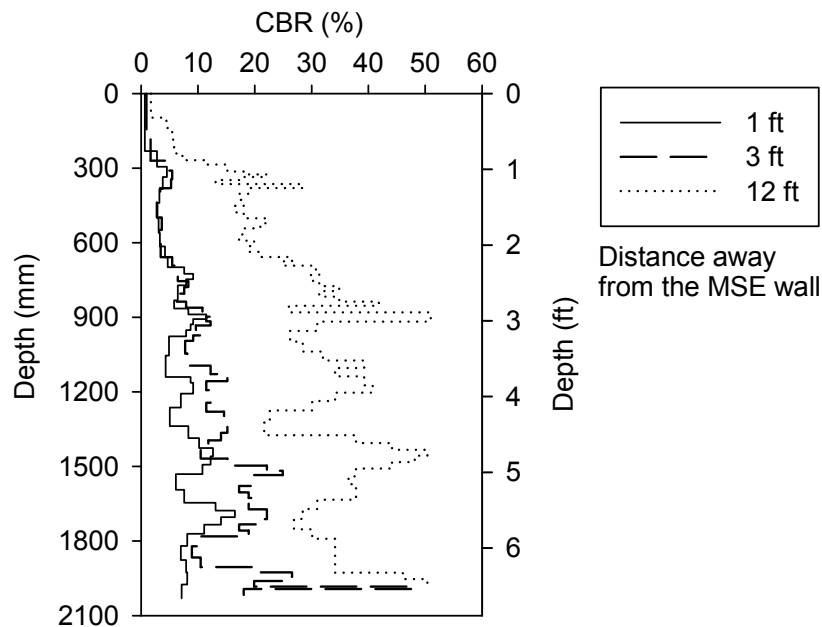


Figure 6.28. DCP-CBR profiles at selected distances away from the MSE wall – Bridge #2 (USCS: SW-SM)

6.4.3. Bridge # 3: MOT-75-1393

Bridge #3, in downtown Dayton, Ohio, is a curved girder bridge with a 30 x 45 ft approach slab. The bridge structures will consist of four-span composite welded curved steel plate girders on a cap and column pier, single column piers, and stub abutments behind MSE walls. A 2 ft thick layer of porous backfill (USCS: GP) was used behind the abutment underneath the approach

slab. A 1 ft thick layer of sub-base (GW) was placed on top of the select granular backfill (SW-SM).

In situ testing was conducted along the paving notch and the south wall (See Figure 6.29). DCP tests were conducted at distances of 1.5 ft, 3 ft, and 6 ft away from the paving notch and 0.5 ft, 1.5 ft, 3 ft, and 6 ft away from south wall. LWD tests were conducted at the same locations as the DCP tests and at three points behind the east end of the approach slab.

CBR values generally increased with distance away from the paving notch and the south wall (See Figure 6.30). There is a soft layer at about 3.28 ft below the surface along the tested lane perpendicular to the paving notch. The CBR values near the paving notch ranged from 1 to 15% at the distance from 1.5 to 3 ft away from the paving notch and, at 6 ft away from the paving notch, the CBR values ranged from 9 to 48%. The tests conducted along the lane perpendicular to the south wall returned CBR values that ranged from 0.2 to 20% within 3 ft of the wall and the CBR values for the test at 6 ft away the wall ranged from 5 to 35%.

Figure 6.31 shows the LWD modulus change at test locations away from the paving notch and the south wall. LWD tests were conducted at the same location as DCP tests, in general, and three tests were conducted at the location behind the end of the approach slab. For these test locations, the individual LWD modulus values ranged from 4,786 to 10,443 psi, and there was no significant difference between the test locations with sub-base or those without sub-base material. Figure 6.31 shows the modulus values with distance from the paving notch.

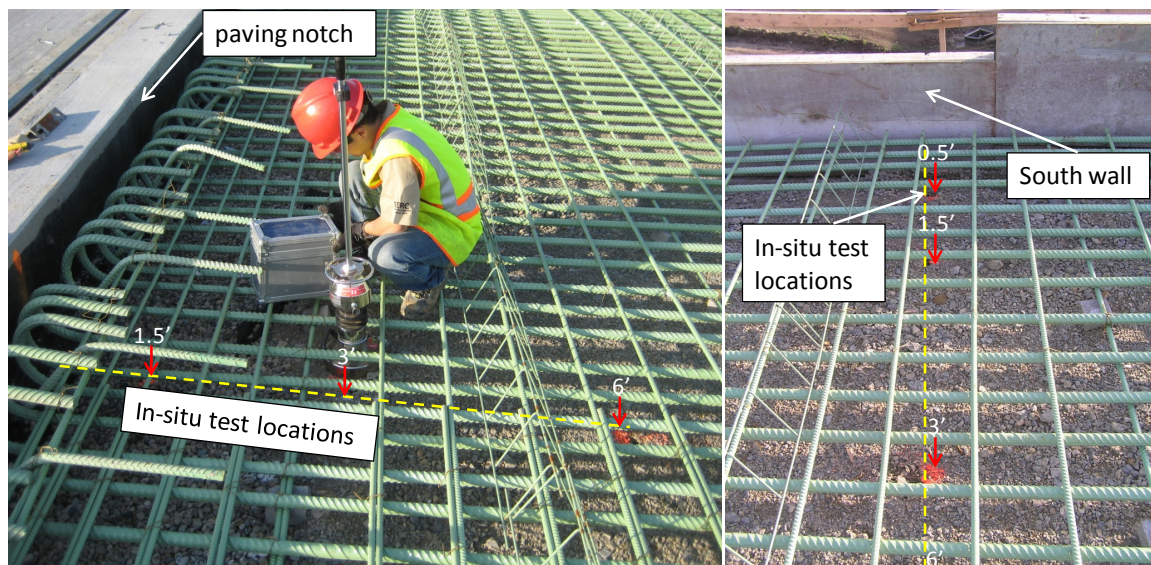


Figure 6.29. Bridge #3 (MOT-75-1393) at downtown Dayton

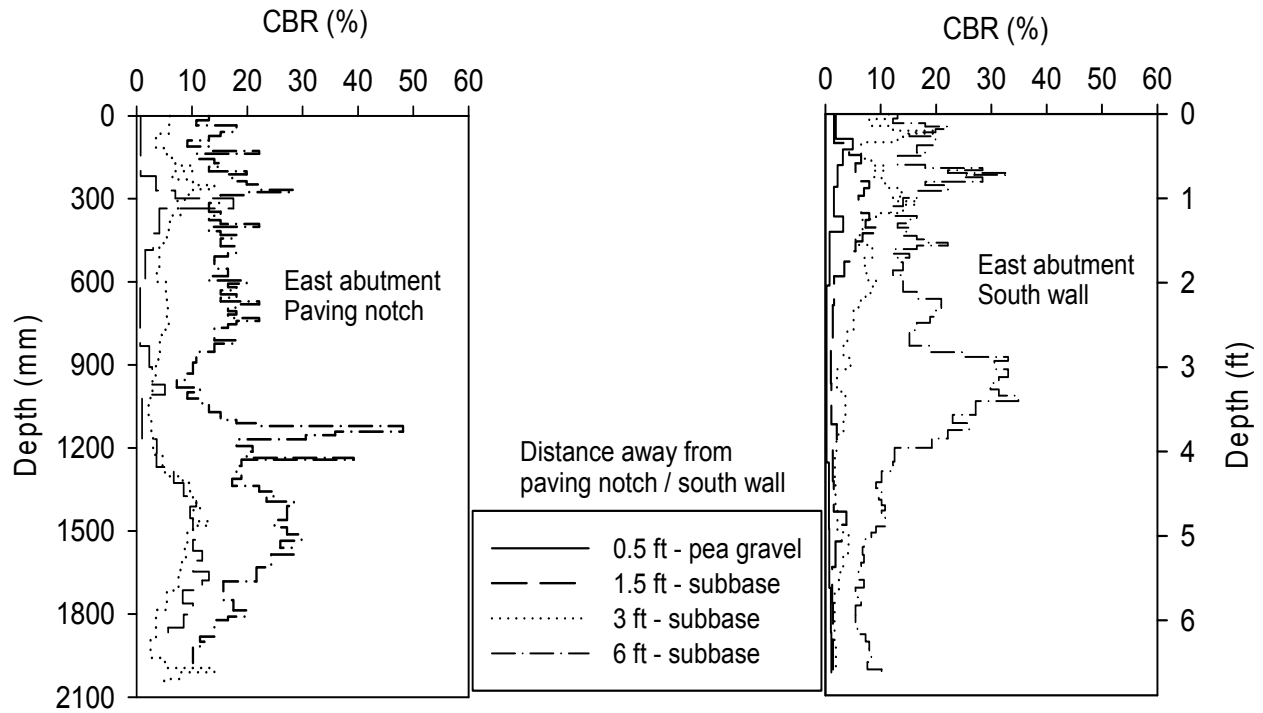


Figure 6.30. DCP-CBR profiles at selected distances away from south wall and paving notch at east abutment – Bridge #3

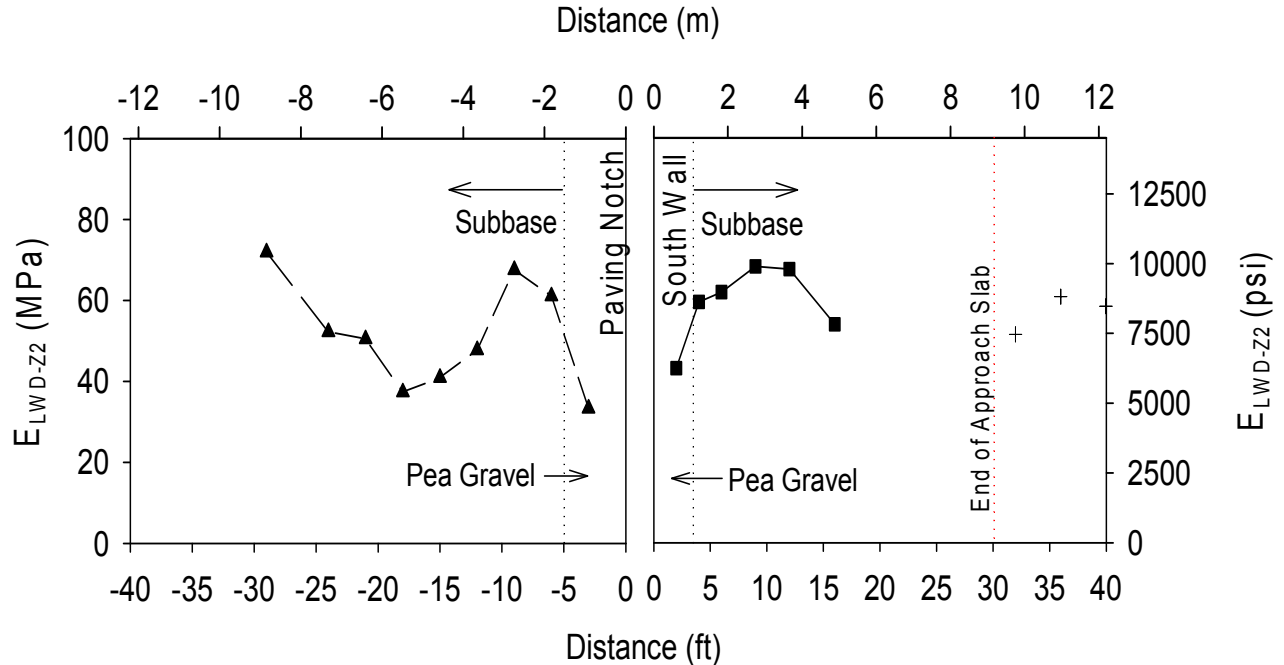


Figure 6.31. E_{LWD-Z2} measurements at selected distances away from paving notch and south wall on the east abutment – Bridge #3

6.4.4. Bridge # 4: FRA-670-0904B

Bridge # 4 is located at the Columbus, Ohio airport, near Johnstown Road. The bridge structure is 61.7 ft long, a single-span pre-stressed concrete I-beam bridge with reinforced composite deck on semi-integral abutments, supported by piles behind MSE walls. The approach slabs were modified 30 ft long sections. Aggregate base (USCS: GP) was the backfill material for the MSE wall.

DCP tests were conducted at four locations inside the approach slab at 1 ft, 3 ft, 6 ft, and 28 ft away from the paving notch and one test was conducted at 2 ft behind the end of the approach slab (See Figure 6.32). The CBR value generally increases with distance away from wall, and there is a stiff layer at a depth of 3.9 ft to 4.9 ft for the test location within 3 ft away from the wall (See Figure 6.33). The test conducted at 6 ft away from the paving notch indicates that the backfill started getting stiff from 1 ft below the surface and no significant changes of strength for the location near the end of the approach slab.

Because of the construction stage at Bridge #4, LWD tests were not conducted *inside* the approach slab region. LWD tests were conducted at five points *behind* the end of the approach slab and the tests results indicate modulus values from 7,252 to 9,427 psi (See Figure 6.34).

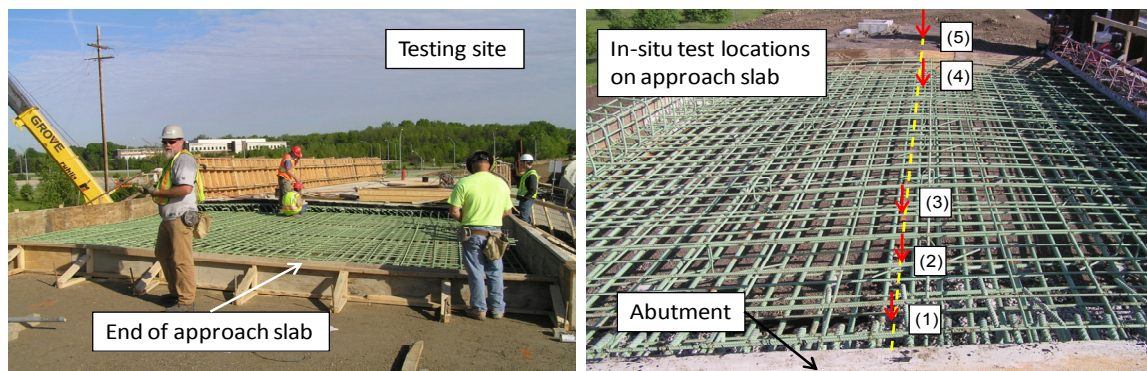


Figure 6.32. Bridge #4 (FRA-670-0904B) near the Columbus airport

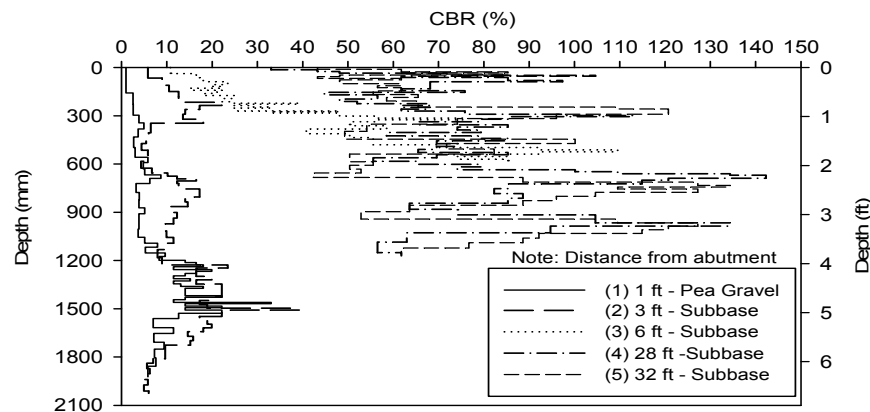


Figure 6.33. DCP-CBR profiles at selected distances away from the abutment – Bridge #4

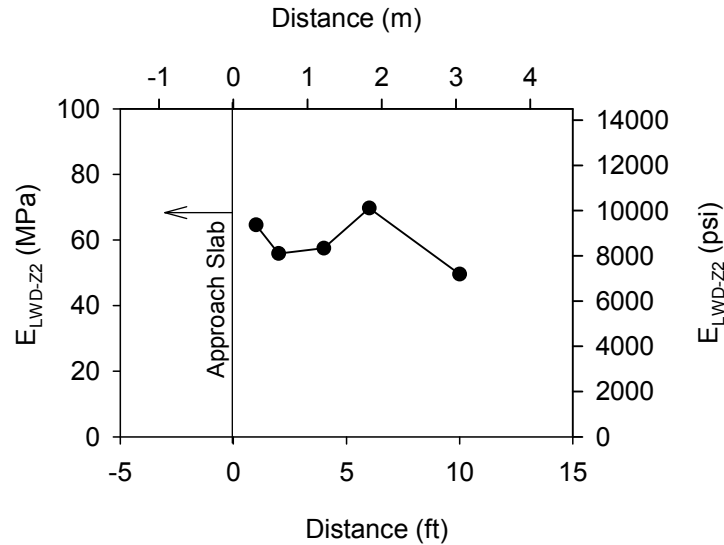


Figure 6.34. E_{LWD-Z2} at selected distances away from the approach slab – Bridge #4

6.4.5. Bridge # 5: LIC-37-1225L

Bridge # 5 is located at Licking 161 over Moots Run. The bridge consists of three-span composite pre-stressed concrete I-beams with cap, column piers, and semi-integral abutments with 25 ft long approach slabs. The backfill material used in the east abutment and west abutment were silty sand with gravel and silty gravel with sand, respectively. A layer of pea gravel was placed next to the abutment with 5 to 6 ft deep. The west abutment rests on hard rock shale and the east abutment rests on alluvium soils. Figure 6.35 provides the test location, site overview, and location of Bridge # 5.

Standard Proctor tests were conducted on the material sampled from both the east and west abutments. The maximum dry unit weights for the materials sampled from the east abutment and west abutment are 125.4 pcf and 124.1 pcf, respectively. The optimum moisture content for the material sampled from east abutment is 11.2% and 10.6% for the material sampled from near the west abutment. By comparing the in situ moisture-dry unit weight measurements and the standard Proctor test result, the moisture content of the backfill material in the field was close to the optimum moisture content. However, the dry unit weight was lower than the maximum dry unit weight from the standard Proctor test. Figure 6.36 provides the Standard Proctor test results and the in situ moisture-dry density measurement for the backfill material.

DCP tests were conducted at distances of 1 and 2 ft away from the abutment for the eastbound lane and then at 5 ft intervals to 35 ft away for the westbound center lane on the west abutment. For the east abutment, DCP tests were conducted at 1 ft away and then at intervals of 5 ft to 30 ft from the abutment. Figure 6.37 shows the DCP-CBR profiles for the tested locations. LWD tests were performed at the same locations as DCP tests, except for the tested points at 5 ft from the west abutment and at 1 ft from the east abutment. Moisture and dry density measurements were obtained using an NG at the same locations as the LWD tests. Figure 6.38 and Figure 6.39 provide the LWD tests and NG tests results, respectively.

The tests performed at the distances of 1 ft and 2 ft away the abutment provided similar CBR profiles along the west abutment eastbound lane. The CBR profiles for both west abutment and east abutment indicate the lowest strength occurred at 1 ft away from the abutment and then increase with distance away the abutment. LWD test results show that the modulus generally increases with distance away from wall and that the dry unit weight measurements also show a similar trend.

The moisture measurements near the west abutment indicate the lowest moisture content was next to the abutment and that, at further distances away from the abutment, the moisture content ranged from 9 to 12%. The results from the east abutment indicated the moisture content decreased with distance away from the abutment from 5 to 20 ft and within the range of 9 to 12%.



Figure 6.35. Bridge #5 (LIC-37-1225L) at Licking 161 over Moots Run

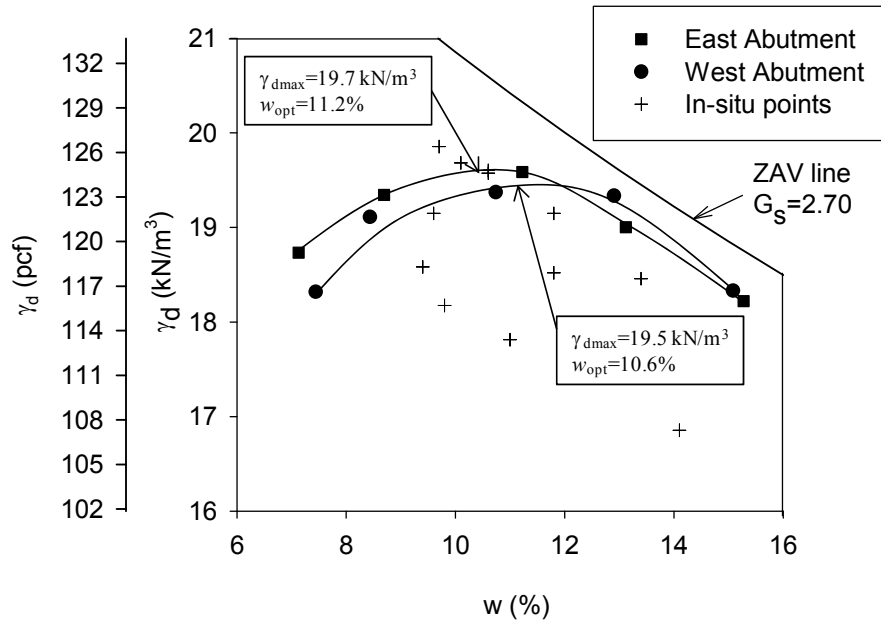


Figure 6.36. Proctor curve and field moisture and dry density measurement – Bridge #5

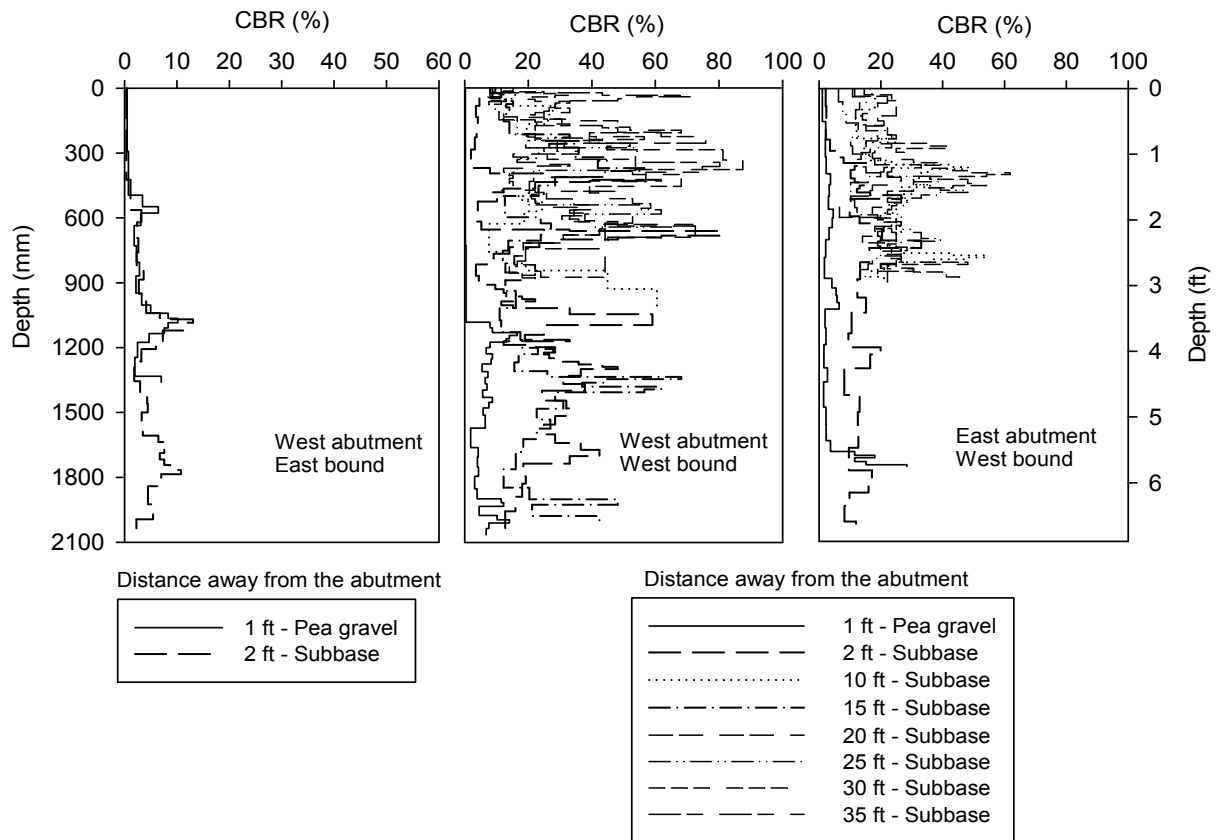


Figure 6.37. DCP-CBR profiles at test locations away from the east and west abutments – Bridge #5

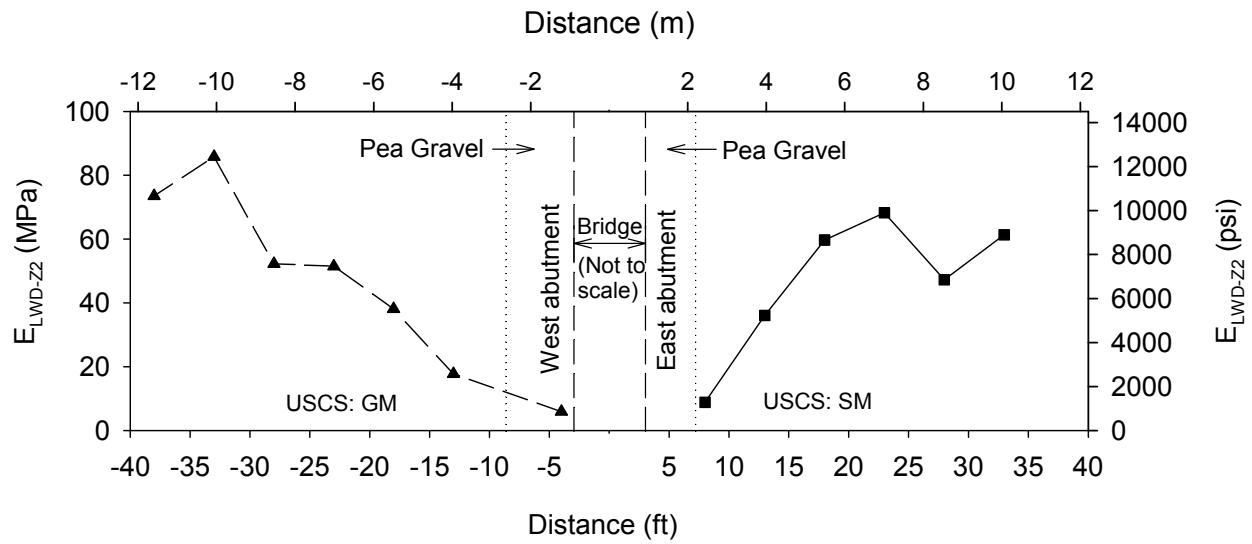


Figure 6.38. E_{LWD-Z2} measurements at test locations away from the east and west abutments – Bridge #5

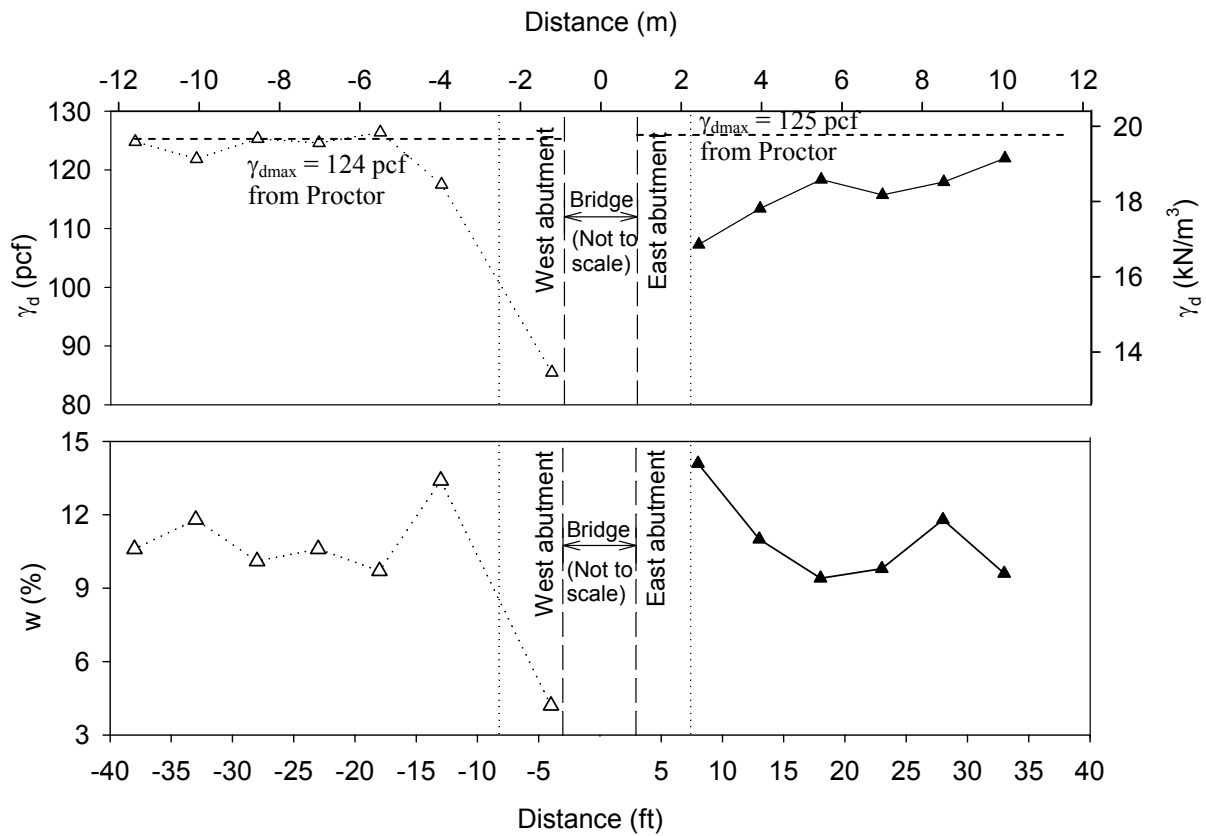


Figure 6.39. Moisture and dry density measurements at test locations away from the east and west abutments – Bridge #5

6.4.6. Bridge # 6: MED-71-0729

Bridge # 6 is located at the interchange of I-71 and I-76 on the section of I-71 over Greenwich Road in Medina County, Ohio. The structure for this project site was a single-span steel girder bridge with a reinforced concrete deck and semi-integral wall type abutments with a 30 ft long modified approach slab. There was an existing wall about 30 ft away from the new wall. A gap between the existing slab and the exposed fill material can be seen from the existing wall. Figure 6.40 shows the project site conditions and in situ test locations for this bridge study and the conducted in situ tests include DCP, LWD, and PLT along two testing lanes. The material from the site is classified as poorly graded gravel to silty gravel with sand per USCS.

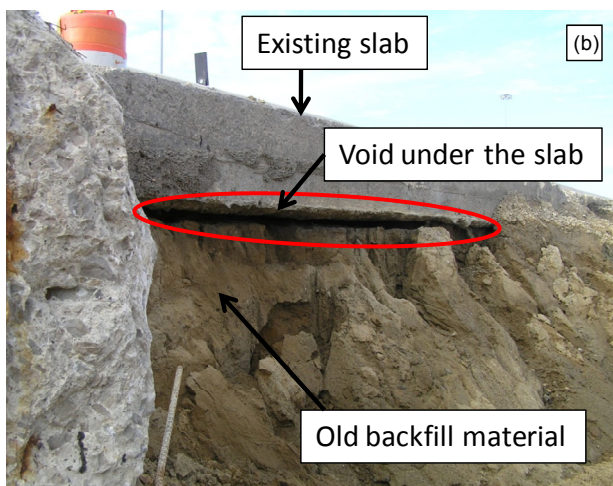
DCP tests were conducted on two testing lanes (west and east) at 1 ft away from the abutment and then at 5 ft intervals to 30 ft from the abutment. Figure 6.41 provides the DCP tests results in terms of CBR values for both west and east lanes. The CBR profile from the west lane shows that, at the same depth, CBRs increase with distance away from wall, but, with the distance of 25 to 30 ft away from the wall, the CBR values start to decrease. That may be caused by an existing wall near the end of the west lane. The DCP tests conducted on the east lane also give the similar conclusion.

LWD tests were performed on both the east and west testing lanes along the south abutment. Nine testing points were constructed on the east testing lane and thirteen points were tested along the west lane on the south abutment. Figure 6.42 shows that the E_{LWD-Z2} varies with distance away from the south abutment for both testing lanes. The LWD test results indicated that the modulus values at the middle part of the two testing lanes are higher than the E_{LWD} measured at the ends, and the modulus values from the two testing lanes show similar trends and the typical E_{LWD-Z2} range was 725 to 4,351 psi.

PLTs were conducted at three locations that parallel the new abutment within a distance of 16 ft from the abutment. At the third testing point, water was introduced at the surface of the soil, while maintaining a static stress of 58 psi, to evaluate the collapse potential of the backfill material. Figure 6.43 shows the PLT results and test setup. The PLT indicates minimal in situ collapse potential, because only 0.3 in. additional settlement was shown when the material was saturated and, that was less than the settlement during the loading stage.



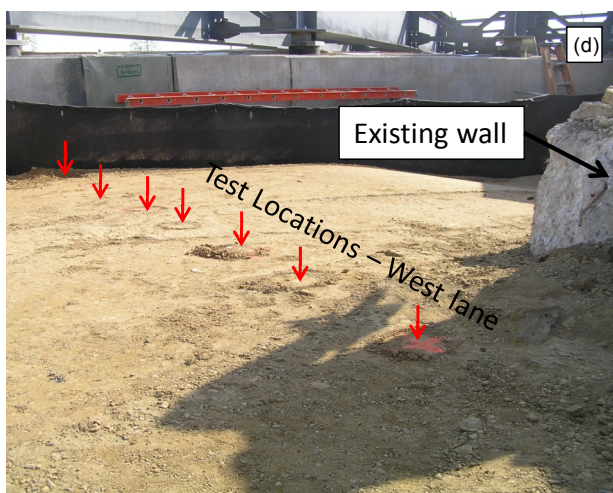
a. location of test site



b. void under the existing slab and old backfill material



c. in situ test locations – east lane



d. in situ test locations – west lane

Figure 6.40. Bridge #6 (MED-71-0729) at I-71 and I-76 interchange

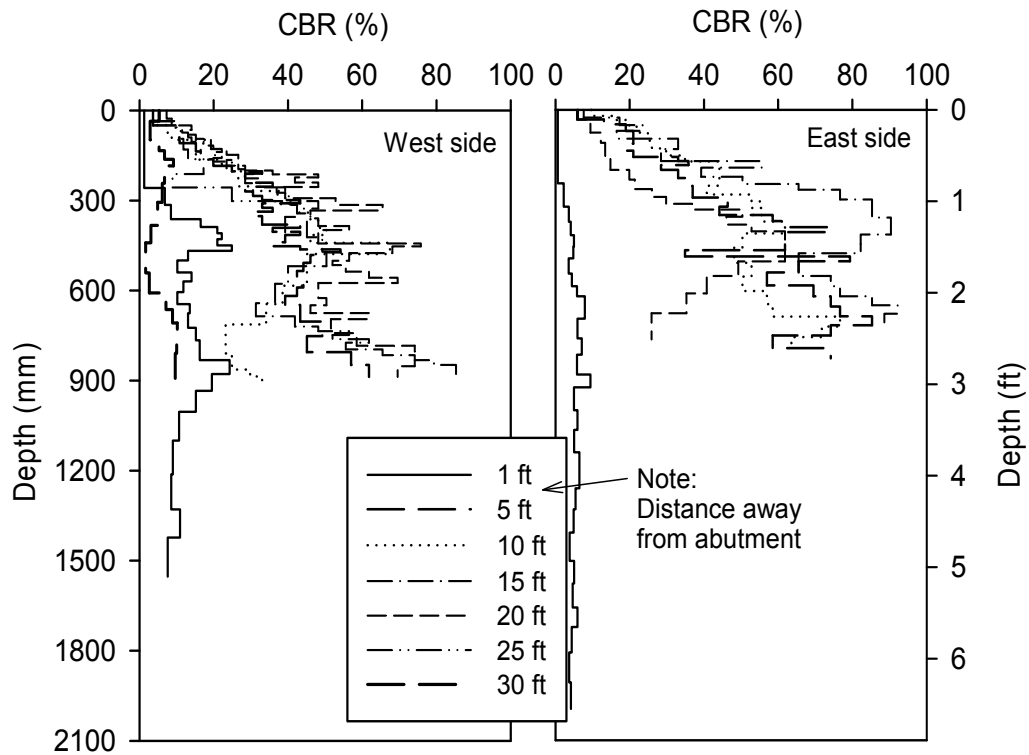


Figure 6.41. DCP-CBR profiles at test locations away from the south abutment for west and east lanes – Bridge #6 (USCS: GP-GM)

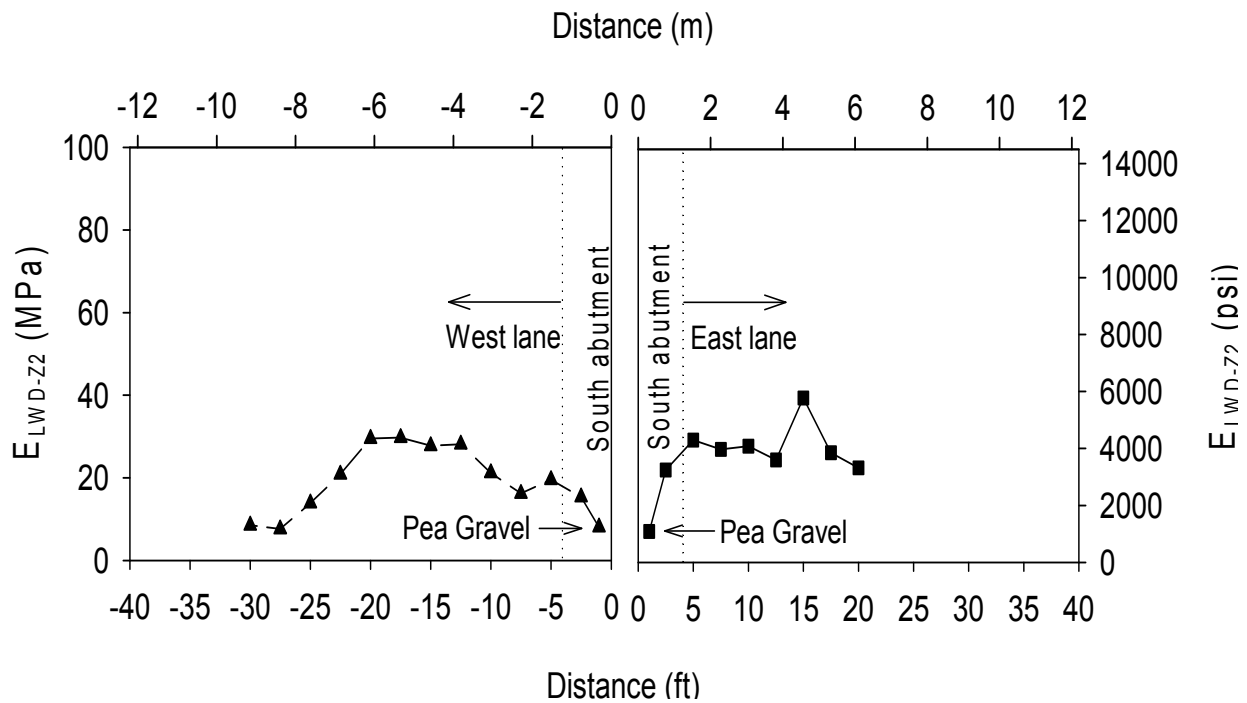


Figure 6.42. E_{LWD-Z2} at test locations away from the south abutment on east and west lanes – Bridge #6 (USCS: GP-GM)

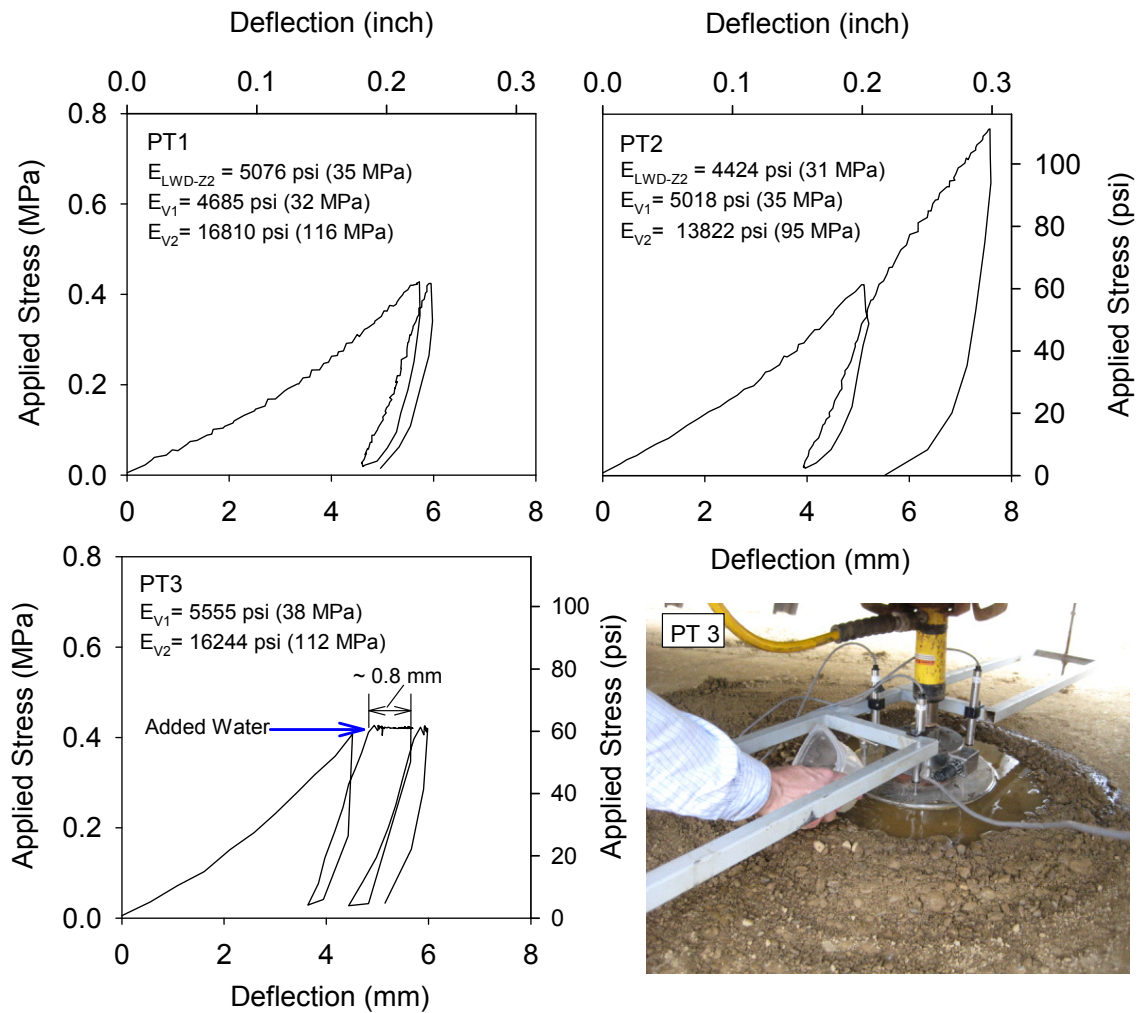


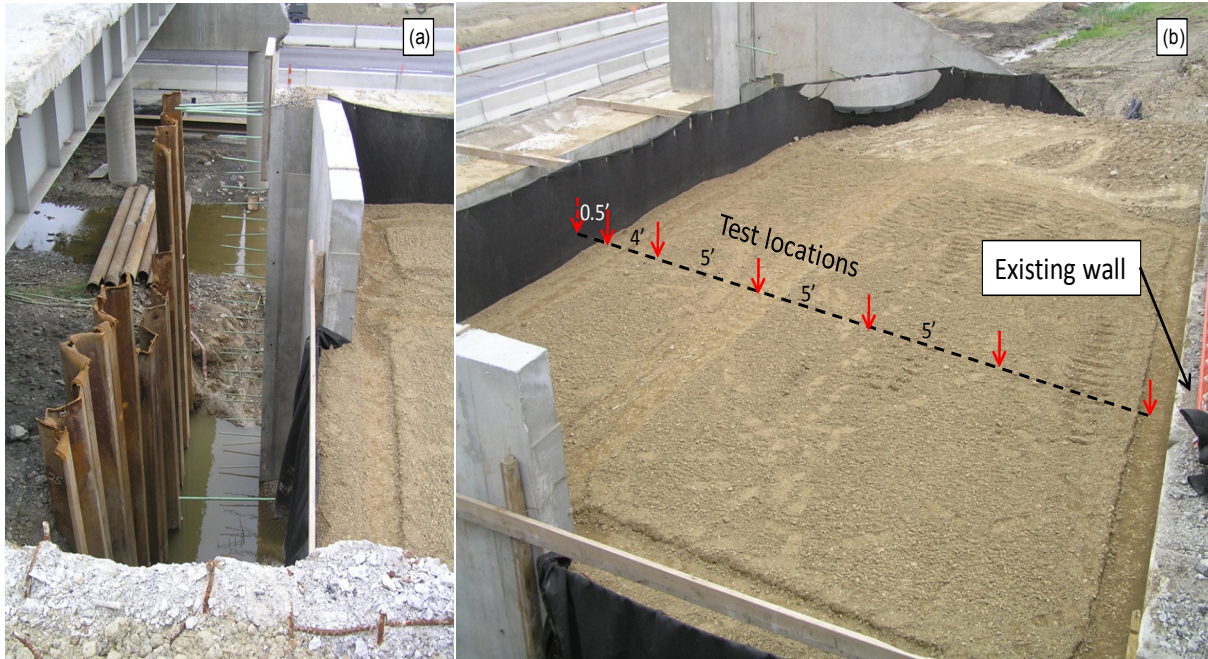
Figure 6.43. Stress-strain curves for static plate load tests – Bridge #6 (USCS: GP-GM)

6.4.7. Bridge # 7: MED-71-0750

Bridge #7 is located at the interchange of I-71 and I-76 over I-71 at Medina County, Ohio. The structure is a continuous steel girder bridge with a reinforced concrete deck on semi-integral abutments and cap and column piers. The 30 ft modified approach slabs were specified for this bridge. Figure 6.44 shows the in situ testing locations.

DCP tests were performed at 0.5 ft and 1 ft and then at 5 ft intervals to 20 ft away from the abutment. The tests results are provided in Figure 6.45 in terms of CBR. LWD tests were conducted near the DCP test locations; LWD test results are shown in Figure 6.46.

Based on the DCP-CBR profile, the measurements at 1 ft away from the wall indicated the lowest strength profile. The LWD test results indicate that modulus values for the backfill ranged from 725 to 5,076 psi.



a. site location

b. in situ test locations

Figure 6.44. Bridge #7 (MED-71-0750) at I-71 and I-76 interchange

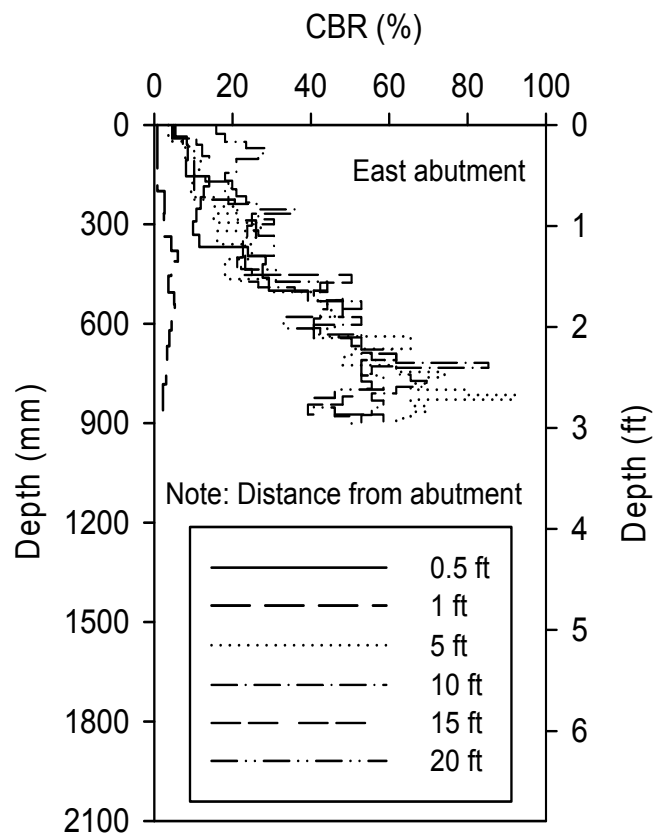


Figure 6.45. DCP-CBR profiles at test locations away from east abutment – Bridge #7

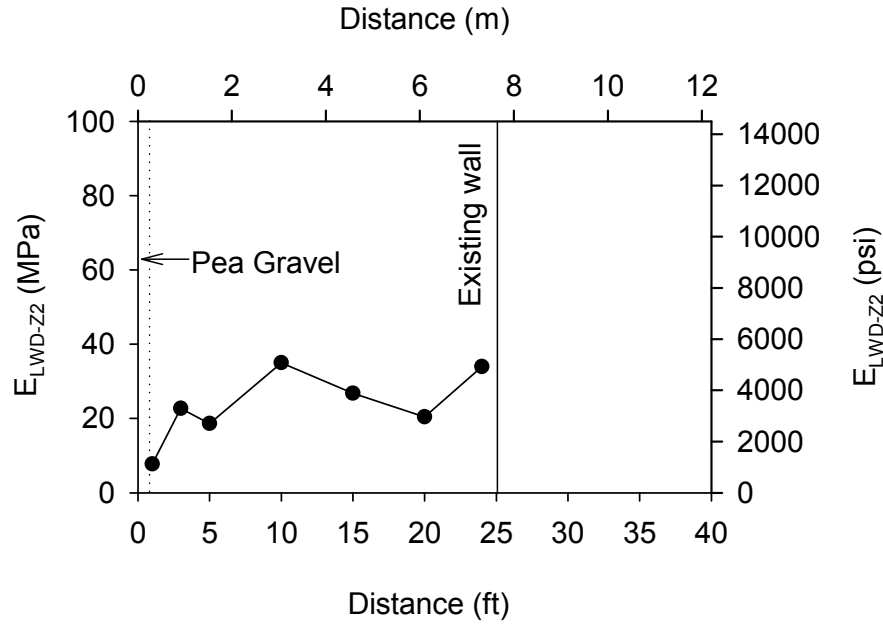


Figure 6.46. E_{LWD-Z2} at test locations away from east abutment – Bridge #7

6.5. Summary of Key Findings

The laboratory and in situ studies conducted for this project yielded these key findings:

- Laboratory tests demonstrated that sandy granular backfill is susceptible to collapse upon wetting and saturation if the material is compacted with moisture content near the bulking moisture content (about 3 to 6%). Collapse potential can be as high as 14%.
- In general, at each of the seven bridges in this study, the LWD test results and DCP-CBR profiles showed that the backfill materials within about 5 ft of the abutment or MSE wall were poorly compacted. Poorly compacted backfill materials in this region will provide less support to the approach slab and are more susceptible to post-construction compaction and void formation.

7. SUMMARY, CONCLUSIONS, AND RECOMMENDATIONS

7.1. Summary

The goal of this work was to provide ODOT with information to help improve the ride quality of their bridges. Bridge rideability can be influenced by a wide variety of factors that include design errors (such as miscalculation of beam camber), bridge construction errors (such as improper screed settings), and incompatible (and sometimes rapidly changing) stiffness characteristics on and around the bridge. Of these three, the incompatibility in stiffness characteristics is the source of most long-term bridge ride quality (and maintenance) problems.

Variability in stiffness characteristics generally result from: loss of backfill materials, poor construction practices of the supporting materials and joints (such as poor joint and drainage system installation and poor compaction of embankment materials), and settlement of embankment soils.

To achieve the project goals, the following activities were performed:

- Review of ODOT design and construction standards and specifications
 - Geotechnical
 - Structural
- Literature review
- Review and summary of current nationwide state-of-the-practice
 - Material selection
 - Structural details
 - Approach slab details
- Field investigation of the behavior and condition of in-service bridges
 - Global geometry measurement
 - Profiler and IRI testing (conducted by ODOT)
 - FWD testing (conducted by ODOT)
 - Live load testing
- Laboratory and field testing of bridge embankment materials
 - Laboratory
 - Grain-size analysis
 - Material classification according to AASHTO and USCS
 - Relative density testing
 - Standard Proctor testing
 - Collapse potential testing
 - Field
 - NG testing (moisture content and density)
 - DCP testing to determine the CBR
 - Zorn LWD and static PLTS to determine the material elastic modulus
- Compilation and comparison of collected information
- Development of recommendations

7.2. Conclusions/Findings

The following conclusions/findings were developed from the activities mentioned above and detailed in this report. The conclusions/findings have been loosely grouped into three categories: General, Structural, and Geotechnical/Drainage. In some cases, the categories overlap.

General

- ODOT appears to be an industry leader in the following:
 - Including ride quality as a part of a bridge construction contract
 - Performing the most current and state-of-the-art corrective actions solely to improve rideability, when no “failed” component exists
 - Performing research related to techniques for improving bridge rideability (when other state DOTs have performed similar/overlapping research, but with the focus on reducing maintenance issues and improving bridge life)

Structural

- The ODOT definition of and details for integral and semi-integral abutments appear to differ from most other states. Of most importance is the ODOT integral abutment details, which do not allow for full connectivity/stiffness compatibility (rotational and translational) of the superstructure and substructure. The advantages of using integral abutments are well documented (also in this report) and ODOT may not be fully realizing all the known benefits. Again, the current ODOT semi-integral abutment detail is different from those used by most states. Specifically, the ODOT semi-integral detail does not provide for any connectivity between the superstructure and substructure (only between individual beams of the superstructure).
- ODOT approach slabs are detailed to have a partial positive connection to the bridge substructure. With this detail, the substructure and approach slab translate together. Likewise, any rotation of the substructure (which is designed to be zero) would similarly rotate the approach slab. However, because the superstructure and substructure do not rotate together, a rotational discontinuity exists between the superstructure and approach slab. This means, any rotation of one element (or both) results in a rapid change in slope at the interface.
- ODOT approach slabs appear to be doubly reinforced. It is not clear, however, if the reinforcing details are sufficient (strength and/or stiffness) to allow the approach slab to bridge voids that may form below the slab.
- The ODOT Office of Pavement has standard details for the transition between the approach pavement and the mainline pavement. When asphalt pavement constitutes the mainline pavement, the asphalt is butted directly against the face of the approach slab. When concrete pavement is used, an asphalt pressure relief joint (4 ft of asphalt on a sleeper slab) is used. In either case, the only mechanism to accommodate temperature-induced expansion of the bridge and approach slab is compression of the asphalt. This compression generally results in upward bulging of the asphalt.
- Comparison of global geometric data and the IRI data indicate that some sources of poor ride quality are missed by one type of measurement. In other words, sometimes the global

geometric data indicate ride quality issues, sometimes the IRI data indicate ride quality issues, and in some cases both indicate problematic conditions.

- Highly-variable FWD test results were found. Of interest was the fact that, very near the bridge, FWD tests indicated a very stiff system (as expected). Moving away from the bridge, system stiffness decreases immediately and is thereafter highly variable. This study also found that a reliable correlation between fill depth and FWD test results did not exist. This may indicate that the “quality” of material installation, rather than the amount of material installation, may be the greatest influence on vertical stiffness.
- Under live load, ODOT bridge abutments do not appear to be rotating under live loads. Given the standard integral and semi-integral abutment details, this is not surprising. Conversely, the study found the abutments do translate under live loads. If the abutment backfill materials and their installation were not designed correctly, this could lead to the formation of voids behind the backwall.
- Differential settlement was observed at most bridge sites. However, the study found a lack of consistency in the location of the differential settlement, indicating there may be multiple sources of ride quality issues.

Geotechnical/Drainage

- Gradation results indicate that the granular backfill materials being placed below the approach slab and around the abutment back walls have bulking moisture content of about 6%. Sandy granular backfill materials placed at the bulking moisture content can experience collapse under load. The collapse potential for the granular backfill materials tested in this study ranged from almost zero to more than 10%. Without exception, bulking moisture content should be avoided during construction. Field-controlling the moisture content to avoid the bulking range can mitigate the potential for collapse and therefore eliminate one behavior known to impact bridge ride quality. Incremental wetting of the material in situ during construction can be effective at reducing post-construction collapse.
- According to the LWD tests results and DCP-CBR profiles from all investigated bridges, the backfill materials placed within about 5 ft of the abutment back wall are poorly compacted. This is due to limiting the compaction effort in this zone next to the abutment wall. Specific observations for some of the bridge sites are as follows:
 - Based on field moisture and dry unit weight measurements for Bridge # 1, the backfill material was placed at the bulking moisture content, and the dry unit weight was close to the minimum dry unit weight obtained from laboratory testing.
 - The moisture content and dry unit weight measurement for Bridge # 5 show that the backfill material had the moisture content within the optimum moisture content, but the dry unit weight was lower than the maximum dry unit weight.
 - Field collapse potential tests indicate minimal in situ collapse of the backfill material for Bridge # 6.

7.3. Recommendations

The following recommendations were developed from the activities mentioned above and detailed in previous pages. The recommendations have been loosely grouped into four categories: General, Structural, Geotechnical/Drainage, and Bump Identification Metrics and Troubleshooting. Note that categories overlap.

General

- In addition to profiling bridges, it is recommended that ODOT begin a program of measuring the gross vertical geometry of all bridges. The combination of gross vertical geometry and profiler data provide information that can help identify sources of ride quality degradation.
- It is recommended that all new bridges be profiled and have the gross vertical geometry measured immediately after construction. These measurements provide an important baseline for assessing future performance (further, they can be used as part of a recommended construction specification that is subsequently discussed).
- It is recommended that all bridges be profiled and have the gross vertical geometry measured at least every 10 years and when rideability is noted to have started to degrade. The gross vertical geometry and IRI data should be compared with previous measurements and examined for changes. Furthermore, the approach pavement slope should be calculated and examined for changes. When the slope reaches a specified value (suggested to be on the order of 1/200) corrective actions may be needed. Similarly, ODOT should begin also calculating the Bridge Approach Performance Index so that it may be examined for changes over time.
- A specification that ensures an acceptable ride quality at the time of construction should be created and adopted by ODOT. Once created, it is suggested that an annual review of this specification be completed for a minimum of five years to ensure that ODOT is achieving the desired results (acceptable ride quality at a reasonable cost). It is suggested that the specification should contain two parts:
 - A maximum global roughness
 - A maximum local roughness
- On structures where unusual/unproven construction practices are required (or requested by the contractor), the bridge deck should be constructed with a minimum of 1/2 in. of additional sacrificial thickness, such that planned, blanket grinding can, without question, occur, unless deemed unnecessary. This sacrificial thickness will give ODOT the flexibility to correct structures that unexpectedly have poor ride characteristics.

Structural

- Improve the stiffness compatibility between the bridge superstructure, substructure, approach slab, and supporting materials:
 - Follow the geotechnical recommendations below.
 - Use integral abutments whenever possible and revise the integral abutment details such that the superstructure and piles are rigidly connected (so they rotate and translate as a unit). Note that some modification to the pile design process may be

required to ensure that the substructure is not overstressed. It is felt that, by changing the integral abutment details, the following changes in behavior will occur: (1) reduction in the amount of superstructure and substructure rotation, (2) reduction in the total temperature-induced lateral displacement of the substructure, and (3) reduction in the interaction of soil-structure interaction, which will reduce the possibility of developing voids in the surrounding soil. It is generally believed that integral abutments (as defined by the research team) are easier to construct (leaving less room for alignment problems), have fewer moving components, and have greater structural redundancy. It is recommended that ODOT consider using integral abutment details similar to those shown in Figure 4.7 (Iowa DOT) and/or Figure 4.57 (TDOT). Although a variety of different details are used by different states, the reviewed details were evaluated based on the following criteria: (1) constructability, (2) history of successful use, (3) owner reported problems, (4) record of improving designs through research, and (5) compatibility with other recommendations made in this report.

- Most literature indicates that using an angle bar between the substructure and approach slab is the desired connection (to allow them to rotate independently). However, given the ODOT desire for high ride quality, it may be worth trying a fully integral slab to bridge connection detail that ensures that the substructure, superstructure, and approach slab rotate as a unit. If such a detail is developed and tried, the approach slab should be designed to have sufficient strength so that no top-of-slab cracking will result. It is recommended that ODOT consider using a detail that combines the details shown in Figure 4.6 (the horizontal bar, except that it should be moved upward to resist negative moment bending) and Figure 4.27 (the angled bar). As no literature indicates that the proposed detail will perform better than other current details, the authors are making this recommendation based on their experience with similar structures. It may be advisable for ODOT to adopt such a detail on an experimental basis. It is felt that adopting such a detail will reduce the potential for rapid grade changes between the approach slab and bridge.
- Although published literature does not provide enough data to indicate if the use of sleeper slabs improves rideability, it is the researchers' recommendation that, regardless of the mainline approach type, support the approach slab on a sleeper slab. It is recommended that ODOT consider using a detail similar to those shown in Figure 4.59 (TDOT). The selection of the detail shown in Figure 4.59 considered the following criteria: (1) constructability, (2) compatibility with other recommendations made in this report, and (3) history of successful use. The use of a sleeper slab is intended to improve the support structure of the approach slab. It is widely accepted that proper material compaction in this area is critical to the rideability of the bridge. It is also widely accepted that getting high quality material installation in this area sometimes proves challenging. It is felt that the use of a sleeper slab reduces the impact that material compaction may have on bridge rideability.
- Minimize the frictional resistance between the approach slab and supporting materials by casting the slab on a low-friction material, such as polyethylene sheeting. The use of a

friction-reducing material will help to reduce the forces induced on the bridge superstructure and approach slab-to-bridge connection.

- Strive to limit bridge skew to 30 degrees to minimize the magnitude and lateral eccentricity of the longitudinal forces.
- Design the approach slab with sufficient strength to bridge settlement extending from the bridge abutment to the recommended sleeper slab. Further consider designing the approach slab with stiffness sufficient to minimize any deflection with such settlement.
- Replace the current ODOT approach slab to mainline pavement joint detail with an expansion joint that is sized to accommodate the expected bridge and approach slab expansion and contraction. It is recommended that ODOT consider using either the doweled expansion joint shown in Figure 4.9 (with one side of dowel in the sleeper slab as needed) or the appropriate detail, considering the material types, in Figure 4.59. It is felt that the current ODOT approach slab, because it uses a viscoelastic material, creates a bump at each bridge end. The use of a detail that does not rely upon such a material is anticipated to improve bridge rideability.
- Actively maintain the recommended expansion joint to prevent the development of high stresses in the approach slab and bridge. Such maintenance activities will ensure that the bridge is free to expand and contract with temperature variations.

Geotechnical/Drainage

Earth materials used to support the bridge approach pavement system would ideally provide support with no differential movement relative to the bridge superstructure. The recommendations summarized below highlight alternatives to traditional practice that will help to mitigate observed differential movements. The focus of these recommendations is on the backfill materials placed behind the back wall and under the approach pavements. Some of the concepts are presented as possible alternatives, but would need to be evaluated in the field on an experimental basis to document impact on performance. Furthermore, these recommendations should be implemented with consideration of the bridge superstructure design, in that the superstructure design may overcome some of the deficiencies in the backfill materials (sleeper slab on foundation to bridge settlement in backfill, for example).

The primary focus of the recommendations is to reduce the potential for differential settlement of the backfill through improved compaction, reduced erosion, and/or use of alternative materials. Reducing differential settlement will increase the longevity of the approach pavement and reduce roughness. Table 7.1 highlights possible specification deficiencies, the changes suggested, and impacts.

- Develop a lab test protocol to determine the bulking moisture content for granular backfill materials and establish a practice to field-control the moisture content to avoid bulking moisture contents. Compaction curves for cohesionless sands readily show bulking in the range of 3 to 5% moisture content.
- Consider use of alternative backfill materials, such as geosynthetic-reinforced soil, geofoam, or flowable fill, as an alternative to collapsible backfill.
- Improve compaction effort within 5 ft of the abutment backfill using thin lifts with a light vibratory compactor. If concerns exist due to compaction equipment imposing high

lateral stresses next to the wall, instrument a wall (or walls of different configurations) to monitor stress development and movement during compaction and during service loading to conclusively determine the impact of compaction loading. In general, vibratory compactors should be used to compact granular backfill materials.

- Water drainage needs to be an integral part of the bridge and embankment design. The bridge and embankment need to be detailed to drain water away from the bridge deck, joints, and embankment without causing erosion or changes in the soil properties. The following are recommended drainage details:
 - Full-width approach slabs should be used and have curbs or raised parapets to prevent deck drainage from eroding shoulder support. If a future asphalt overlay is a possibility, the curbs should be built high enough to compensate for the overlay.
 - Provide a tiled drainage outlet near the approach slab to pavement joint to prevent water from the bridge flowing onto the embankment.
 - Provide surface drainage channels on the embankments with erosion control cloth, erosion control mat, or rock to prevent pavement runoff from eroding the embankment. The water runoff management system should be designed such that water is directed to the channels.
 - Place drainage tile in the embankment that has adequate crushing resistance with respect to the depth of soil placed above the tile.
 - Place concrete gutters at the top of MSE walls and under bridges to direct water away from the embankment to prevent erosion of the embankment materials.
 - Place weep holes in the bridge deck near the approach joint to allow water to be drained prior to reaching the joint.
 - If water infiltrates the joints (bridge-to-approach, approach-to pavement), provide a drainage path for the water to escape the joint.
- Table 7.1 summarizes a review of geotechnical-related specifications, as requested, that relate to geotechnical and earthwork construction and testing aspects of bridge approaches. For each specification, a brief statement is provided to highlight possible changes or additions to the specifications. Recommendations are based on the field results and primarily focus on backfill material selection, placement, compaction, and drainage.
- Figures 7.1 through 7.4 provide some alternative backfill options that could be implemented on a research basis to evaluate performance changes in the approach slab. The options cover the following alternatives:
 - Use porous backfill behind the abutment in lieu of granular backfill.
 - Placement of geotextile-reinforcing layers to the granular backfill.
 - Use a geocomposite vertical drainage system behind the abutment.
 - Use a layer of tire chips behind the abutment as an elastic/resilient and drainage fill material.

Table 7.1. Summary of geotechnical-related specifications reviewed and suggestions for future specification updates

Manual ID	Specification ID	Specification Name	Page #	Key Notes	Suggestions for future specification changes/additions
Construction Inspection Manual of Procedures (Columbus, Ohio 2006)	201	Clearing and Grubbing	93	Use all suitable excavation material in the work. Alternatively, legally use, burn, or dispose of all material according to 105.16 and 105.17. Backfill of cavity created by removal of existing bridge per 503.09	Section 503.0-9 was not included in the Manual.
	203	Roadway Excavation and Embankment	137	If pavement is to remain smooth and stable during years of service under traffic, the earthwork on which it is built must be stable and must furnish uniform support	Consider adding a schematic/notes showing an approach pavement backfill highlighting proper compaction within 5 ft of back wall. For compacting granular materials susceptible to collapse upon wetting, a section could be added to focus on avoiding bulking moisture contents
	703.1 6.C	Granular Embankment Material Types	163	Six different gradations or types are available for use in construction	No changes suggested
	203.0 6.A 203.0 6.B	Soil and Granular Embankment/Shale	189	Use a maximum lift thickness of 8" for soil and granular embankment. Soil compaction acceptance is based on the proctor testing	Consider using relative density test for granular material by following ASTM 4253 and ASTM 4254.
	518	Drainage of Structures	649	Porous backfill is No.57 size gradation. It must be compacted. Even rounded No. 57 gravel is not self compacting	No changes suggested
	526	Approach Slabs	663	Materials (the concrete used to construct the approach is the same class as the bridge deck and should be placed using the same specifications as the bridge deck concrete) / Setting Grades (the final grade of the approach slab can be established by using a string line)	No changes suggested
	SS-840	Mechanically Stabilized Earth (MSE) Walls	883	The granular embankment materials have special requirements that are not normally associated with granular material in other items of work	On p. 927 it states that material is to be compacted 3% below optimum moisture content. It is possible that the material will be placed within bulking moisture content range. Suggest changes to avoid bulking moisture content range. Also consider incremental flooding after compaction.
Construction Inspection Manual of Procedures (Columbus, Ohio 2008)	203.0 2 R.	Suitable Materials	88	All suitable materials are restricted in 203.03. Furnish soil or embankment material conforming to 703.16, when Item 203 Embankment is specified.	Consider adding a note under section B to avoid placing granular material within the bulking moisture content range
	203.0 6	Spreading and Compacting	92	Spread all embankment material, except for rock in 203.06C and RPCC in 203.06D, in successive horizontal loose lifts, not to exceed 8" in thickness.	No changes suggested
	203.0 7	Compaction and Moisture Requirements	94	A. moisture Controls; B. Compaction Requirements. Table 203.07-1	No changes suggested
	304	Aggregate Base	157	304.02 Materials. Furnish materials conforming to 703.17. / 304.03 Prior to spreading / 304.04 Spreading / 304.05 Compaction	Consider adding note for compacting granular materials susceptible to collapse upon wetting, a section could be added to focus on the need to compaction to prevent collapse.
	415.1 2	Surface Smoothness	258	Ensure pavement surface variations do not exceed 1/8" in a 10' length of pavement. For ramp pavements and for those pavements with curvature greater than 8 degrees, or with grades exceeding 6%, ensure the surface variations do not exceed 1/4" in 10'.	No changes suggested
	503.0 8	Backfill	287	Use backfill embankment materials conforming to 203.02.R, except behind the abutments below the approach slabs use material conforming to Item 203 granular	Refers to other sections

				material type B. In bridge abutment areas compact backfill material to meet the compaction requirements in 203.07. Elsewhere, compact backfill material to 95% of the maximum laboratory dry density.	
	516	Expansion and Contraction Joints Joint Sealers And Bearing Devices	393	Ensure the expansion joints are completely open for the dimension specified for their full length. / Join Sealers Apply joint sealer with a minimum depth of 1" at its thinnest section.	No changes suggested
	518.05	Porous Backfill	398	When porous backfill not shown on the plans place at least 18" thick behind the full length of abutments, wing walls, and retaining walls. Place sufficient coarse aggregate or other material adjacent to, but not more than 6" below, the bottom of the weep hole to retain the porous backfill.	No changes suggested
	526	Approach Slabs	417	Do not allow forms to vary more than 1/8" from a 10' straightedge. Furnish reinforcing steel and place it in the position shown on the standard construction drawing and firmly secure the steel during placing and setting of the concrete.	No changes suggested
	603.11	Placement and Compaction Requirements	440	Place soil, granular embankment, or structure backfill type 1 or 2 in lift not to exceed 8". / For soil embankment, compact each lift until 96% of AASHTO T 99 is achieved. / Place structure type 3 in layers not to exceed 12" loose depth. Vibrate, tamp, or compact to approximately 85% of the original layer thickness.	Section C provides a provision for use of flooding to aid compaction. This approach would apply for reducing post construction collapse potential.
	703.16	Suitable Materials for Embankment Construction	694	Natural soil, natural granular material, granular material types, slag material, brick, shale, rock, random material, RACP, RPCC, or PCS as further defined below are suitable for use in embankment construction.	No changes suggested
	703.17	Aggregate Materials for 304	696	Furnish aggregate that is CCS, crushed gravel, crushed ACBFS, GS, or OH slags. Ensure that the CCS, crushed gravel, crushed ACBFS, and OH slag meet the gradation requirements.	No changes suggested
Supplemental Specification, April 21, 2006	1015	Compaction Testing of Unbound Materials		Compaction Testing for Soils: Use the direct transmission method according to AASHTO T-310 when testing soils. Use a 12" depth for subgrade and an 8" depth for embankment.	Alternative QA testing should consider dynamic cone penetration to test up to 1 m and light weight deflectometer for rapid testing and the ability. The purpose would be to increase the number of measurements in the field whereby identifying areas that need improvement.
	879	QC/QA for Embankment Construction		Provides means for incentive pay adjustment	No changes suggested
Supplemental Specification, January 19, 2007	842	Correcting Elevation of Concrete Approach Slabs with High Density Polyurethane		Describes application of injected polyurethane for in situ treatment of the approach slab support conditions.	No changes suggested. From a research perspective, study of the backfill conditions at site where this technology is implemented, might provide some insights as to backfill attributes contributing the pavement problems.
Supplemental Specification, January 16, 2009	840	Mechanically Stabilized Earth Wall		Select Granular Backfill Placement: Use SGB material conforming to 703.17 for a height of at least 3' above the bottom of the leveling pad elevation. Place and compact the initial lifts of SGB until it is about 2" above the connection for the bottom layer of soil reinforcement.	Consider use of flooding in the 3ft zone at the back of the wall and also avoid granular material placement within bulking moisture content range.

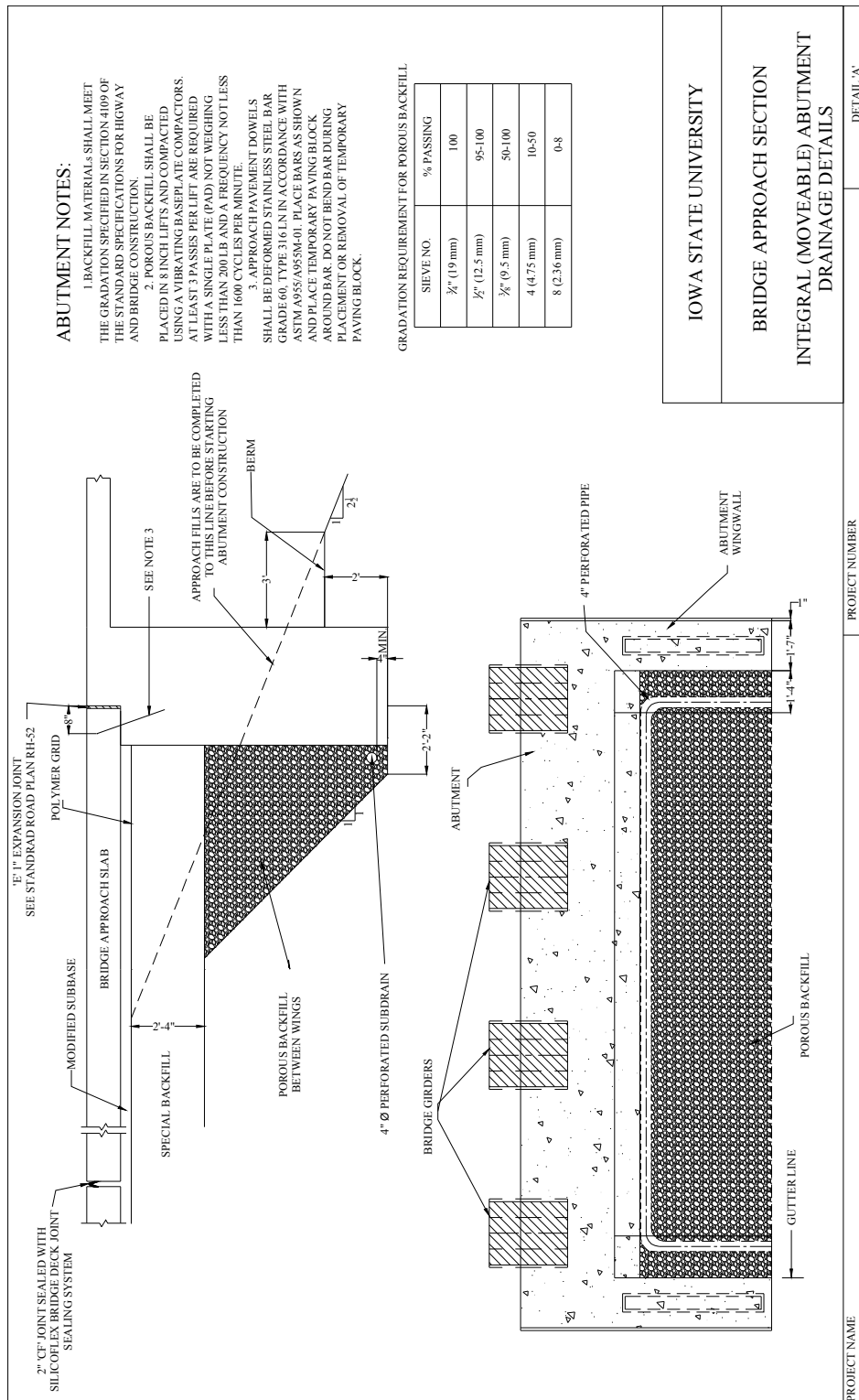


Figure 7.1. Alternative integral bridge approach drainage detail with porous backfill (White et al., 2005)

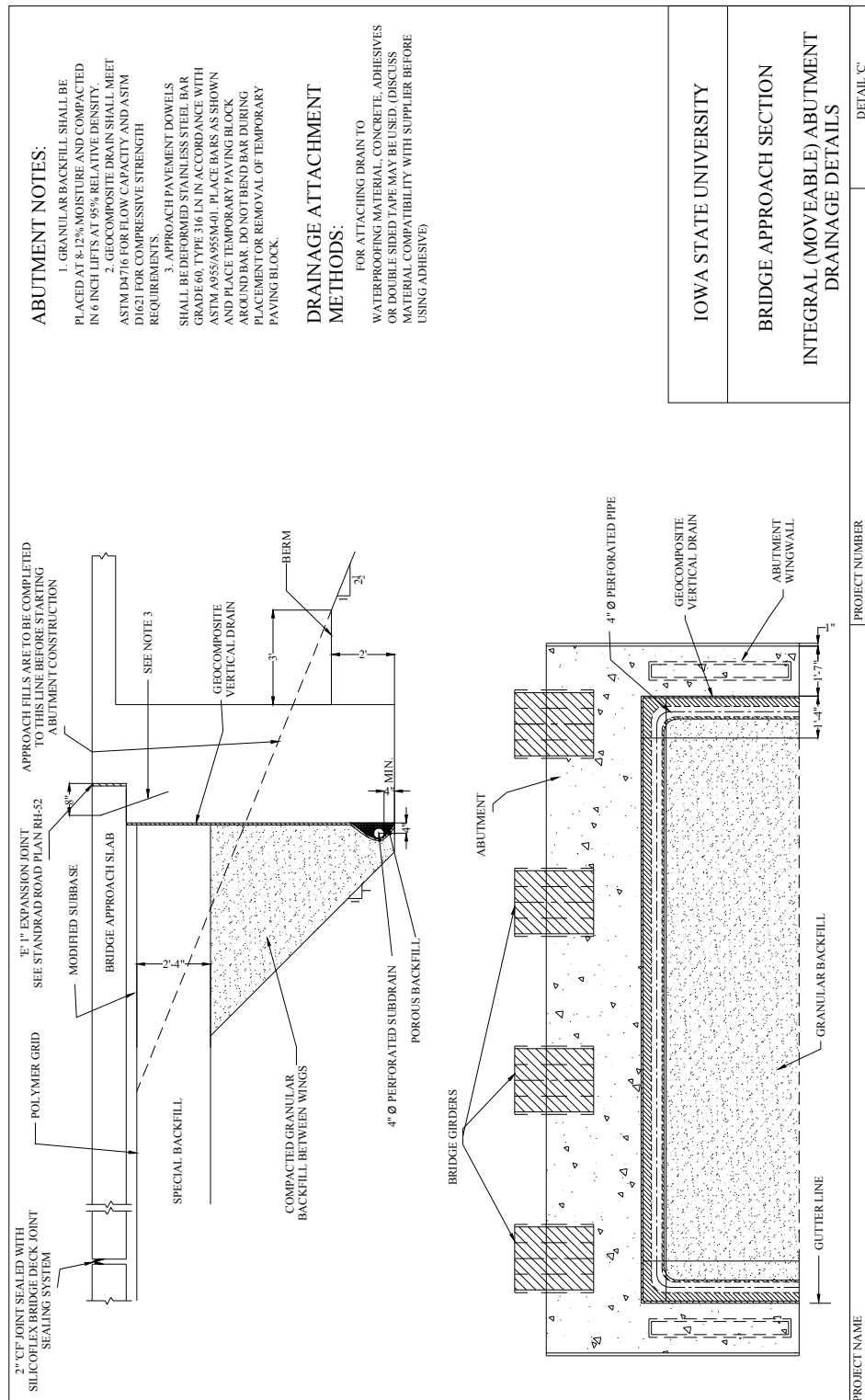


Figure 7.2. Alternative integral bridge approach drainage detail with geocomposite (White et al., 2005)

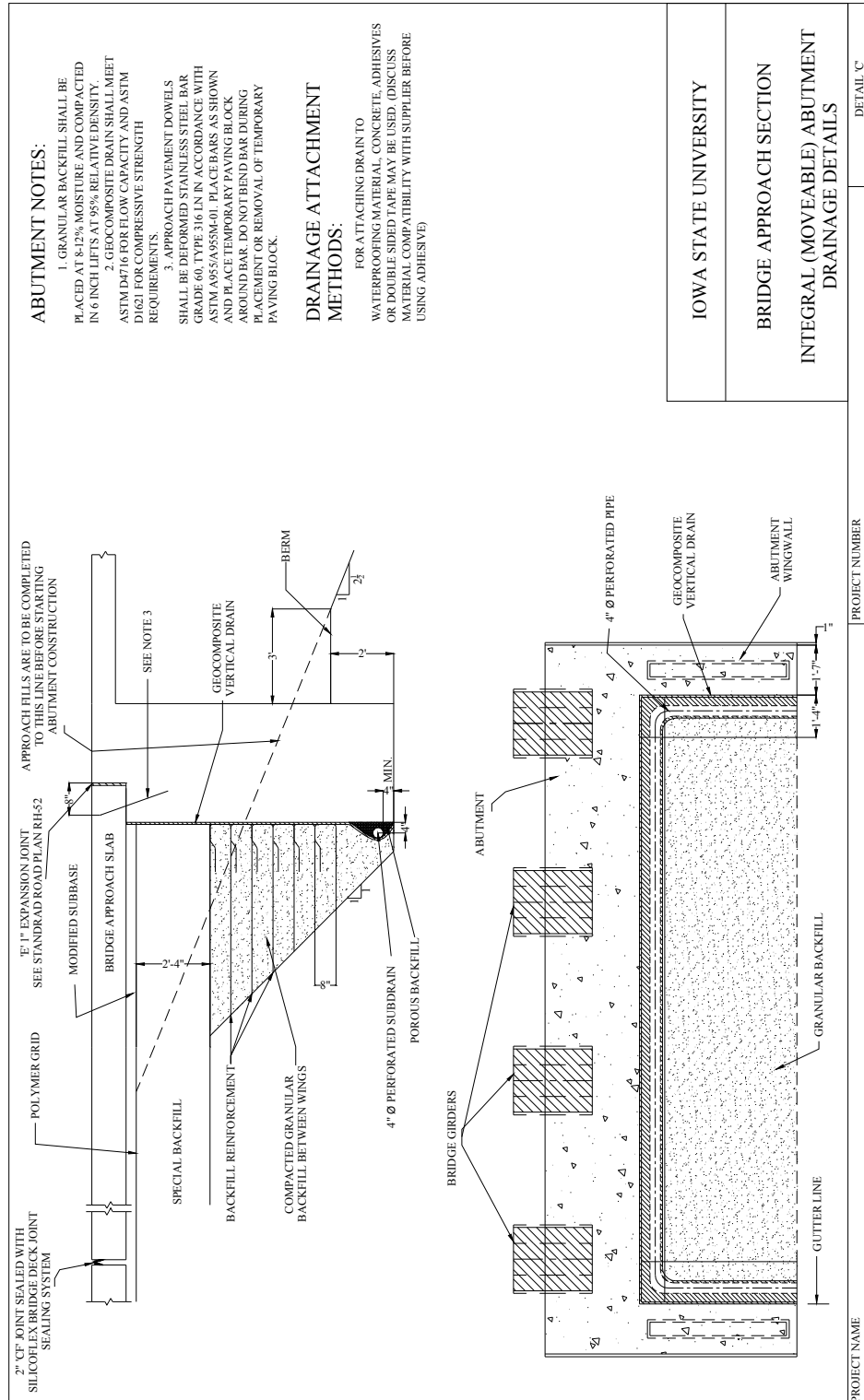


Figure 7.3. Alternative integral bridge approach drainage detail with geotextile reinforcement (White et al., 2005)

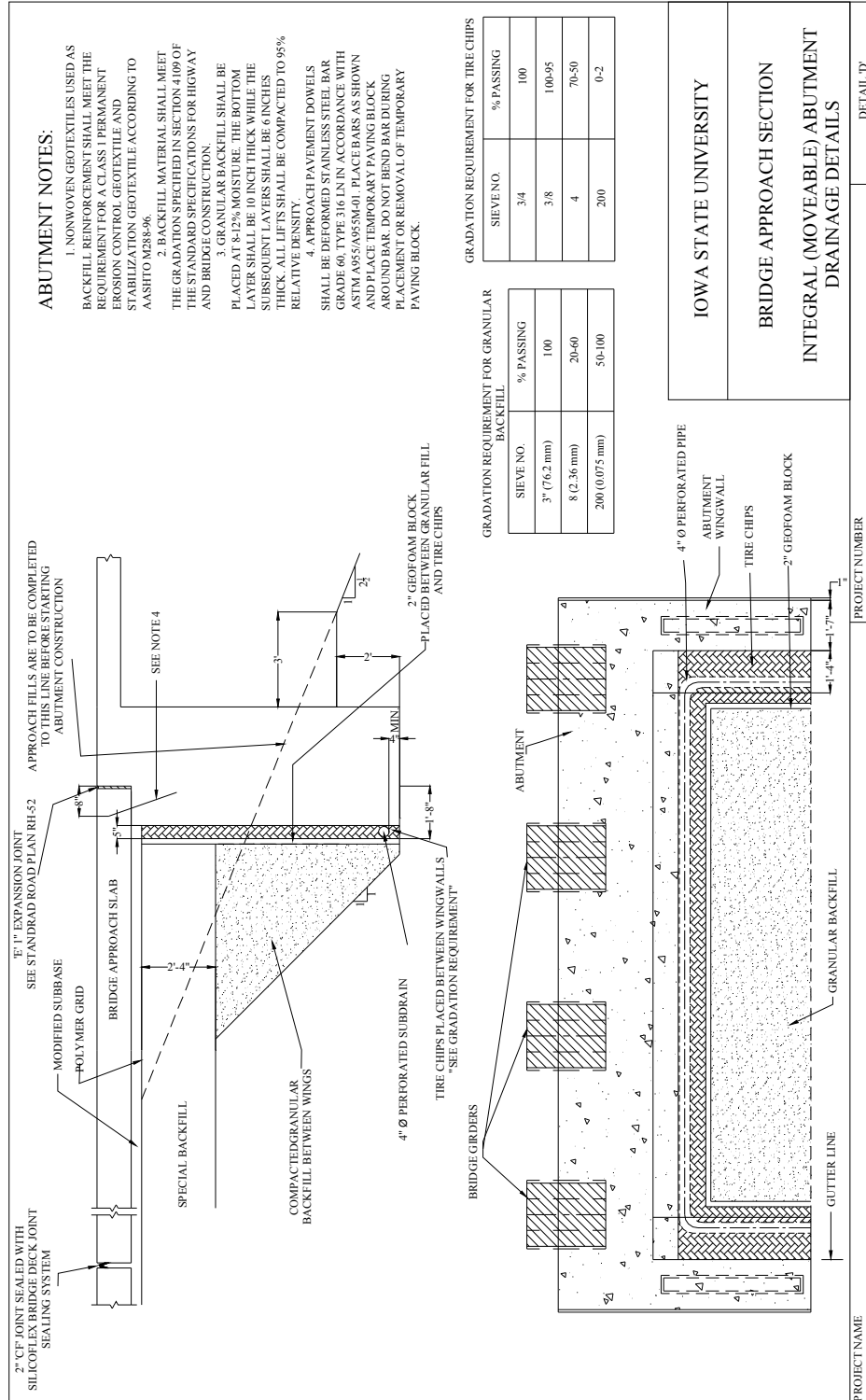


Figure 7.4. Alternative integral bridge approach drainage detail with tire chip backfill (White et al., 2005)

Bump Identification Metrics and Troubleshooting

Many bridge approaches do not provide an ultra-smooth transition on or off the bridge, but they do not exhibit enough rider discomfort to warrant repair or rehabilitation work. The difficulty is determining at what point the discomfort is sufficient enough to examine the bridge more closely and take steps to prevent worsening of the bump problem. To identify if an approach has enough rider discomfort, qualitative and quantitative measures are presented below. Threshold values are given for the qualitative methods provided; however, these are only guidelines and quantitative methods should be used in conjunction with the qualitative methods to get a true idea if the bump problem exists to an extent that warrants further investigation.

Qualitative Technical Methods of Determining Approach Problems:

- If the IRI rating of the approach slab area is greater than 380 in. per mile, the bridge should be visually inspected
- If original and current elevation profiles have been completed on the bridge approach slabs, the Bridge Approach Performance Index can be performed (White et al., 2005). If the index for the approach is greater than 0.016, the bridge should be visually inspected.

Quantitative Methods of Determining Approach Problems:

- Evaluate the ride quality of a 12 or 15 person passenger van driving at a speed of 55 to 60 mph over the bridge approach slabs. The rating is based upon how severe a bump was felt by personnel riding in the back seat.
- If more than two complaints are reported by different users of the bridge about bump problems, the bridge should be visually inspected.

After determining the bridge causes rider discomfort, a visual investigation should be conducted to determine the cause of the problem and how the problem should be remedied. Because each bridge has different character make-ups, based on the embankment fill, structural details, soil conditions, schedule, and economy, it is impractical to suggest that a single problem causing the bump warrants a specific solution. Many of the problems associated with the bump require an interactive multi-solution approach to provide a long-lasting durable solution.

Table 7.2 provides a troubleshooting guide to the general underlying problems causing the bump, visual signs of those problems, possible solutions, and relative parameters of the solutions. Table 7.2 was developed from information and knowledge gained from Tasks 1A, 1B, 1C, 2A, and 2B.

Table 7.2. Summary bump identification metrics and troubleshooting

Possible Cause of Bridge Bump Problem	Indications of Problem Based on Visual Investigation	Possible Solutions ^{ss}							
		Brief Description	Relative Cost*	Relative Durability**	Relative Installation Time/Speed ^	Relative Personnel Expertise ^^	Ease of Installation ^	Relative Effectiveness	Overall Rank
Soil Erosion of Embankment	<ul style="list-style-type: none"> • Loss of spill through slope soil • Ditching of embankment slopes • Virgin soil deposits at toe of slope • Curbs and surface drains plugged with debris or crushed • Elevation of surface drain is higher than pavement • Gap forming between abutment and embankment under bridge • Slope protection under bridge shows signs of more than 1 in. of settlement • Concrete slope protection has large fractures or broken void areas • Poor grass cover and growth 	1. Fill ditches and eroded areas with compacted soil and reestablish seeding	4	1	4	4	3	1	2.8
		2. Place piles of rip-rap rock in locations of erosion and ditches to slow water; fabric can be placed under rock	4	2	4	3	3	1	2.8
		3. Build fabric underlaid rock chutes/channels down embankments and under bridge to control movement of water away from embankment	3	3	4	3	3	2	3.0
		4. Build concrete gutters down embankments and under bridge to control movement of water	2	4	3	3	3	3	3.0
		5. Place curb and gutters along pavement and approach slab to control path of water	2	3	3	3	3	3	2.8
		6. Place surface drains with subsurface piping in pavement shoulder and embankment to drain water on bridge.	1	4	3	3	2	4	2.8
Soil Erosion Under Approach Slab	<ul style="list-style-type: none"> • Void seen under approach slab from shoulder near abutment • Soil deposits at shoulder or on embankment coming from approach slab • Loss or deteriorated expansion joint material at joint • Curbs and surface drains plugged with debris or crushed • Elevation of surface drain is higher than pavement 	1. Place curb and gutter on approach slab to control water movement	3	3	3	3	3	3	3.0
		2. Clean and remove debris from plugged drains	4	2	4	4	4	4	3.6
		3. Place surface drains in pavement with subsurface piping to drain water away from bridge	2	4	1	2	2	4	2.5
		4. Clean joints and expansion joints. Replace compressible joint fill material and strip seals to prevent water from getting below slab	3	3	2	3	4	3	3.0
		5. Remove approach slab; place compacted fill up to grade; dig in drainage tile field under slab and shoulder that is connected to existing subsurface drains; replace approach slab: include shoulder with curb, gutter, and surface drain in approach slab	1	4	1	3	2	4	2.5
		6. Fill erosion void below slab with flowable grout	2	2	3	1	2	2	2
		7. Use geocomposite drainage systems between abutment/backwall and backfill	3	3	2	3	3	3	2.8
Settlement/Compression of Embankment or Abutment	<ul style="list-style-type: none"> • Approach slab relative gradient is greater than 1/200 (0.005)¹ • Settlement cradled is evident in 	1. Place asphalt wedge overlay to bring pavement up to grade	3	2	3	2	2	1	2.2
		2. Grout or liquid polyurethane jacking of the slab	2	2	3	2	2	2	2.2

	<ul style="list-style-type: none"> pavement profile Determine if abutment has settled based on constructed elevations and existing elevations Dip or crown in any 30 ft segment of mainline pavement going away from the approach slab up to 300 ft having a relative gradient larger than 1/200¹ 	and pavement 3. Grind pavement and approach surface to create smooth transition 4. Monitor Settlement; If settlement is complete remove slab and pavement; place compacted fill up to original grade; replace slab and pavement 5. Monitor Settlement; If settlement is not complete remove slab and pavement; Stabilize embankment by use of geotechnical practices such as light weight back fill, rammed aggregate piers, or in situ densification techniques 6. If possible jack or shim abutment to align with pavements	3	2	4	2	2	3	2.7
			1	4	1	1	2	4	2.2
			1	4	1	1	2	4	2.2
			3	3	4	3	4	3	3.3
Differential Vertical Movements	<ul style="list-style-type: none"> Dip or crown greater than 1 in. seen in riding surface relative to curbs or barriers of the approach slab Approach slab relative gradient is greater than 1/200 (0.005)¹ Differential joint movement greater than ½ in. at approach slab to mainline pavement or bridge interface² Broken paving notch seen from the shoulders or suspected due to differential movement Non-uniform vertical gap in bridge parapet at the approach slab 	1. Grind pavement and approach surface to create smooth transition 2. Grout or liquid polyurethane jacking of the slab and pavement 3. Remove and replace approach slab with new cast-in-place or precast approach slab 4. Remove approach slab; add sleeper slab at approach slab to pavement interface; replace slab 5. Remove end of approach slab and cast-in-place a doweled type expansion joint at pavement interface 6. Remove enough approach slab to repair or replace failed paving notch, replace approach slab ensuring adequate bearing, approach slab depth and connection to abutment 7. Resurface mainline pavement creating a smooth transition 8. Remove mainline pavement and poor base and subbase material; replace with good compacted fill material and new pavement	3	2	3	2	2	2	2.3
			2	2	2	2	2	2	2.0
			2	3	2	3	3	3	2.7
			1	4	2	3	3	4	2.8
			2	4	2	3	3	3	2.8
			1	4	1	3	3	2	2.3
			3	1	2	3	3	2	2.3
			1	4	1	3	3	4	2.7
Differential Horizontal Movements	<ul style="list-style-type: none"> Approach slab has moved horizontally away from bridge more than ½ in.² Approach slab has pushed into asphalt mainline pavement causing a vertical bulge of 1 in. or more Approach slab has pulled away from asphalt mainline pavement causing a ½ in. or greater gap at interfaces² Approach slab and concrete mainline pavement have compressed pressure relief joint 	1. Clean debris from joints and refill with compressible joint material or replace strip seal 2. For concrete pavements remove pressure relief joint and cast in doweled type expansion joint 3. For asphalt pavements remove portion of approach slab and pavement; place a sleeper slab; relay asphalt pavement and cast in a doweled type expansion joint 4. Grind any crowns caused by horizontal movement 5. Apply asphalt wedge at any dip locations 6. Remove enough approach slab to repair or replace failed connection to paving notch/bridge abutment	4	3	4	4	4	2	3.5
			3	4	2	3	2	4	3.0
			2	4	2	3	2	4	2.8
			3	1	4	2	3	1	2.3
			3	1	3	2	3	1	2.2
			1	4	2	3	2	2	2.3

	<ul style="list-style-type: none"> causing a vertical bulge of 1 in. or greater Asphalt in pressure relief joint has rutted, or channelized Approach slab and concrete mainline pavement have contracted causing a ½ in. gap at pressure relief joint interfaces² Expansion joint material is present but filled with debris on faces of joint 								
Approach to Mainline Pavement Joint Area Deterioration	<ul style="list-style-type: none"> Transverse cracking on the surface of the approach slab or pavement Spalling of approach slab or pavement near joint Loss of expansion joint material in joint Deteriorated strip seal at the expansion joint Asphalt overlay placed over expansion joint causing cracking or spalling Expansion joint filled with debris and fines Vegetation growing in the expansion joint Strip seal cut short allowing water and debris into joint and under slab 	<ol style="list-style-type: none"> Clean debris from joints and refill with compressible joint material or replace strip seal Remove enough approach slab and pavement to place a sleeper slab; replace pavement; cast in a doweled type expansion joint into approach slab. Saw cut out spalling and cracked approach slab and pavement locations and replace 	4	3	4	4	3	1	3.2
			1	4	2	3	2	4	3
			3	2	3	4	2	2	2.7
Water Improperly Drained	<ul style="list-style-type: none"> Plugged or crushed perforated drainage tiles and outlets Drainage outlets are covered by soil at base of embankment Ponding of water on roadway surface or on or near the bridge embankment Embankment soil erosion as stated previously in table Erosion under approach slab as stated previously in table Approach slab shoulder shows signs of heavy water runoff No surface curbing or surface drains to direct water away from bridge, approach slab, and joints 	<ol style="list-style-type: none"> Unplug or dig out crushed drainage tile and replace with tile that has adequate strength Uncover outlets that have been silted over or covered by embankment material Excavate or fill embankment locations that have ponding water to allow water to drain away from embankment Overlay approach slab and/or pavement with enough transverse crown in road to prevent water from ponding Place curb and gutters to direct water away from approach slab and joints Clean and unplug existing surface drains Install surface drains to prevent erosion of the shoulder or embankment Remove approach slab and pavement and replace with proper subbase and drainage 	1	4	1	3	1	3	2.2
			4	3	4	4	4	4	3.8
			4	3	4	4	4	4	3.8
			3	2	3	3	3	3	2.8
			2	3	3	3	3	4	3.0
			4	3	4	4	4	4	3.8
			2	4	3	3	3	4	3.2
			1	4	2	2	2	3	2.3

	<ul style="list-style-type: none"> • Surface drains blocked by debris 								
Riding-Surface Defects	<ul style="list-style-type: none"> • Large quantities of transverse cracks in approach slab with gaps larger than 0.016in.³ • Pot holing in concrete approach slab, asphalt overlays, mainline pavement, or bridge surface • Rutting, shoving, and channelizing of asphalt pavements • Heavy oil staining, generally dark black and located 10 to 15 ft ahead of bump on the surface 	1. Saw cut out spalling, cracked, or potholed approach slab and pavement locations and replace	3	2	3	3	3	2	2.7
		2. Remove pavement areas with rutting, shoving, and channelized asphalt pavements; correct base and subbase then replace pavement	1	4	3	3	3	3	2.8
		3. Remove approach slab and replace with one that has adequate reinforcing, especially at the end bearing regions to prevent cracking.	1	4	2	3	4	4	3.0

Notes:

* Relative cost scale: 1 = high cost to 4 = low cost

** Relative durability scale: 1= low durability to 4= high durability

^ Relative installation time/speed scale: 1= long time to 4= short time

^^ Relative personnel expertise scale: 1= high expertise to 4= no expertise

‘ Ease of installation scale: 1= hard installation to 4= easy installation

‘‘Relative effectiveness scale: 1= low effectiveness to 4= high effectiveness

^{\$5}To get a true comparison of solution alternatives, a benefit/cost analysis should be completed, based on cost, material availability, and longevity for the area of the bridge

¹Based on work completed by Long et al., 1998 and White et al., 2005

²Based on work completed by Long et al., 1998

³Based on work completed by Oesterle

In general, the possible causes of post-constructed bridge bump problems can be grouped into eight different categories, as presented in Table 7.2. The first cause, soil erosion of embankment, deals with loss of support of the bridge, approach slab, and/or pavement due to the embankment soils being washed away. As soil erosion of the embankment progresses, the embankment soils shift and move to create a stable state. This shifting and movement translates into shifting and movement of the bridge, approach slab, and pavements, causing driving discomfort. To limit this movement, several things can be done, as listed in Table 7.2. The overall rank of the options suggest that building rock or concrete chutes/pathways for the water to drain away from the embankment is the best value; however, the other options can be just as effective, depending on the location, severity, and cause of the erosion.

The second possible bump cause also deals with soil erosion, specifically under the approach slab. Erosion under the approach slab many times is caused by poorly designed, constructed, or maintained joints at the approach slab or by poor shoulder drainage. When water is allowed to get under the approach slab, the soils become wet, changing the material properties. If a “flow” of water is allowed under the slab, soil loss occurs. In both cases, approach slab support is compromised, leading to possible rider discomfort. To solve this problem, simple tasks, such as cleaning plugged surface drains, replacing compressible joint filler, and placing a curb and gutter on the approach, can be very effective at minimizing water infiltration below the slab. If extensive erosion and loss of support has taken place, more drastic measures, such as removing the approach slab, installing a drainage system, and replacing soil and the approach slab, may be required.

The settlement or compression of the embankment or bridge abutment is the third listed bump cause in Table 7.2. The most prominent causes of settlement are due to improper soils used for embankments, improper compaction of embankment soils, and inadequate foundation soils. One of the most effective methods of correcting settlement issues is to jack or shim the abutment to align with the pavements. This solution, however, requires the abutment to be designed as a moveable abutment. Most abutments are not designed to be moved in a vertical manner. Grinding the riding surface or placing wedges are cost-effective ways of providing a smooth riding surface; however, in many cases they are not long-term fixes. To provide a permanent fix, the embankment needs to be monitored for further settlement and addressed according to whether settlement is still occurring or if settlement has ceased. The permanent fix has a low rank in Table 7.2 due to the time and cost it takes to fix the problem. The effectiveness and durability of the permanent fix, however, may outweigh the increased cost and time.

The fourth item pertains to the differential vertical movements that occur between the bridge and the approach slab and the approach slab and the pavement. Many variables, all discussed within this report, can be attributed to the cause of differential vertical movements. The highest-ranked solution for fixing differential vertical movements is to remove the approach slab, fix the underlying problems, and replace the approach slab. Many times the differential movement is not the problem, but a symptom of the problem. Items discussed in bump causes one through three can all cause differential movement. To obtain a permanent fix of the differential movement, the source problem also needs to be determined and corrected.

The fifth bump cause in Table 7.2, differential horizontal movements, pertains to the longitudinal expansion and contraction of the bridge, approach slab, and pavement. Generally, differential horizontal movement at the bridge to approach interfaces results from connection failure of the approach slab to the abutment. The highest-ranked solution for this problem would be to remove a portion or the entire approach slab and replace the failed connection. Differential movement at the approach to pavement interface generally indicates problems when a doweled expansion joint is not present, the expansion joint is not maintained, or a pressure relief joint is used. Maintaining the expansion joints, if present, is the easiest way to provide ride comfort. If an expansion joint is not present or a pressure relief joint is used, a doweled-type expansion joint should be installed to relieve stresses in the approach slab and pavement.

Deterioration of the pavement and approach slab, the sixth bump cause in Table 7.2, results from poor joint maintenance, poor design practices, and deteriorated overlay repairs. The highest-ranked solution consists of maintaining joints; however, if no joint exists, an expansion joint should be placed at the interface. If deteriorated overlays exist, they should be removed. The overlay can be replaced, but fixing the underlying reason for the corrective overlay may provide a longer-lasting and more durable solution.

The seventh bump-causing problem is improperly drained water. This problem relates very closely to the erosion and settlement issues presented earlier and reaffirms that the bump problem is not a one solution problem, but requires a multi-solution. One of the easiest ways to ensure proper water drainage is to maintain drainage outlets and inlets. This requires unplugging and uncovering drainage ways, so water flow is not blocked. Excavating or filling embankment, shoulder, and ditch locations where water ponds can also be an easy and effective way to control water movement. If the bridge location was not previously designed for water drainage, curb, gutters, and drains should be placed on the approach slab, shoulder, and pavement to direct water away from the bridge without causing damage, as stated within this report.

Ride surface defects is the eighth major cause of bump problems at a bridge. Surface defects are generally caused by poorly-designed or -placed concrete or asphalt or inadequate reinforcing in the approach slab. The most effective way to repair bad or rough pavement locations are to saw cut out the locations and replace them with good sound pavement. If the locations are in the approach slab, it may be more cost-effective to replace the entire slab, rather than attempt to patch it. Locations of bad concrete on the bridge would require resurfacing or patching, depending on the extent of the deteriorated surface.

Corrective Action Evaluation

Although a limited number of sites were available to help the research team evaluate the corrective strategies listed above, the rideability of several bridges before and (sometimes) after corrective actions were evaluated. The results of that evaluation were used to aid the research team in making some of the assessments in Table 7.2. A summary of the corrective evaluation results are shown in Table 7.3.

Table 7.3. Evaluation of bridge rideability and corrective strategies

Bridge	Status	Lane	Right Wheel Line						Left Wheel Line					
			Average	Reduction	Peak Entrance	Reduction	Peak Exit	Reduction	Average	Reduction	Peak Entrance	Reduction	Peak Exit	Reduction
SR161 over Beech Road	Before Correction	East Lane 1	151		302		311		151		258		329	
		East Lane 2	156		276		336		161		259		328	
		West Lane 1	197		312		315		225		362		428	
		West Lane 2	225		384		477		223		436		281	
	After Correction	East Lane 1	86	43%	133	56%	124	60%	86	43%	136	47%	180	45%
		East Lane 2	84	46%	173	37%	88	74%	95	41%	190	27%	132	60%
		West Lane 1	75	62%	84	73%	86	73%	82	64%	86	76%	227	47%
		West Lane 2	84	63%	109	72%	121	75%	89	60%	121	72%	217	23%
SR161 over Mink Road	Before Correction	East Lane 1	164		333		272		183		267		247	
		East Lane 2	171		247		253		181		350		230	
		West Lane 1	166		240		324		185		208		332	
		West Lane 2	199		319		368		198		328		276	
	After Correction	East Lane 1	67	59%	119	64%	85	69%	74	59%	137	49%	83	66%
		East Lane 2	81	53%	120	51%	163	36%	85	53%	175	50%	101	56%
		West Lane 1	82	51%	113	53%	104	68%	122	34%	111	47%	220	34%
		West Lane 2	101	49%	142	56%	188	49%	92	53%	187	43%	131	52%
Lic 310 over SR 161	Before Correction	South Lane	120		222		254		119		282		237	
		Center Lane	143		336		182		131		264		196	
		North Lane	129		303		239		141		316		204	
	After Correction	South Lane	52	57%	101	55%	91	64%	51	57%	105	63%	71	70%
		Center Lane	55	62%	110	67%	66	64%	56	57%	107	60%	74	62%
		North Lane	53	59%	91	70%	78	67%	53	62%	89	72%	83	59%
CR 539 Service Road Bridge	Before Correction	East Lane	203		446		361		196		429		312	
		West Lane	187		335		343		192		256		431	
	After Correction	East Lane	97	52%	209	53%	126	65%	109	44%	246	43%	205	34%
		West Lane	93	50%	157	53%	220	36%	104	46%	124	52%	345	20%
Bridge Leading to Golf Course (OM)	Before Correction	East Lane	228		364		337		206		428		246	
		West Lane	190		343		286		229		424		389	
	After Correction	East Lane	128	44%	194	47%	211	37%	85	59%	140	67%	110	55%
		West Lane	92	51%	150	56%	130	55%	88	62%	151	64%	150	61%
161 Chimney Rd Bridge	Before Correction	West Driving Lane	226		375		346		228		457		324	
		West Passing Lane	197		387		335		175		274		271	
		East Driving Lane	156		297		252		165		272		283	
		East Passing Lane	163		258		243		174		292		286	
	After Correction	West Driving Lane	90	60%	147	61%	108	69%	112	51%	232	49%	173	47%
		West Passing Lane	90	54%	171	56%	148	56%	74	58%	106	61%	118	56%
		East Driving Lane	62	60%	126	58%	113	55%	71	57%	124	55%	117	58%
		East Passing Lane	74	55%	119	54%	122	50%	74	57%	145	50%	84	71%
161 Moot Rd Bridge	Before Correction	West Driving Lane	121		233		320		126		308		339	
		West Passing Lane	150		361		330		140		325		307	
		East Driving Lane	140		290		307		143		246		392	
		East Passing Lane	132		336		304		156		390		335	
	After Correction	West Driving Lane	63	48%	140	40%	128	60%	60	52%	123	60%	142	58%
		West Passing Lane	65	57%	125	65%	118	64%	66	53%	139	57%	94	70%
		East Driving Lane	72	48%	167	42%	163	47%	71	51%	122	51%	235	40%
		East Passing Lane	64	52%	172	49%	161	47%	74	53%	205	47%	175	48%
York Road over SR161	Before Correction	South Lane	154		464		226		137		363		237	
		North Lane	128		285		305		151		402		389	
		Center Lane	165		360		486		161		372		447	
	After Correction	South Lane	56	64%	121	74%	95	58%	58	58%	139	62%	74	69%
		North Lane	56	56%	102	64%	140	54%	59	61%	103	74%	152	61%
		Center Lane	73	56%	118	67%	94	81%	69	57%	135	64%	88	80%
Outville Road over SR161	Before Correction	South Lane	172		490		256		159		404		317	
		North Lane	135		278		249		147		236		380	
	After Correction	South Lane	76	56%	246	50%	88	66%	63	60%	195	52%	107	66%
		North Lane	56	58%	128	54%	94	62%	52	64%	128	46%	96	75%

Bridge	Status	Lane	Right Wheel Line						Left Wheel Line					
			Average	Reduction	Peak Entrance	Reduction	Peak Exit	Reduction	Average	Reduction	Peak Entrance	Reduction	Peak Exit	Reduction
LAW 7 Bridge 6.90	Before	Northbound	281		676		609		295		748		415	
	Correction	Southbound	291		742		701		302		597		719	
LAW 7 Bridge 8.34	Before	Eastbound	170		332		323		161		284		276	
	Correction	Westbound	204		335		440		187		361		325	
LAW 7 Bridge 8.83	Before	Eastbound	143		426		627		150		475		889	
	Correction	Westbound	168		415		628		149		452		606	
Marion SR 47	Before	Down	274		734		907		248		725		828	
	Correction	Up	334		759		961		307		826		998	
Cuy 1480 Over Libby Road	Before	Eastbound Left Lane	149		299		409		159		447		377	
		Eastbound Right Lane	184		455		866		180		464		601	
		Westbound Left Lane	197		501		467		211		450		639	
		Westbound Right Lane	169		464		565		205		648		709	
	After	Eastbound Left Lane	119	21%	302	-1%	187	54%	141	12%	509	-14%	307	19%
		Eastbound Right Lane	147	20%	346	24%	575	34%	150	17%	385	17%	284	53%
		Westbound Left Lane	166	16%	391	22%	433	7%	172	18%	303	33%	408	36%
		Westbound Right Lane	141	16%	323	30%	486	14%	155	24%	444	31%	516	27%
XR795 Over I-280	Before	East	59		288		221		97		300		190	
	Correction	West	113		282		359		118		251		384	
FAI 033 14 17	Before	RD Lane 1	157		478		440		177		524		534	
		RD Lane 2	163		468		537		160		571		315	
		RI Lane 1	171		465		513		173		479		577	
		RI Lane 2	155		577		408		174		604		413	
FRA 270 32 36	Before	RD Lane 1	162		281		440		151		305		384	
		RD Lane 2	136		278		235		137		329		228	
		RD Lane 3	180		394		260		169		362		249	
		RI Lane 1	159		346		344		172		335		402	
		RI Lane 2	172		342		445		176		420		346	
		RI Lane 3	168		494		328		173		506		280	
FRA 317 8 09	Before	RD Lane 1	162		220		380		253		360		536	
		RD Lane 2	203		287		484		245		456		722	
		RI Lane 1	215		384		407		263		353		502	
		RI Lane 2	339		596		750		283		454		733	
MUS 016 7 69	Before	RD Lane 1	282		653		756		276		677		694	
		RD Lane 2	226		583		835		248		729		737	
		RI Lane 1	218		845		603		206		624		628	
		RI Lane 2	171		485		573		167		418		633	
PRE 070 12 49	Before	RD Lane 1	154		684		608		159		461		836	
		RD Lane 2	141		429		631		199		371		794	
		RI Lane 1	161		503		781		189		530		887	
		RI Lane 2	196		541		627		165		780		663	
RIC 403 9 98	Before	RD Lane 1	183		488		459		195		725		447	
	Correction	RI Lane 1	218		635		1178		187		433		753	
Ross 207 over Scioto River	Before	West	155		612		383		156		492		434	
	Correction	East	157		500		509		158		392		467	
	After	West	132	15%	555	9%	359	6%	127	18%	474	4%	377	13%
	Correction	East	132	16%	456	9%	431	15%	132	16%	394	0%	457	2%
	After	West	138	11%	749	-22%	350	9%	135	13%	733	-49%	389	10%
	Correction	East	139	11%	485	0%	640	-26%	129	18%	443	-18%	568	-23%
Scioto US23	Before	North Left Lane	115		318		321		124		403		392	
		North Right Lane	135		531		471		222		517		536	
		South Left Lane	110		447		328		103		450		307	
		South Right Lane	137		379		382		223		583		610	
	After	North Right Lane	119	12%	265	50%	368	22%	117	47%	239	54%	257	52%
		South Right Lane	131	4%	237	37%	375	2%	156	30%	692	-19%	656	-8%

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