1. Report No. FHWA/TX-05/9-8132-3	2. Government Accession No.	3. Recipient's Catalog No.
4. Title and Subtitle TESTING AND EVALUATION OF THE FLORIDA F SHAPE BRIDGE RAILWITH REDUCED DECK THICKNESS		5. Report Date March 2005
		6. Performing Organization Code
7. Author(s) Dean C. Alberson, William F. Williams, and Wanda L. Menges		8. Performing Organization Report No. Report 9-8132-3
9. Performing Organization Name and Address Texas Transportation Institute The Texas A&M University System College Station, Texas 77843-3135		10. Work Unit No. (TRAIS)
		11. Contract or Grant No. Project 9-8132
12. Sponsoring Agency Name and Address Texas Department of Transportation Research and Technology Implementation Office P.O. Box 5080 Austin, Texas 78763-5080		13. Type of Report and Period Covered Technical Report: September 2002 – January 2005
		14. Sponsoring Agency Code

15. Supplementary Notes
Project performed in cooperation with the Texas Department of Transportation, Florida Department of Transportation and the Federal Highway Administration.
Project Title: FDOT Bridge Rails

16. Abstract

Ultimately, a bridge rail should contain and redirect errant vehicles with minimal damage to the bridge structure. A number of different types of concrete safety-shaped bridge rails are used by most states. Over the years, a number of different reinforcement schemes have been used and most have withstood the rigors of the highway environment. Obviously, reinforcement schemes may vary significantly and still achieve the objective to contain and redirect errant design vehicles.

As experience is gained with bridge rails, designs change. The geometry, such as height, shape, and openness, may change due to vehicle mix, vehicle design changes, or public opinion. However, a move to a new design does not necessarily negate the usefulness of older systems. Or, an upgrade in performance requirements does not automatically indicate the older system will not perform acceptably when impacted under new performance requirement conditions. The safety performance of bridge rails is ultimately evaluated by a performance-based test, i.e. a full-scale crash test.

Safety-shapes have been shown to perform acceptably in field applications. Many different types, shapes, and differently reinforced bridge parapets are used throughout the United States. Typical failure patterns of the safety-shaped bridge parapets follow the multiple hinge/pattern shown in the *AASHTO LRFD Specification* Section 13 Figure CA 13.3.1-1, however, the yield lines are usually confined to the upper portion of the safety-shape. This report investigates the required deck thicknesses associated with safety-shapes and more specifically the F-shape as adopted by Florida Department of Transportation.

17. Key Words Bridge Rail, F Shape, LRFD Bridge Deck Requirement, Roadside Safety	18. Distribution Statement No restrictions. This document is available to the public through NTIS: National Technical Information Service Springfield, Virginia 22161 http://www.ntis.gov	
19. Security Classif.(of this report) Unclassified	20. Security Classif.(of this page) Unclassified	21. No. of Pages	22. Price

Form DOT F 1700.7 (8-72)   Reproduction of completed page authorized

TESTING AND EVALUATION OF THE FLORIDA F SHAPE BRIDGE RAIL WITH REDUCE DECK THICKNESS

by

Dean C. Alberson, P.E.
Associate Research Engineer
Texas Transportation Institute

William F. Williams, P.E.
Assistant Research Engineer
Texas Transportation Institute

and

Wanda L. Menges
Associate Research Specialist
Texas Transportation Institute

March 2005

TEXAS TRANSPORTATION INSTITUTE
The Texas A&M University System
College Station, Texas 77843-3135

DISCLAIMER

The contents of this report reflect the views of the authors, who are responsible for the facts and the accuracy of the data, and the opinions, findings, and conclusions presented herein. The contents do not necessarily reflect the official view or policies of the Texas Department of Transportation (TxDOT), Federal Highway Administration (FHWA), Florida Department of Transportation, the Texas A&M University System, or the Texas Transportation Institute. This report does not constitute a standard, specification, or regulation, and its contents are not intended for construction, bidding, or permit purposes. In addition, the above listed agencies assume no liability for its contents or use thereof. The names of specific products or manufacturers listed herein do not imply endorsement of those products or manufacturers. The engineer in charge was Dean C. Alberson, P.E., (State of Texas, #74891).

ACKNOWLEDGMENTS

TABLE OF CONTENTS

LIST OF FIGURES

CHAPTER 1. INTRODUCTION

PROBLEM

The provisions of Section 13 of the American Association of State Highway and Transportation Official's (AASHTO) *Load Resistance Factor Design Bridge Specifications* are based on the assumption that the yield line failure pattern is confined to the concrete parapet and does not extend into the bridge deck *(1)*. In the event that the yield line pattern extends into the deck, the equations for strength of the system will not be correct.

There is argument for the philosophy that structural failure, in the event that such occurs from excessive load caused by an excessively severe collision, should be restricted to the parapet and not be allowed to extend into the bridge deck. Presumably, a structural failure that extends into the deck would be much more catastrophic and costly to repair than one that would be confined to the parapet only. On the other hand, one could argue that in an extremely severe collision, structural failure of the deck could be acceptable.

A further argument can be made based on the philosophy that the parapet and the deck should each be designed to carry the design load without undue conservatism, and without regard to the manner in which the structure would fail, in the event that an excessive overload would occur.

The question to be addressed in this task is the magnitude of bending moment for which the bridge deck should be designed. Should the deck be designed for the moment capacity of the barrier at its base? Or, should the deck be designed for the average bending moment over the height of the barrier (The bending moment value used to calculate the strength of the barrier)? Or, should the deck be designed to resist some other value of bending moment?

BACKGROUND

The *AASHTO LRFD Specification* Section 13 sets forth test levels and the required test conditions for demonstrating a bridge rail meets a certain test level. The Appendix to Section 13 gives engineering guidelines for designing bridge rails that will perform satisfactorily in full-scale crash tests. The Appendix to Section 13 was originally written as an aid to designers and not intended to be a mandatory design requirement. It is now interpreted by some individuals as a mandatory design requirement. Bridge rails may be designed by other methods and would be considered acceptable if the rail performed acceptably in full-scale crash tests.

Ultimately, a bridge rail should contain and redirect errant vehicles with minimal damage to the bridge structure. A number of different types of concrete safety-shaped bridge rails are used by most states. Over the years, a number of different reinforcement schemes have been used and most have withstood the rigors of the highway environment. One end of the spectrum for steel reinforcement in concrete barriers is the Ontario "Tall Wall" *(2)*. The Ontario "Tall Wall" is a safety shaped median barrier that was successfully crash tested with an 80,000-lb

(36 000 kg) tractor/trailer, and no steel reinforcement was used in the system. A common safety-shaped bridge rail used extensively in Texas is the T501. The T501 uses a moderate amount of steel reinforcement. Other barriers use extensive reinforcement. Obviously, reinforcement schemes may vary significantly and still achieve the objective to contain and redirect errant design vehicles.

As experience is gained with bridge rails, designs change. The geometry, such as height, shape, and openness, may change due to vehicle mix, vehicle design changes, or public opinion. However, a move to a new design does not necessarily negate the usefulness of older systems. Or, an upgrade in performance requirements does not automatically indicate the older system will not perform acceptably when impacted under new design requirement conditions. The safety performance of bridge rails is ultimately evaluated by a performance-based test, i.e. a full-scale crash test.

Safety-shapes have been shown to perform acceptably in field applications. Many different types, shapes, and differently reinforced bridge parapets are used throughout the United States. For non-uniform thickness bridge parapets (such as safety shape) yieldline failure patterns, as shown in the *AASHTO LRFD Specification* Section 13, Figure CA 13.3.1-1, may not extend to the deck. Failure patterns sometimes tend toward punching type failures.

This report investigates the required deck thicknesses (bending moment capacity) associated with safety-shapes and more specifically the F-shape as adopted by Florida Department of Transportation (FDOT).

CHAPTER 2. STUDY APPROACH

TEST FACILITY

The test facilities at the Texas Transportation Institute's Proving Ground consist of a 2000-acre (809 hectare) complex of research and training facilities situated 10 mi (16 km) northwest of the main campus of Texas A&M University. The site, formerly an Air Force Base, has large expanses of concrete runways and parking aprons well suited for experimental research and testing in the areas of vehicle performance and handling, vehicle-roadway interaction, durability and efficacy of highway pavements, and safety evaluation of roadside safety hardware. The site selected for placing of the Florida F-shape bridge rail is along a wide concrete apron/runway. The apron/runway consists of an unreinforced jointed concrete pavement in 12.5 ft by 15 ft (3.8 m by 4.6 m) blocks nominally 8-12 in (203-305 mm) thick. The aprons and runways are about 50 years old and the joints have some displacement, but are otherwise flat and level.

TEST ARTICLE

Design and Construction

Texas Transportation Institute (TTI) received drawings from Florida Department of Transportation entitled "Traffic Railing Barrier – (32-inch [813 mm] F-Shape) Index No. 700 (Drawing 2 of 2), and dated June 30, 2000. These drawings provided details for the concrete F-shape barrier constructed for his project.

For this project, TTI constructed three different test installations using the Florida 32-inch (813 mm) F-shape barrier. Each test installation incorporated a different cantilever deck design supporting the concrete F-shape barrier. The concrete F-shape barrier was 32 inches (813 mm) in height and 10-3/4 inches (273 mm)wide at the top and 1 ft 6 inches (457 mm) wide at the base. Vertical reinforcement in the barrier consisted of #5 (#16) bars designated 5V and 5P spaced 8 inches (203 mm) on centers. The 5V bars extended 6 inches (152 mm) into the 8 inch (203 mm) thick cantilever decks supporting the barrier. Longitudinal reinforcement in the barrier consisted of eight #5 (#16) bars designated 5S with four bars moderately spaced on each side of the barrier cross-section. Two of the 5S bars were located outside of the vertical reinforcement with the remaining bars located within the vertical reinforcement. All exposed corners on the barriers and decks received 3/4-inch (19 mm) chamfer.

Three different concrete cantilever deck designs were constructed and used to support the F-shape barrier. All three designs were constructed identically, with the exception of the spacing of the transverse reinforcement in the top and bottom layers. The three decks were 8 inches (203 mm) thick and were cantilevered 3 ft 6 inches (1.1 m) from the edge of an existing rigid concrete structure at the testing facility. This cantilever distance was representative of typical concrete deck cantilevers used on typical bridge structures in Florida. Longitudinal reinforcement in the deck consisted of two layers of #4 (#13) bars spaced 12 inches (305 mm) on

centers with two additional #4 (#13) bars located within the 5V bars in the bottom layer of longitudinal reinforcement. Transverse reinforcement in the top and bottom layers in the three deck designs consisted of #5 (#16) bars spaced at 4.5 inches, 6.0 inches and 8.0 inches (114 mm, 152 mm, and 203 mm) on centers. The lengths of the of the three test installations were 30 ft, 36 ft, and 44 ft (9.1 m, 11.0 m, and 13.4 m), respectively.

Strain gages were placed on six transverse #5 (#16) bars in the top layer of reinforcement in the deck and six 5V bars at three different locations to measure the strain during applied static loading on the barrier. The location of the strain gages are shown on Figure 1. Static loading was applied to the barrier section using a hydraulic ram with a pressure transducer as shown in Figure 5. The load was uniformly distributed over a length of 3.5 ft (1.1 m) using a segment of W24x84 (W610x125) structural steel shape. Loading readings were measured and recorded from the pressure transducer until failure occurred in the concrete barrier. Compressive strength tests performed on representative samples of concrete taken from concrete used in the bridge railings and decks revealed an average compressive strength of approximately 4500 psi (31 MPa). Additional details are provided on Figures 1 through 5.

Analysis of Bridge Railing

The *AASHTO LRFD Specification* Section 13 sets forth test levels and the required test conditions for demonstrating a bridge rail meets a certain test level. The Appendix to Section 13 gives guidelines for designing bridge rails that will perform satisfactorily in full-scale crash tests. The yield line procedure in Section A13.3.1 was used to evaluate the Florida F-shape bridge rail. Both the mid-span and end span conditions were evaluated. Results of the yield line analysis for both conditions are included with this report in Appendix A.
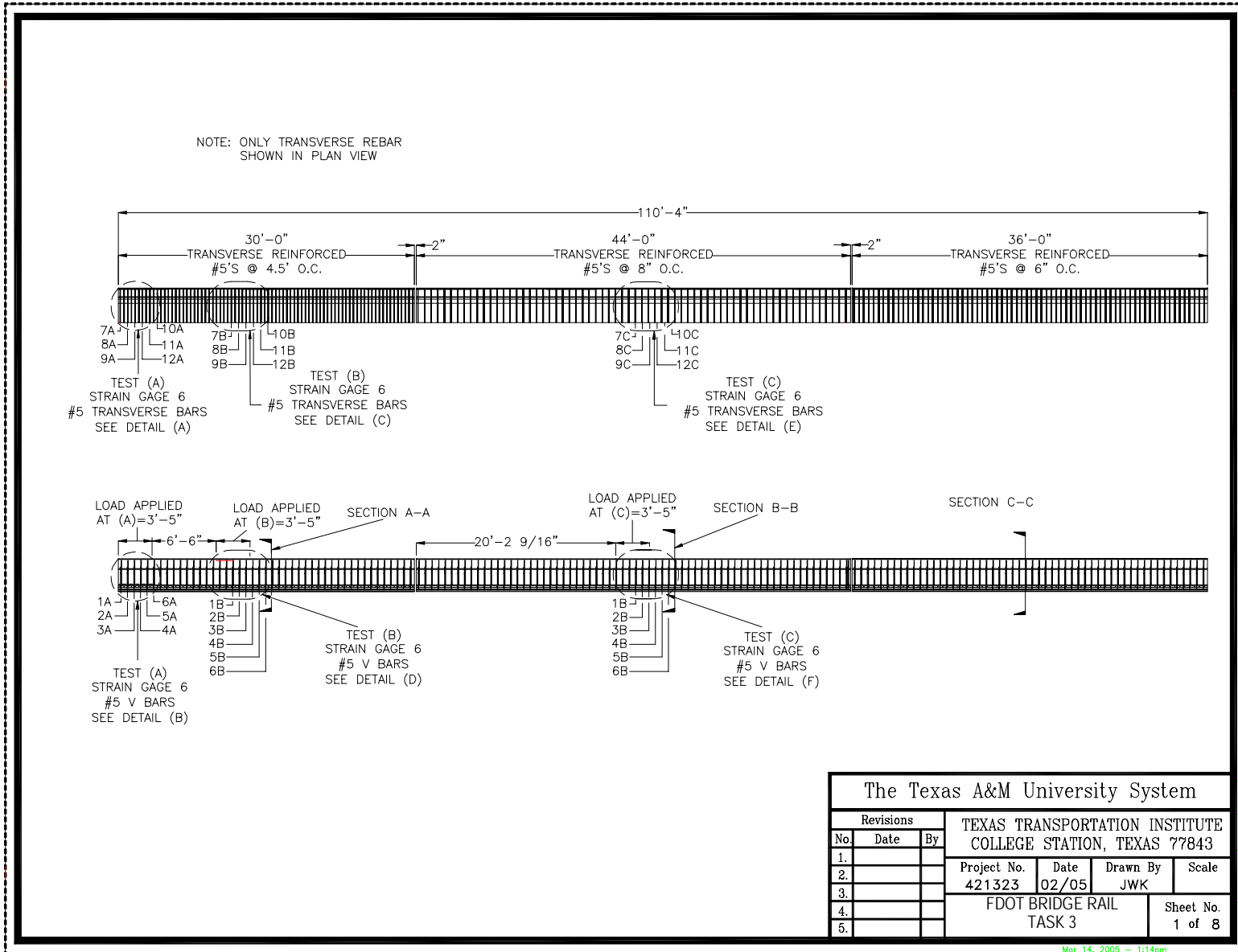
NOTE: ONLY TRANSVERSE REBAR
SHOWN IN PLAN VIEW

|←————————————————————————— 110'-4" ——————————————————————————→|

30'-0"	2"	44'-0"	2"	36'-0"
TRANSVERSE REINFORCED		TRANSVERSE REINFORCED		TRANSVERSE REINFORCED
#5'S @ 4.5' O.C.		#5'S @ 8" O.C.		#5'S @ 6" O.C.

7A┐ ┌10A   7B┐ ┌10B   7C┐ ┌10C
8A┤ ├11A   8B┤ ├11B   8C┤ ├11C
9A┘ └12A   9B┘ └12B   9C┘ └12C

TEST (A)   TEST (B)   TEST (C)
STRAIN GAGE 6   STRAIN GAGE 6   STRAIN GAGE 6
#5 TRANSVERSE BARS   #5 TRANSVERSE BARS   #5 TRANSVERSE BARS
SEE DETAIL (A)   SEE DETAIL (C)   SEE DETAIL (E)

LOAD APPLIED   LOAD APPLIED   LOAD APPLIED
AT (A)=3'-5"   AT (B)=3'-5"   SECTION A-A   AT (C)=3'-5"   SECTION B-B   SECTION C-C

|← 6'-6" →| |←——— 20'-2 9/16" ———→|

1A┐ ┌6A   1B┐ ┌   1B┐ ┌
2A┤ ├5A   2B┤ ├   2B┤ ├
3A┘ └4A   3B┤ ├   3B┤ ├
               4B┤ ├   4B┤ ├
               5B┤ ├   5B┤ ├
               6B┘ └   6B┘ └

TEST (A)   TEST (B)   TEST (C)
STRAIN GAGE 6   STRAIN GAGE 6   STRAIN GAGE 6
#5 V BARS   #5 V BARS   #5 V BARS
SEE DETAIL (B)   SEE DETAIL (D)   SEE DETAIL (F)

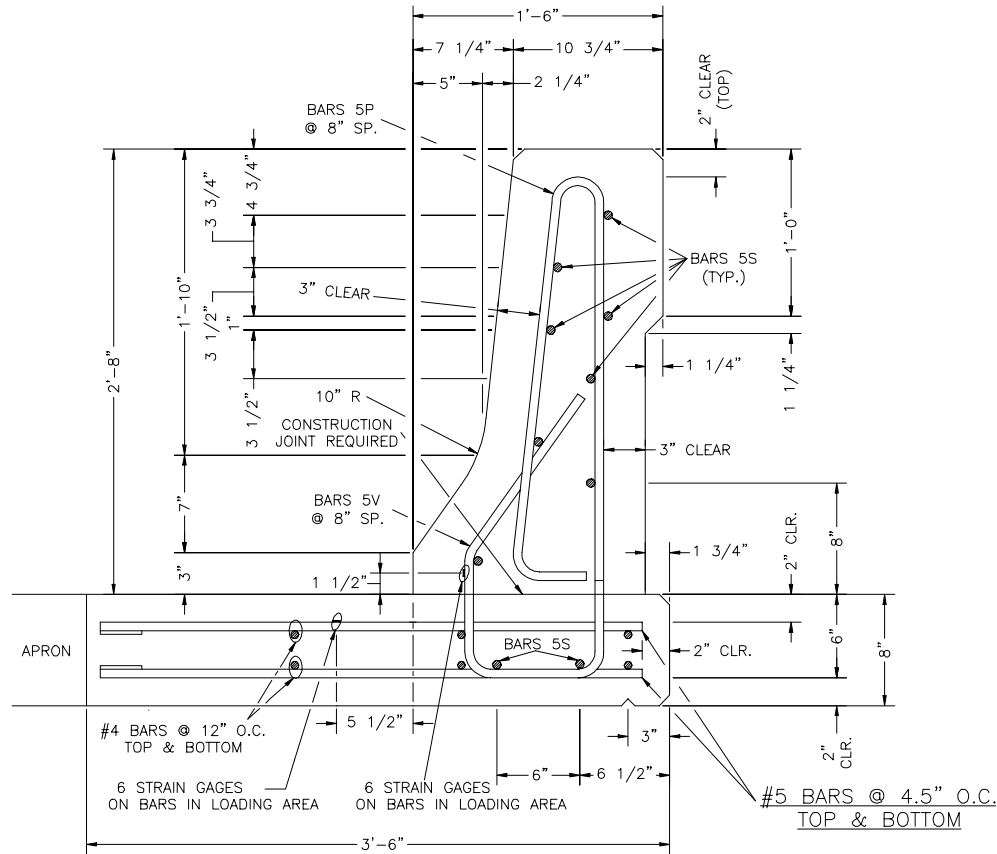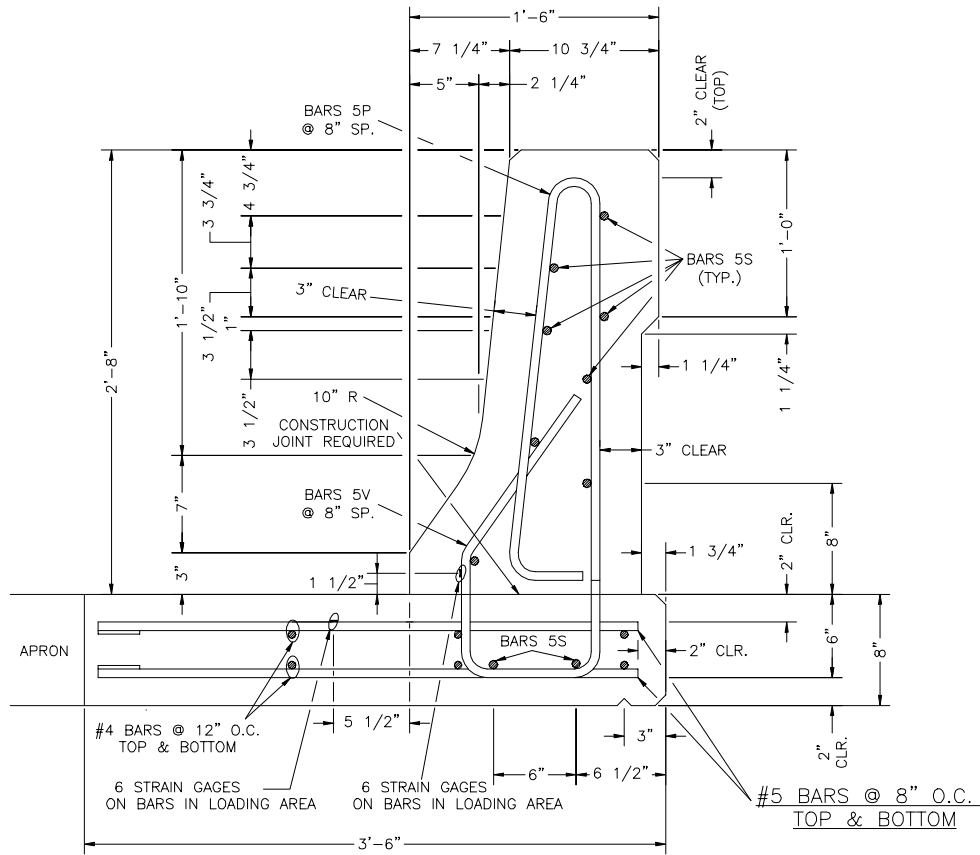| The Texas A&M University System		
Revisions	TEXAS TRANSPORTATION INSTITUTE	
No. Date By	COLLEGE STATION, TEXAS 77843	
1.	Project No.	Date	Drawn By	Scale
2.	421323	02/05	JWK	
3.	FDOT BRIDGE RAIL	Sheet No.
4.	TASK 3	1 of 8
5.		

Mar 14, 2005 - 1:14pm

Figure 1. Strain Gage Layout for Florida F-Shape Bridge Railing.

SECTION A-A
TYPICAL SECTION THRU TRAFFIC RAILING BARRIER
(0.8 IN^2 PER FT. TENSION STEEL)

Figure 2. Cross Section for Tests A and B.

1'-6"
7 1/4" — 10 3/4"
5" — 2 1/4"
2" CLEAR (TOP)

BARS 5P @ 8" SP.

3 3/4"
4 3/4"

BARS 5S (TYP.)

1'-0"

3" CLEAR

1'-10"
3 1/2"
1"

2'-8"

3 1/2"
10" R
CONSTRUCTION JOINT REQUIRED

1 1/4"
1 1/4"

3" CLEAR

BARS 5V @ 8" SP.

7"

2" CLR.
8"

3"
1 1/2"

1 3/4"

APRON

BARS 5S

2" CLR.

6"
8"

#4 BARS @ 12" O.C. TOP & BOTTOM

5 1/2"

3"

2" CLR.

6 STRAIN GAGES ON BARS IN LOADING AREA

6 STRAIN GAGES ON BARS IN LOADING AREA

6"
6 1/2"

#5 BARS @ 8" O.C. TOP & BOTTOM

3'-6"

SECTION B–B
TYPICAL SECTION THRU TRAFFIC RAILING BARRIER
(0.47 IN^2 PER FT. TENSION STEEL)

Figure 3. Cross Section for Test C.

1'-6"

7 1/4" — 10 3/4"

5" — 2 1/4"

2" CLEAR (TOP)

BARS 5P @ 8" SP.

3 3/4"

4 3/4"

BARS 5S (TYP.)

1'-0"

3" CLEAR

2'-8"

1'-10"

3 1/2"

1"

1 1/4"

1 1/4"

3 1/2"

10" R
CONSTRUCTION JOINT REQUIRED

3" CLEAR

BARS 5V @ 8" SP.

7"

8"

1 3/4"

2" CLR.

BARS 5S

3"

APRON

6"

8"

2" CLR.

#4 BARS @ 12" O.C. TOP & BOTTOM

2" CLR.

3"

6" — 6 1/2"

2" CLR.

#5 BARS @ 6" O.C. TOP & BOTTOM

3'-6"

SECTION C-C
TYPICAL SECTION THRU TRAFFIC RAILING BARRIER
(0.62 IN^2 PER FT. TENSION STEEL)

The Texas A&M University System

Revisions			TEXAS TRANSPORTATION INSTITUTE
No.	Date	By	COLLEGE STATION, TEXAS 77843
1.	05/17/04	JWK	

Project No.	Date	Drawn By	Scale
421323	02/04	JWK	

FDOT BRIDGE RAILS -- TASK 3

Sheet No.
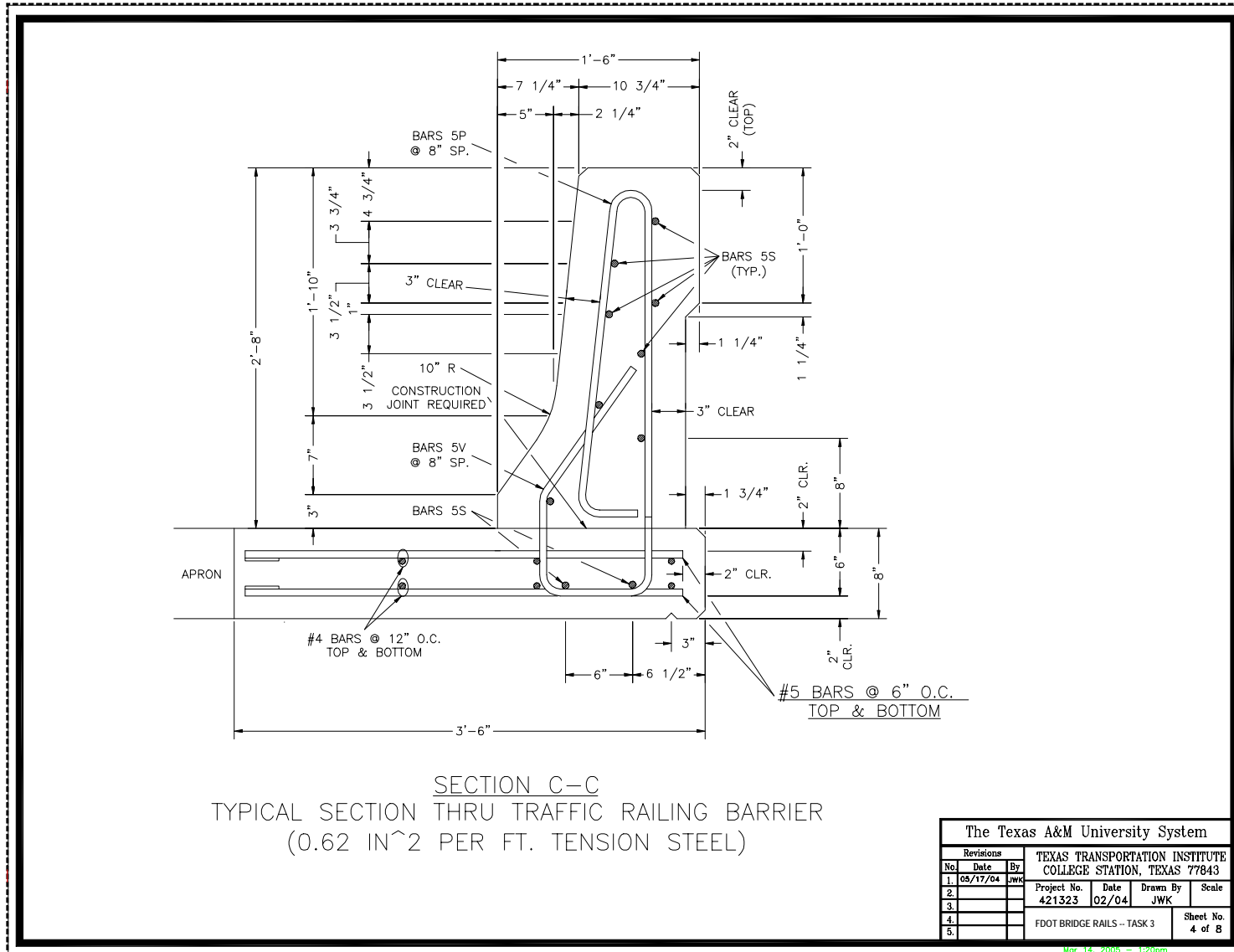4 of 8

Mar 14, 2005 — 1:20pm

Figure 4. Section C-C of Florida F-shape Bridge Rail.
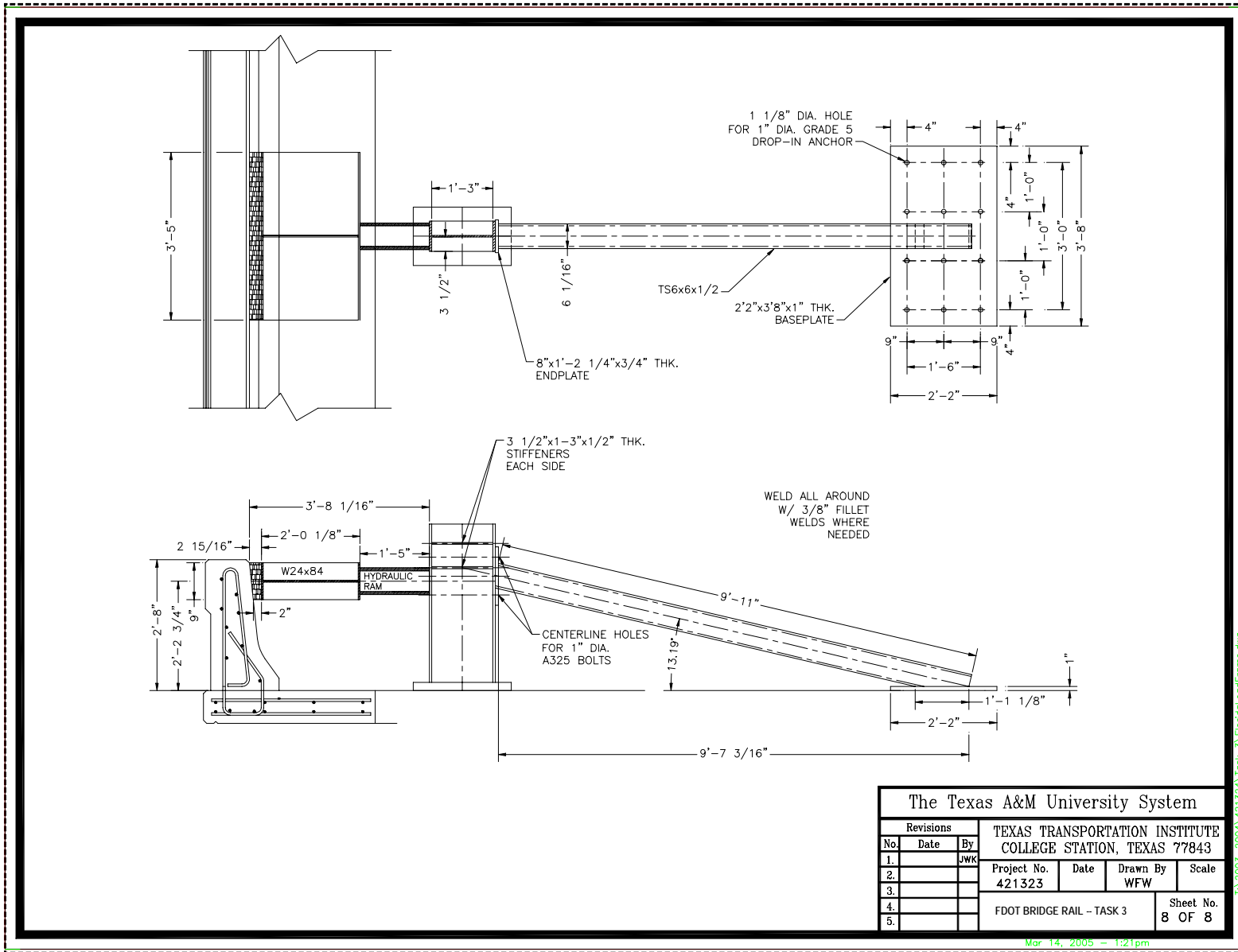(No static load tests on this section.)

Figure 5. Load Frame Setup.

9

CHAPTER 3. STATIC LOAD TESTS

STATIC LOAD TESTS ON SAFETY SHAPED TEST INSTALLATION

The *AASHTO LRFD Bridge Design Specifications* design method analysis indicated the potential for poor performance in the full-scale crash test. Static tests replicating the loads used in the design procedure in *AASHTO LRFD Bridge Design Specifications* were performed to verify actual capacities of the bridge parapet. The static load tests were performed with a hydraulic ram attached to a braced load frame, pushing on a load cell, and placed against a spreader beam, W12×50 (W310×74), 42 inches (1067 mm) long. A wood bearing surface, tapered to match the slope of the traffic face of the parapet, was placed on the spreader beams. It also minimized stress concentrations due to surface imperfections in the parapet. The test setup is shown in Figure 6.

Test A

Test C

Figure 6. Test Setup for Static Load Testing.

11

Test A

Test A was performed on the end of an F-shape barrier segment to determine the actual strain in the 5V bars located at the end of the parapet. Strain data was obtained in six top transverse bars in the deck and in six vertical bars in the traffic face of the barrier. The application of loading on the end of the barrier was intended to represent the loading at a construction joint (see Figures 1 and 7 for details of strain gauge locations). The maximum load attained was approximately 64 kips (285 kN). The anticipated load from the yield line analysis was approximately 73 kips (325 kN). The results of the static test A are shown in Figures 8 and 9. The failure mode is shown in Figure 10. Highest values of strain were recorded at locations A3 and A4 of the vertical bars and A7 of the horizontal deck bars where cracking propagated through the parapet and end of the deck. Maximum recorded micro-strain was 2355 giving a computed stress (assuming elastic behavior) of 68.3 ksi (471 MPa) in the rebar. Data from location A2 was not obtained.
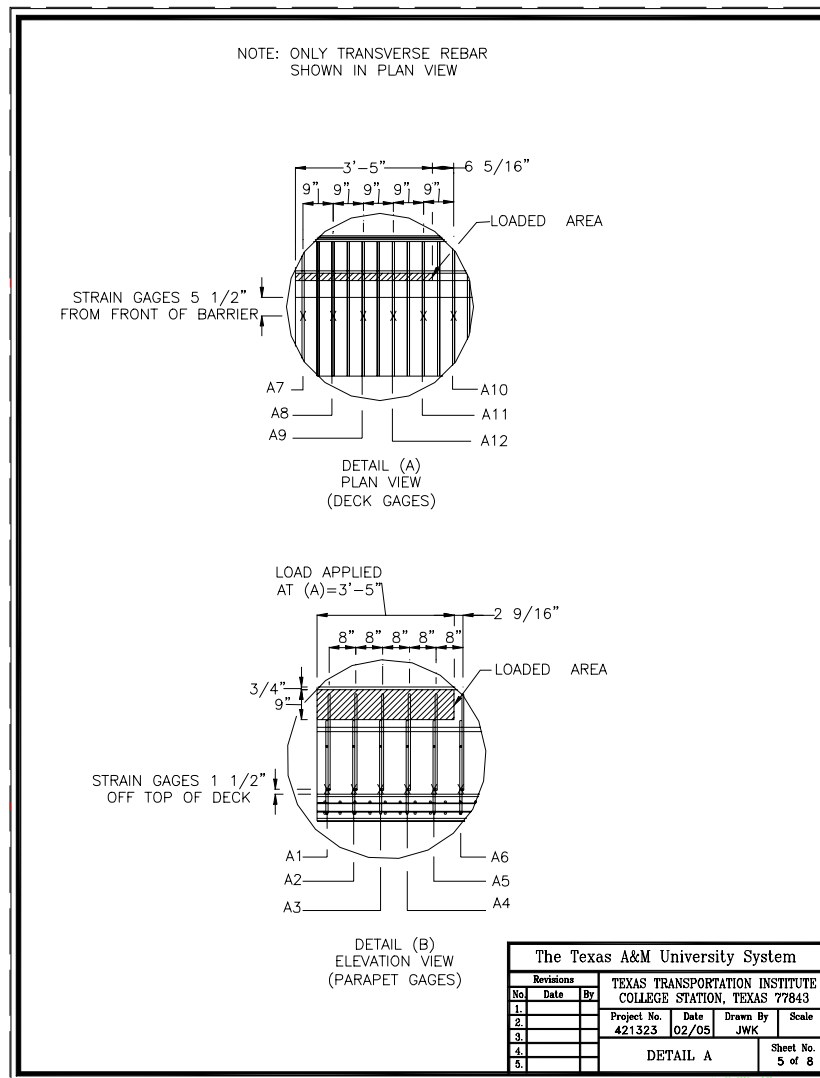

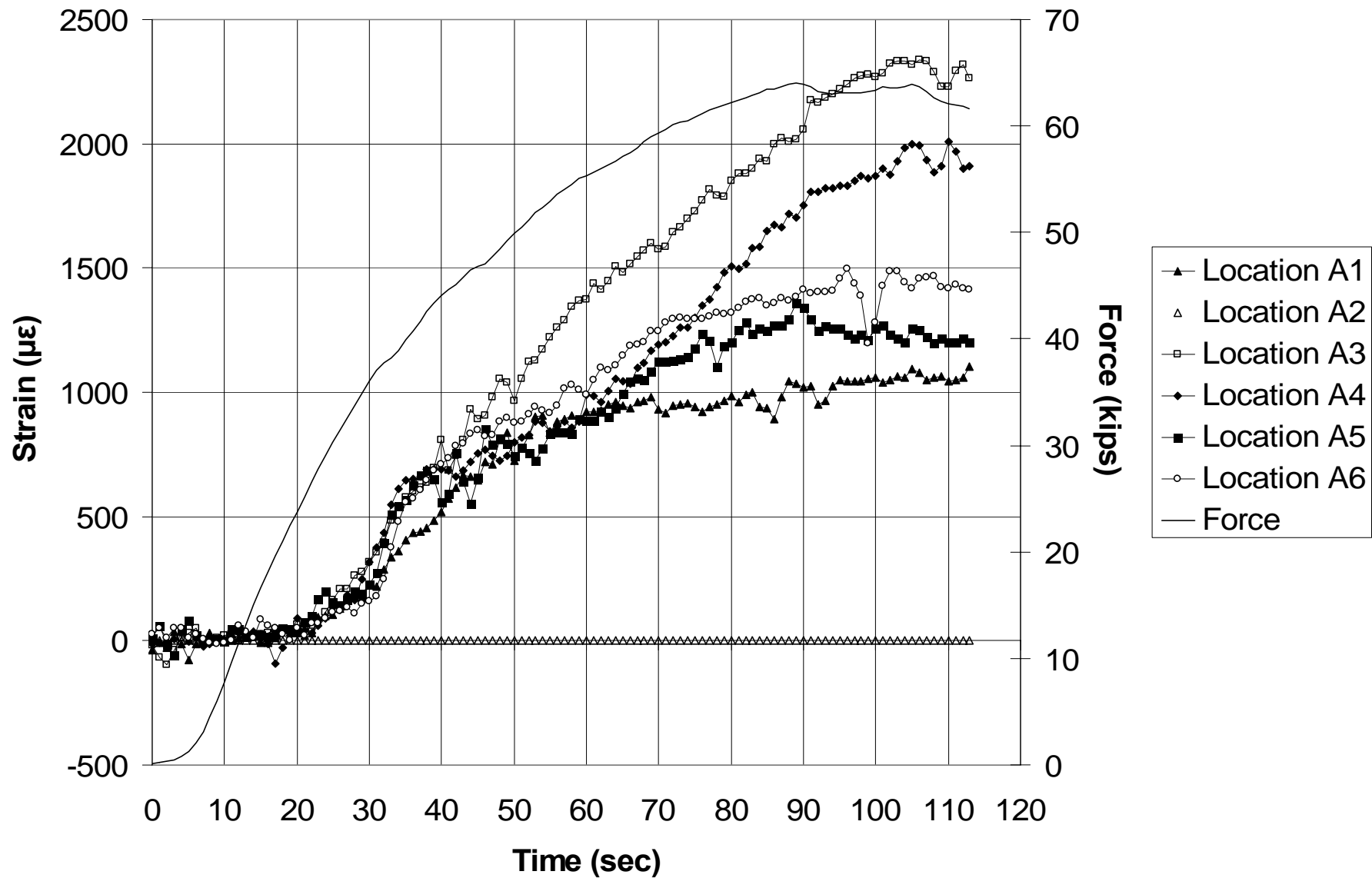**Figure 7. Strain Gauge Details for Test A.**

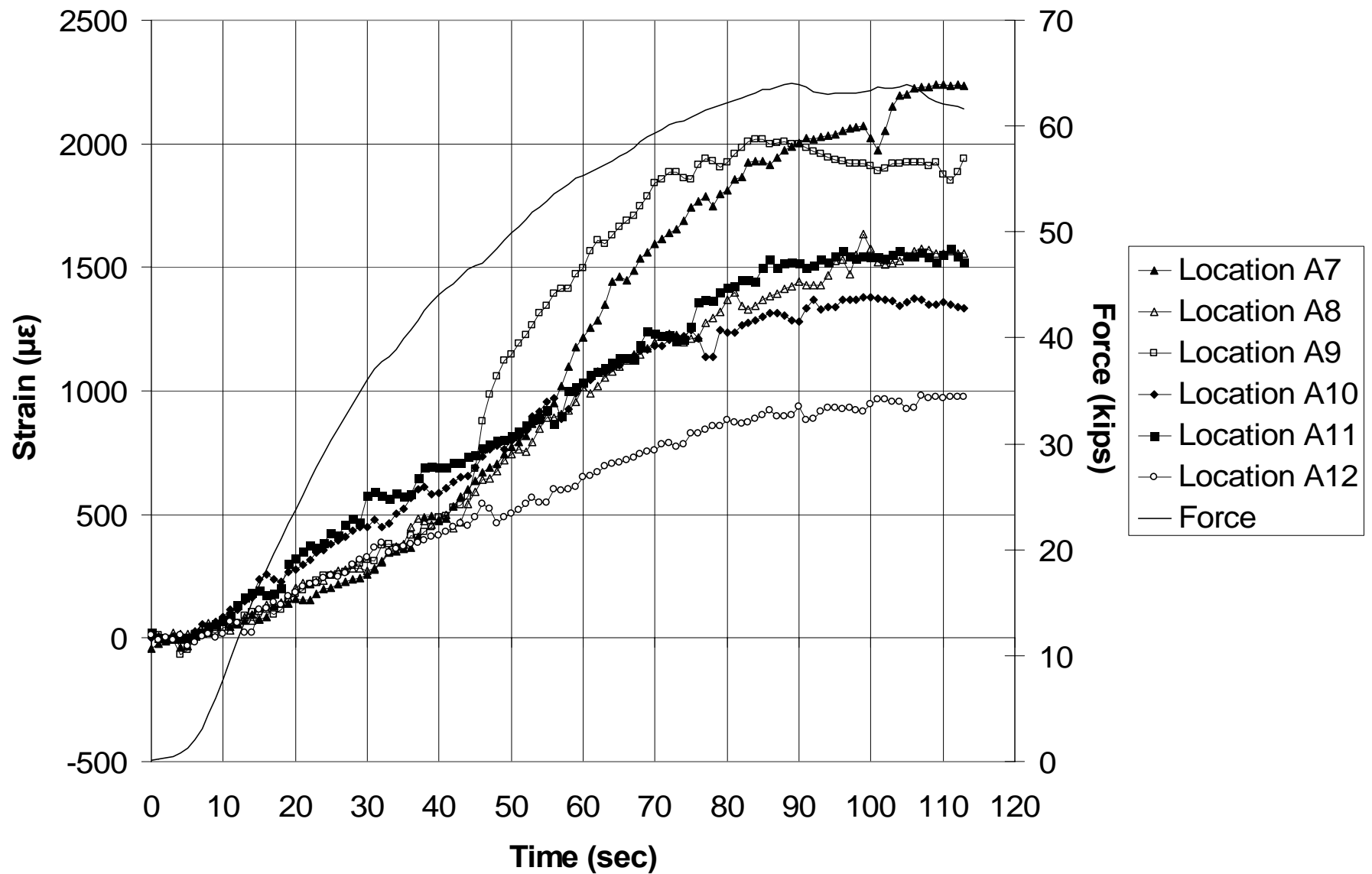**Figure 8. Strain Data for Reinforcing Steel in Traffic Face of Parapet for Test A.**

Figure 9. Strain Data for Reinforcing Steel in Top of Deck for Test A.

Figure 10. Failure mode for Test A.

Test B

Test B was performed at the mid-span location of the F-shape barrier segment tested in Test A. At this location, the transverse reinforcement in the top of the deck was spaced 4 1/2 inches (114 mm) on center. Strain data was obtained in six top transverse bars in the deck and in six vertical bars in the traffic face of the barrier. The application of loading on the barrier was intended to represent the loading in the middle of a barrier section away from the influence of a joint or end conditions (see Figures 1 and 11 for details of strain gauge locations). The maximum load attained was approximately 104 kips (462 kN). The anticipated load from the yield line analysis was approximately 104 kips (462 kN). The results of the static test B are shown in Figures 12 and 13 below. The failure mode is shown in Figure 14. Maximum recorded micro-strain was 1778 giving a computed stress of 51.6 ksi (356 MPa) in the rebar. None of the cracking occurred near any of the strain gages.
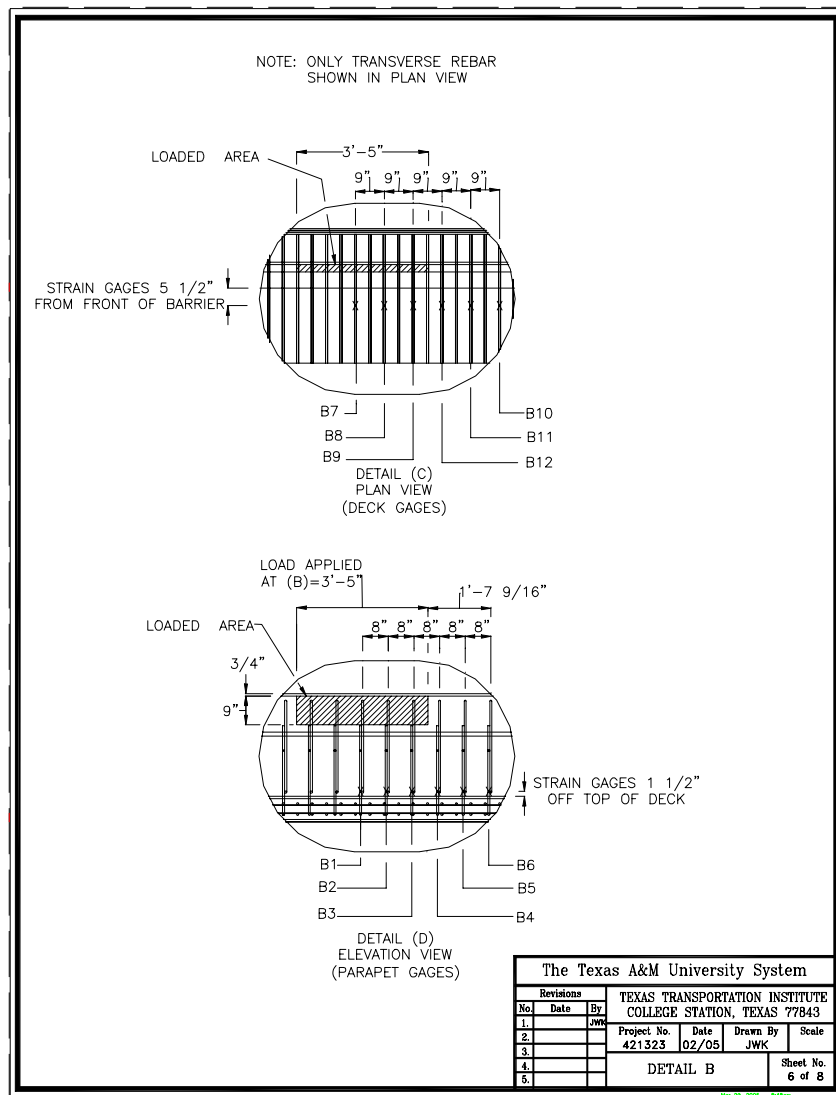

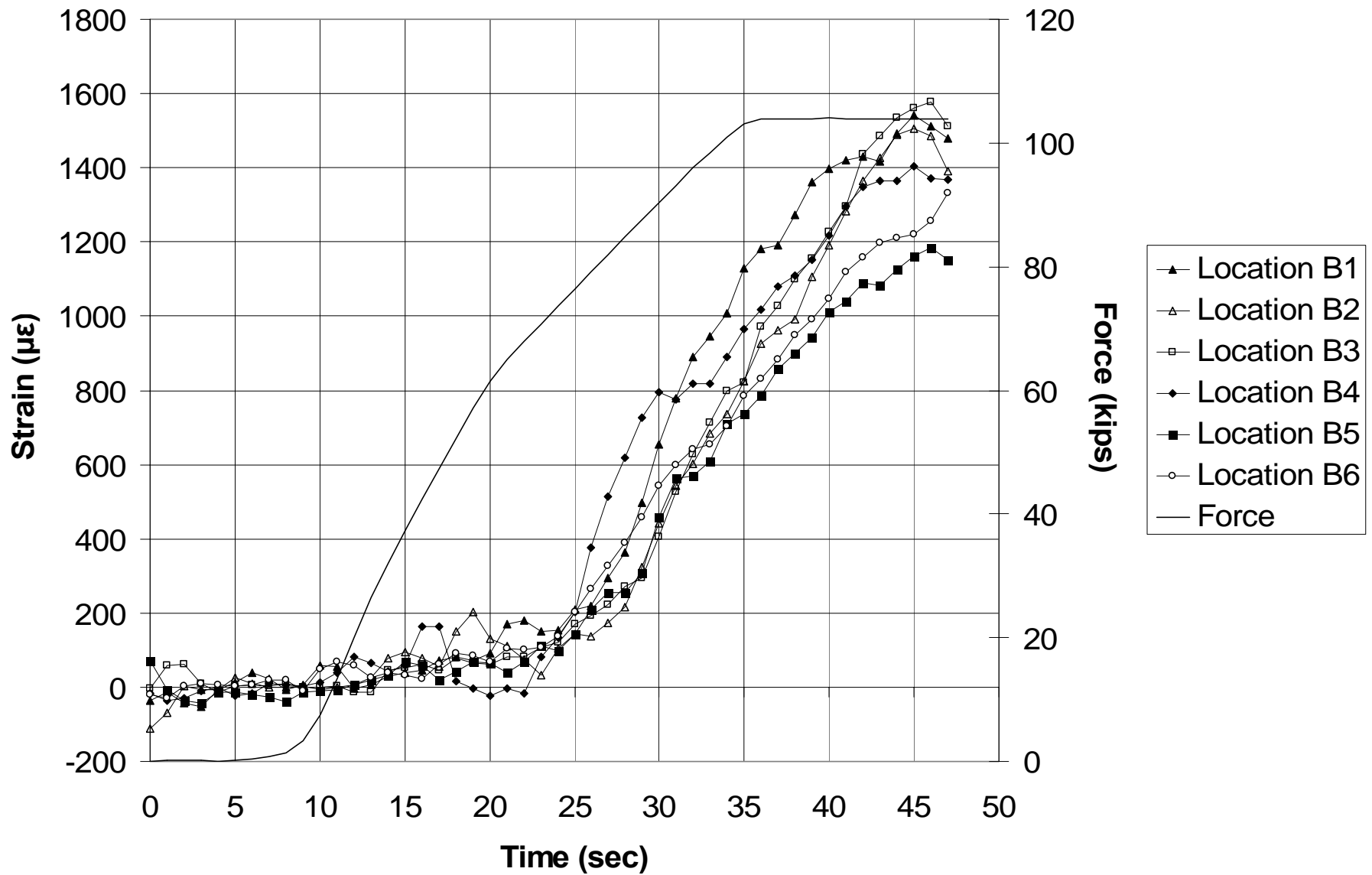**Figure 11. Stain Gauge Details for Test B.**

Figure 12. Strain Data for Reinforcing Steel in Traffic Face of Parapet for Test B.
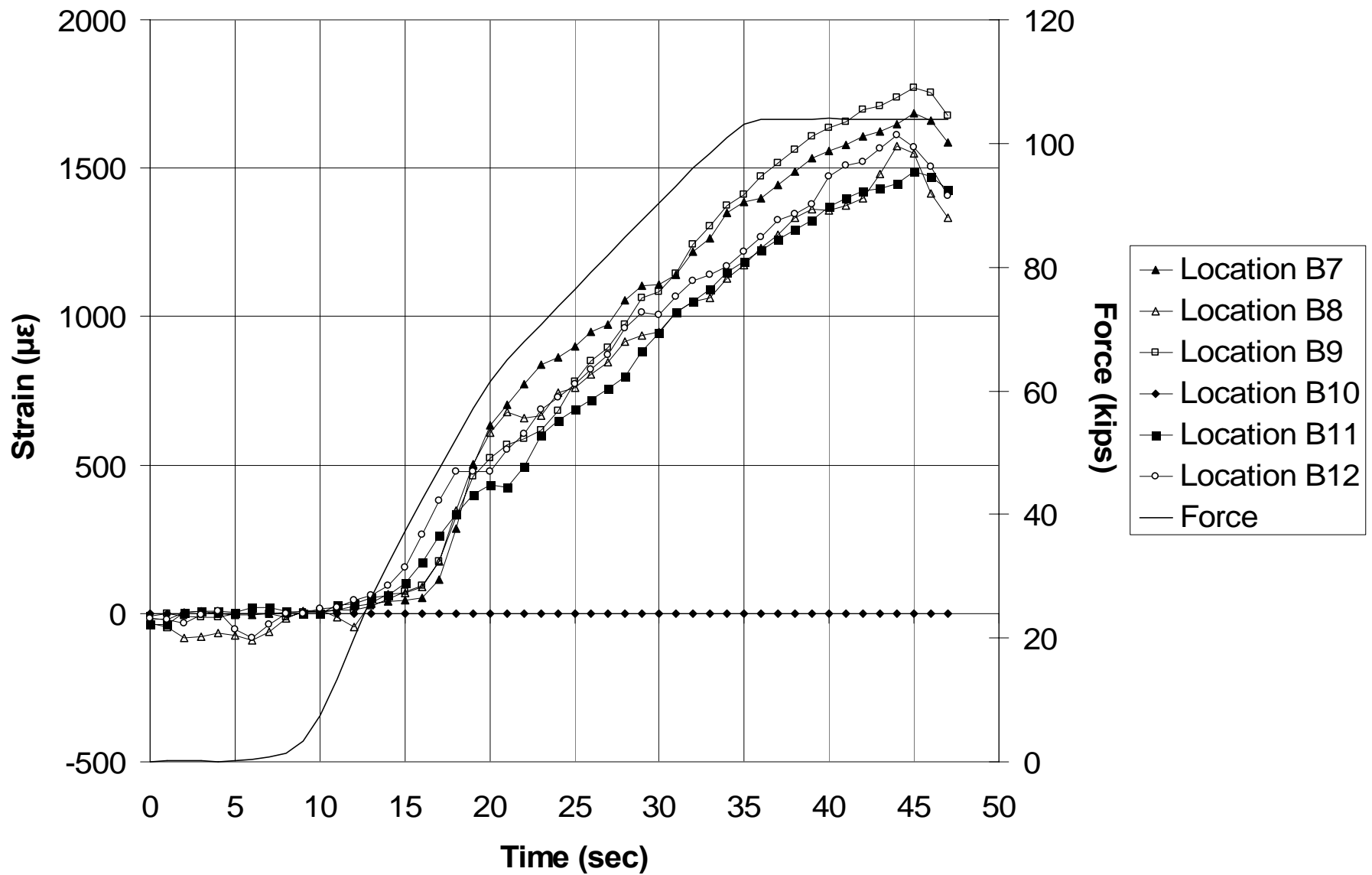
Figure 13. Strain Data for Reinforceing Steel in top of Deck for Test B.

Figure 14. Failure mode for Test B.

Test C

Test C was performed at the mid-span location of the F-shape barrier segment supported by the 8-inch (203 mm) thick deck with the transverse reinforcement spaced on 8-inch (203 mm) centers. Like the previous tests, strain data was obtained in six top transverse bars in the deck and in six vertical bars in the traffic face of the barrier. The application of loading on the barrier was intended to represent the loading in the middle of a barrier section away from the influence of a joint or end conditions (see Figures 1 and 15 for details of strain gauge locations). The maximum load attained was approximately 104 kips (462 kN). The anticipated load from the yield line analysis was approximately 104 kips (462 kN). Maximum micro-strain in the parapet reinforcement was 1688 giving a computed stress of 49.0 ksi (338 MPa) in the rebar while a maximum micro-strain in the deck reinforcement was 2364 for a computed stress of 68.6 ksi (473 MPa). The results of the static test C are shown in Figures 16 and 17 below. The failure mode is shown in Figure 18. Data at location C9 was corrupted.

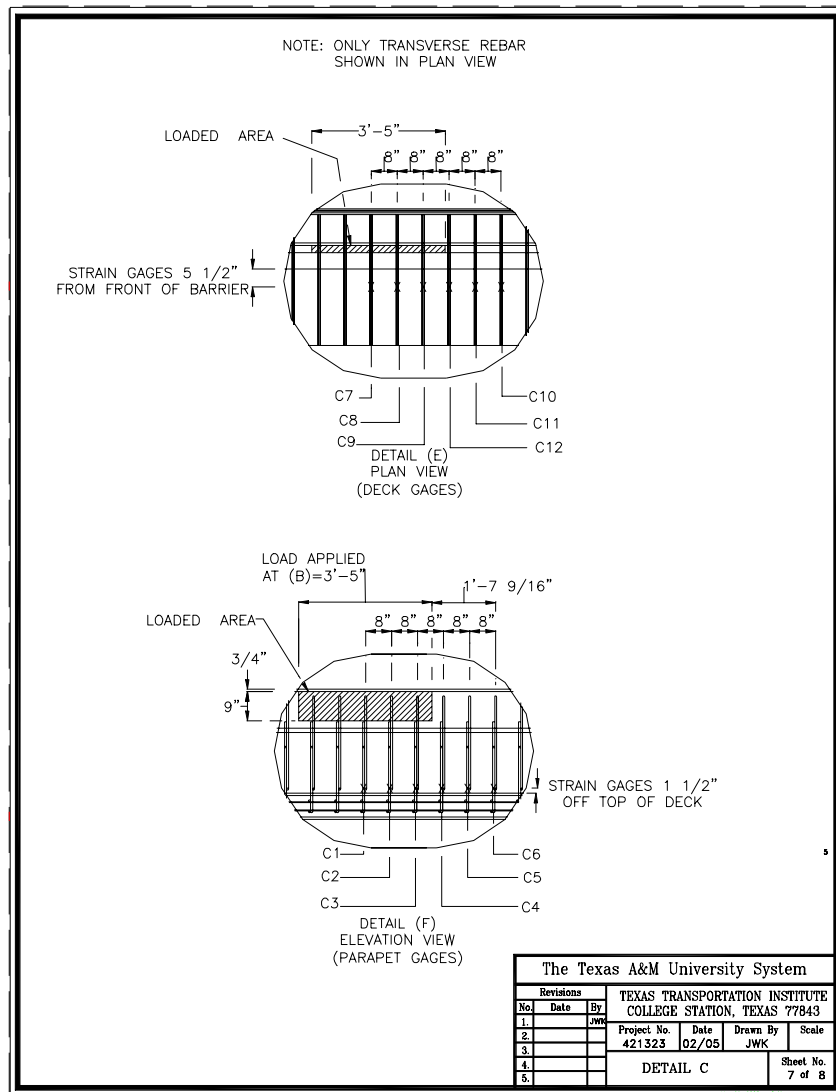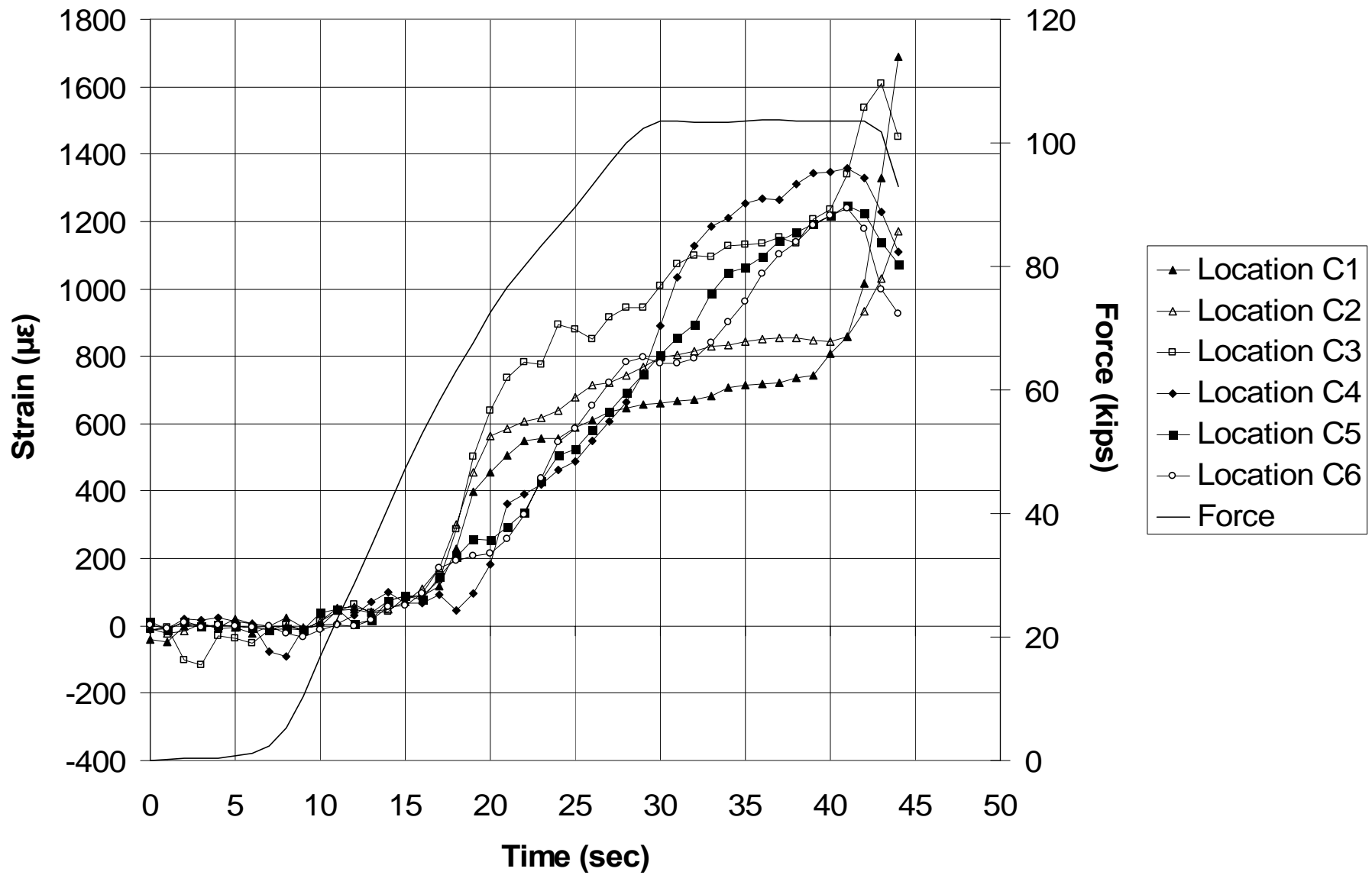**Figure 15. Stain Gauge Details for Test C.**

Figure 16. Strain Data for Reinforcing Steel in Traffic Face of Parapet for Test C.
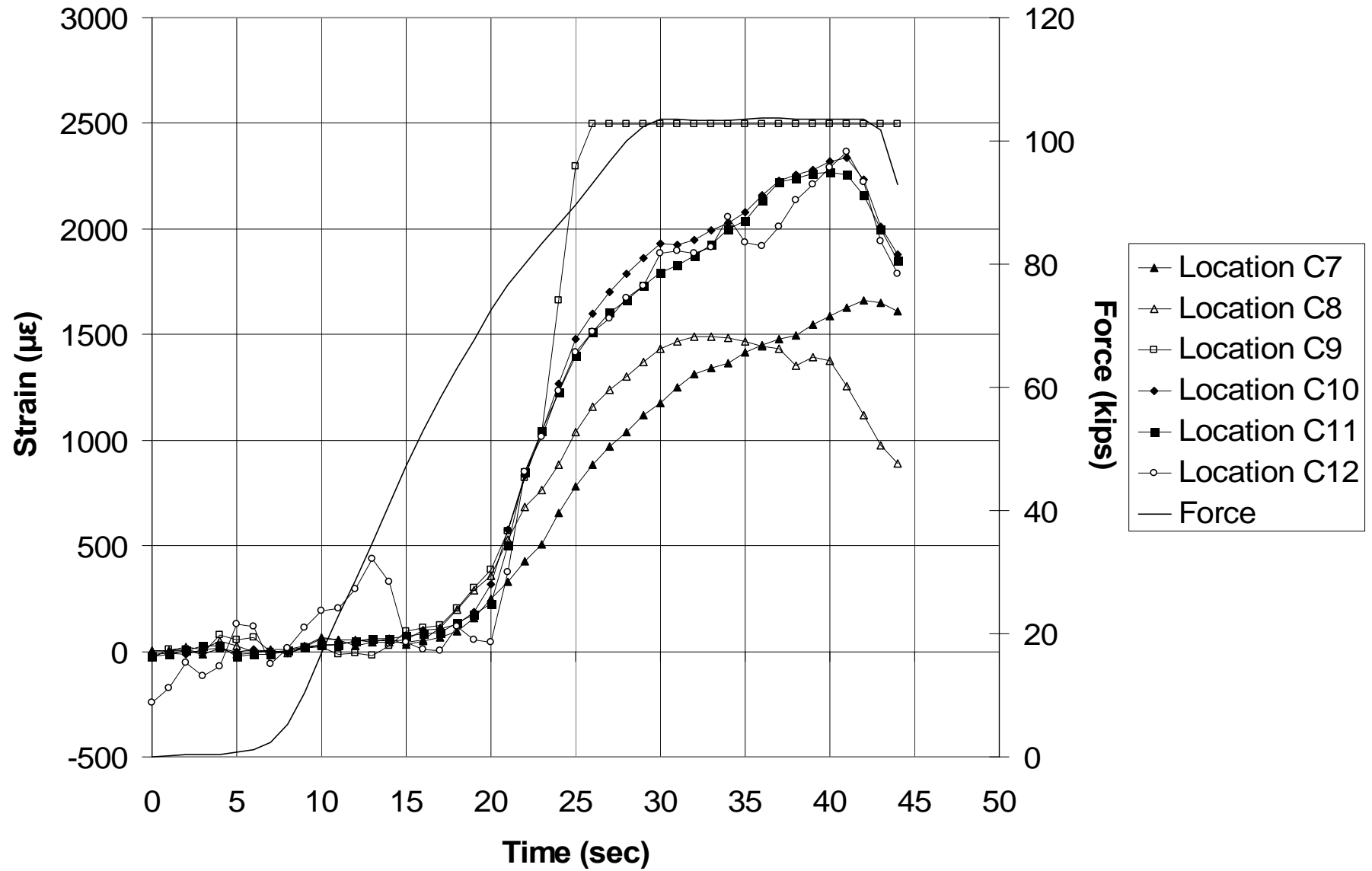
Figure 17. Strain Data for Reinforcing Steel in Top of Deck for Test C.

Figure 18. Failure mode for Test C.

CHAPTER 4. ANALYSIS OF STATIC TEST RESULTS

TEST RESULTS FOR DECK OVERHANG DESIGN ACCORDING TO *AASHTO LRFD BRIDGE SPECIFICATIONS*

Information obtained from the strain gage testing program has been reviewed and analyzed as part of this project. This information is also provided in graphical form in Chapter 3. As previously stated, strain gages were located on bars in the parapet and top layer of tension reinforcement in the deck to determine the actual generated force transferred to the structural elements during loading of the barrier. The static loading applied to the barriers was performed in accordance with the information provided in Table A13.2-1, Section 13, *AASHTO LRFD Bridge Design Specifications, 2000 Interim*. The load was applied to the barrier systems via a hydraulic ram. The loading from the hydraulic ram was increased at a slow rate until failure of the barrier and deck supporting system occurred. Based on previous research, to develop the yield line failure mechanism as shown in *AASHTO LRFD Bridge Design Specifications*, Section A13.3.1, the flexural resistance of the deck per unit length should meet or exceed the flexural resistance of the concrete parapet at its base.

SUMMARY OF STATIC TEST RESULTS

Test A Results ~ Load Applied to F-Shape Barrier End Case with #5 (16) Transverse Reinforcement in Deck on 4 1/2-inch (114 mm) Centers

The ultimate load applied to the barrier was approximately 64 kips (284 kN) just prior to failure of the barrier concrete. When the maximum load on the barrier was initially reached, the recorded strain in the vertical 5V bars varied from approximately 1020 micro-strain to approximately 2060 micro-strain. At the same time, the recorded strain in the transverse deck reinforcement varied from 934 micro-strain to 2014 micro-strain. Cracking of the concrete barrier was observed and this cracking extended down into the concrete deck at the joint. Considering the cross-sectional area of the reinforcement and the modulus of elasticity of the steel material of 29,000,000 psi (200,000 MPa), the calculated force in the 5V bars varied from approximately 9 kips (40 kN) to 18 kips (80 kN). Based on the yield strength of the steel material equal to 60,000 psi (414 MPa), the calculated force to yield a single 5V bar in tension is 18.6 kips (82.7 kN). Highest values of strain were recorded at locations A3 and A4 of the vertical bars and A7 of the horizontal deck bars where cracking propagated through the parapet and deck end. Maximum recorded micro-strain in the top transverse deck reinforcement was 2355 giving a computed stress of 68.3 ksi (471 MPa) in the rebar.

In the static testing performed on the F-shape barrier with the transverse reinforcement closely spaced at 4 1/2 inches (114 mm) on centers (Test A), cracking did propagate into the deck from the testing performed on the end of the barrier segment and deck. However, the ultimate load applied to the barrier greatly exceeded the required load specifications for Test Level 4 loading conditions as stated in *AASHTO LRFD Bridge Design Specifications*. The nominal moment strength of the deck was 21.3 kip-ft per ft (94.8 kN-m per meter) and the

nominal moment strength at the base of the barrier was 28.8 kip-ft per ft (128.1 kN-m per meter). The area of reinforcement per foot of deck width used in Test A was 0.8 square inches per foot of deck width (1693 square mm per meter). This amount of top and bottom transverse reinforcement is currently used by Florida DOT for new 8-inch (203 mm) thick decks supporting the F-shape barrier. Using this area of reinforcement, the nominal flexural strength of the deck is approximately 74 percent of the nominal flexural strength at the base of the F-shape barrier. As observed in Test A, the yield line failure mechanism in the barrier did extend down into the concrete deck when excessive loads are applied at an end or joint. However, the parapet and deck will provide adequate containment at the 54 kip (240 kN) design load conditions for TL-3 and TL-4.

Test B Results ~ Load Applied to F-Shape Barrier Mid-Span Case with #5 (16) Transverse Reinforcement in Deck on 4 1/2-inch (114 mm) Centers

The ultimate load applied to the barrier was approximately 104 kips (462 kN) just prior to failure of the barrier concrete. When the maximum load on the barrier was initially reached, the recorded strain in the vertical 5V bars varied from approximately 700 micro-strain to approximately 1200 micro-strain. At the same time, the recorded strain in the transverse deck reinforcement varied from approximately 1200 micro-strain to approximately 1400 micro-strain. Cracking of the concrete barrier was observed at the maximum recorded load applied to the barrier. Cracking was not observed in the concrete deck at the ultimate load applied to the barrier. Considering the cross-sectional area of the reinforcement and the modulus of elasticity of the steel material of 29,000,000 psi (200,000 MPa), the calculated force in the 5V bars varied from approximately 7 kips (31 kN) to 10 kips (45 kN). Based on the yield strength of the steel material equal to 60,000 psi (414 MPs), the calculated force to yield a single 5V bar in tension is 18.6 kips (82.7 kN). Maximum recorded micro-strain in the top transverse deck reinforcement was 1778 giving a computed stress of 51.6 ksi (356 MPa) in the rebar.

Test B was conducted on the same parapet and deck as used in Test A but the testing was done at the mid-span of the parapet. The recorded load at failure of the parapet was 104 kips (462 kN) and concrete failure was found only in the parapet. Using the associated strain values at the time of parapet failure, the computed moment at the base of the parapet was 18.1 kip-ft per ft (80.5 kN-m per meter). The computed nominal strength of the deck, as previously indicated, was 21.3 kip-ft per ft (94.8 kN-m per meter) and the computed nominal strength of the parapet at the base was 28.8 kip-ft per ft (128.1 kN-m per meter). When the parapet moment value obtained from the strain gages is compared to the design moment capacity of the cantilever deck, the parapet moment produces 85 percent of the deck moment capacity. Additionally, these load conditions are associated with a significant overload situation, 104 kips (462 kN). This load is almost twice the design load of 54 kips (240 kN) typically used in TL-3 and TL-4 parapet designs. Therefore, the current deck design used by FDOT is adequate from both a strength standpoint and a maintenance standpoint when overload situations are encountered at mid-spans. Furthermore, the moment at the base of the parapet obtained from strain data at parapet failure is only 63 percent of the nominal design moment capacity at the base of the FDOT F-shape parapet.

Test C Results ~ Load Applied to F-Shape Barrier Mid-Span Case with #5 (16) Transverse Reinforcement in Deck on 8-inch (203 mm) Centers

The ultimate load applied to the barrier was approximately 104 kips (462 kN) just prior to failure of the barrier concrete. When the maximum load on the barrier was initially reached, the recorded strain in the vertical 5V bars varied from approximately 700 micro-strain to approximately 1000 micro-strain. At the same time, the recorded strain in the transverse deck reinforcement varied from approximately 1200 micro-strain to approximately 1900 micro-strain. Cracking of the concrete barrier was observed at the maximum recorded load applied to the barrier. Cracking was also observed in the concrete deck at the ultimate load applied to the barrier. Considering the cross-sectional area of the reinforcement and the modulus of elasticity of the steel material of 29,000,000 psi (200,000 MPa), the calculated force in the 5V bars varied from approximately 6 kips (26.6 kN) to 9 kips (40 kN). Based on the yield strength of the steel material equal to 60,000 psi (414 MPa), the calculated force to yield a single 5V bar in tension is 18.6 kips (82.7 kN). Maximum micro-strain in the parapet was 1688 giving a computed stress of 49.0 ksi (338 MPa) in the rebar while a maximum micro-strain in the top transverse deck was 2364 for a computed stress of 68.6 ksi (473 MPa).

Based on the results from Test C, the F-shape barrier supported by the concrete deck using 0.47 square inches per ft (995 square mm per meter) of top and bottom transverse steel reinforcement per foot of deck width (significantly less than the 0.8 square inches per ft (1693 square mm per meter) used in Tests A & B) also met the Test Level 4 strength requirements in the *AASHTO LRFD Bridge Design Specifications*. However, concrete cracking in the barrier extended into the deck. Since the ultimate strength of the barrier supported by the deck with 0.47 square inches per ft (995 square mm per m) transverse steel met the strength requirements of TL-4, strength testing of the barrier supported by the decke with 0.62 square inches per ft (1312 square mm per m) transverse steel was not performed. Therefore, based on the results from this study, the Florida F-shape barrier can be supported by a concrete deck with significantly less flexural capacity than the base strength of the barrier and still satisfy the strength requirements of the AASHTO Test Level Four (TL-4) impact conditions.

The deck strength in Test C, with the loading applied within the wall segment (mid-span case) the deck flexural resistance was approximately 45 percent of the flexural resistance of the F-shape barrier at the base. Based on the results from this study, the flexural resistance can be at least one-half the capacity of the flexural resistance of the F-shape barrier at the base and still meet the requirements of TL-4 loading conditions. However, cracking in the deck will likely occur if extreme loading conditions are applied to the barrier that greatly exceed the design force of 54 kips (240 kN) as stated in Table A13.2-1. During Test C, cracking in the barrier and deck was not observed when the applied force on the barrier was 54 kips (240 kN). Therefore, based on this study, deck flexural resistance of the concrete deck within a wall segment (mid-span case) can be limited to 45 percent of the flexural resistance at the base of the F-shape barrier and still meet the strength requirements of AASHTO TL-4. For additional information please refer to the calculations contained in Appendix B.

CHAPTER 5. CONCLUSIONS, IMPLEMENTATION AND FUTURE RESEARCH

CONCLUSIONS

The results of this project clearly indicate that the flexural strength of deck, as designed by FDOT and currently used by FDOT in conjunction with the "F" Shaped Bridge Parapet, is appropriate. The Florida bridge rail and deck designs tested for this project performed acceptably according to the *AASHTO LRFD Bridge Design Specifications* for TL-4 loading conditions. This research indicates that, for safety shaped bridge parapets, the design moment capacity of the deck can be less than the design moment capacity at the base of the safety shape. This research is contrary to the statement found in A13.4.2 in *AASHTO LRFD Bridge Design Specification,* that states "….M_s ….exceeds M_c of the parapet **at its base**." At "end of parapet" or expansion joints in the bridge, the deck flexural capacity should be increased and designed to minimize potential deck damage in "overload" conditions.

IMPLEMENTATION STATEMENT

Based on the results from this study, the flexural resistance of the concrete deck need not meet or exceed the flexural resistance of the concrete barrier at its base as stated in A13.4.2 in *AASHTO LRFD Bridge Design Specification* to meet the design forces for traffic railings as stated in Table A13.2-1. To achieve a crashworthy design, the flexural strength of the deck could be reduced below the flexural strength at the base of the barrier by as much as 45 percent depending flexural resistance of the barrier used in conjunction with flexural strength of the supporting slab. It is recommended that the flexural resistance of the deck meet or exceed the flexural resistance of the barrier base at all joints in the deck for a minimum distance of six feet from the ends of the joint.

FUTURE RESEARCH

Failure modes of the parapets witnessed in this research suggest that the yield line analysis/design procedures used in the *AASHTO LRFD Bridge Design Specifications* should be revisited. At center span load applications to failure in the parapet, the center vertical yield line was never produced. Further review of the damaged zones showed 45 degree shear planes from the load application region. This suggests punching shear may be a more appropriate method of analysis for center span failure of strong concrete parapets.

REFERENCES

1. American Association of State Highway and Transportation Officials, *LRFD Bridge Design Specification*, AASHTO Subcommittee on Bridges and Structures, Washington, D.C., May 2001

2. K. K. Mak and W. L. Campise*, Test and Evaluation of Ontario "Tall Wall" Barrier with an 80,000-Pound Tractor-Trailer*, Texas Transportation Institute, Contract No. 4221-9089-534 for Ontario Ministry of Transportation, Texas Transportation Institute, College Station, TX, 1990.

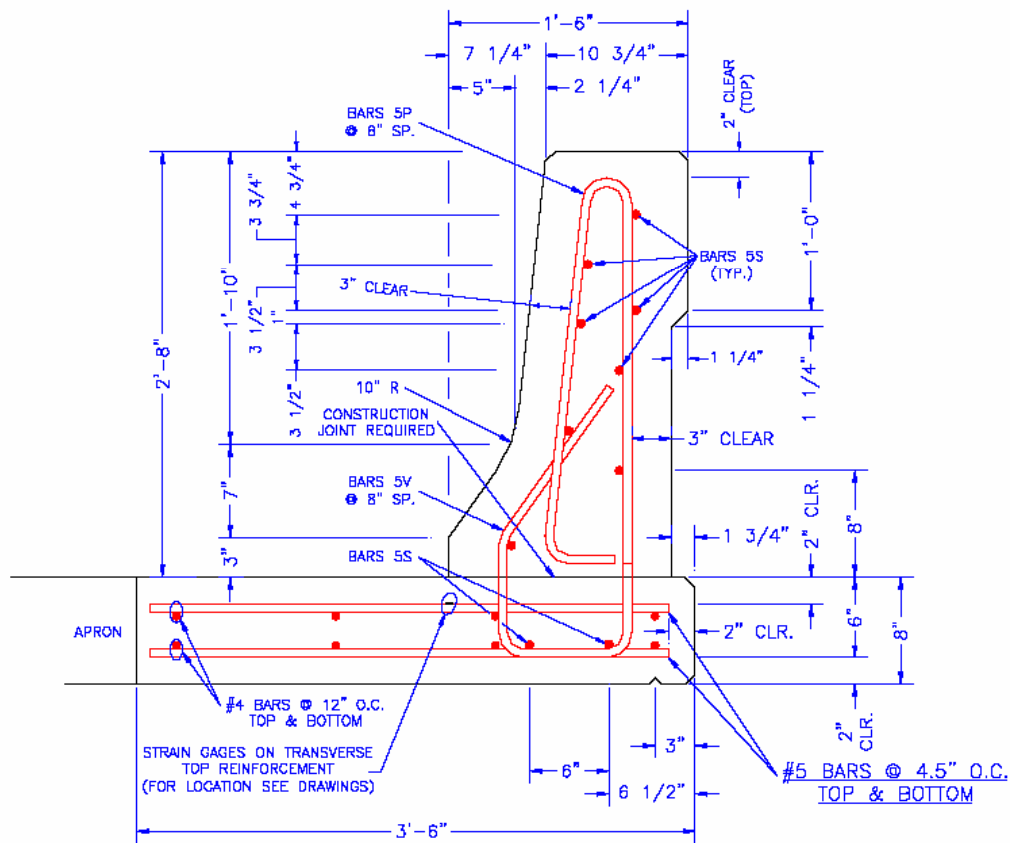# APPENDIX A. YIELD LINE ANALYSIS OF FDOT BRIDGE RAIL

Texas Transportation Institute

PAGE **1** of **8**

JOB NO. **421323**

Date: **06-20-03**

SUBJECT **Florida Bridge Deck Analysis**

Impact Within Wall Segment

BY: **W. Williams**

CKD: _____

CLIENT **FDOT**

1.) Given Information:

1.) Florida DOT 32-inch F-shape barrier details with the 3 different deck cross section details (different transverse steel reinforcement in the deck). Find ultimate strength of barrier in accordance with AASHTO LRFD, Section 13, Specifications:
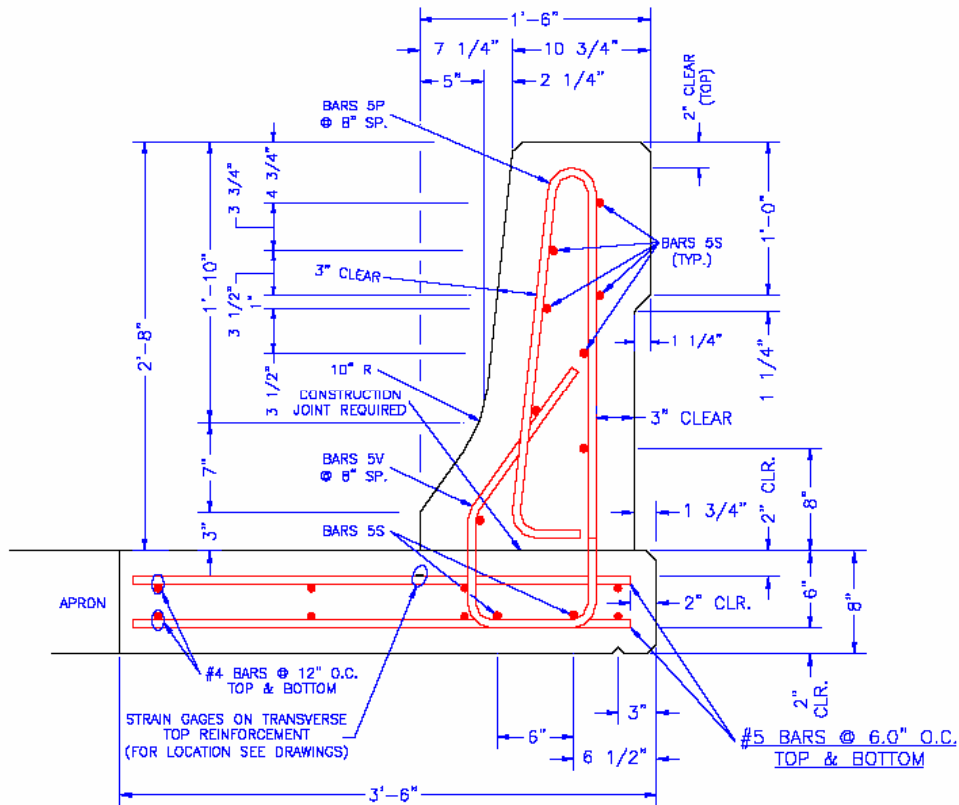
SECTION A-A
TYPICAL SECTION THRU TRAFFIC RAILING BARRIER
(0.8 IN^2 PER FT. TENSION STEEL)

33

Texas Transportation Institute

SUBJECT **Florida Bridge Deck Analysis**

Impact Within Wall Segment

BY: W. Williams

CKD:

CLIENT **FDOT**

BARS 5P @ 8" SP.

BARS 5S (TYP.)

3" CLEAR

10" R

CONSTRUCTION JOINT REQUIRED

BARS 5V @ 8" SP.

BARS 5S

APRON

2" CLEAR (TOP)

1'-6"
7 1/4"
10 3/4"
5"
2 1/4"
3 3/4"
4 3/4"
1'-10"
2'-8"
3 1/2"
1"
3 1/2"
7"
3"
1'-0"
1 1/4"
1 1/4"
1 1/4"
3" CLEAR
2" CLR.
8"
1 3/4"
2" CLR.
6"
8"
2" CLR.
2" CLR.

#4 BARS @ 12" O.C. TOP & BOTTOM

STRAIN GAGES ON TRANSVERSE TOP REINFORCEMENT (FOR LOCATION SEE DRAWINGS)

#5 BARS @ 6.0" O.C. TOP & BOTTOM

6"
3"
6 1/2"
3'-6"

SECTION B-B
TYPICAL SECTION THRU TRAFFIC RAILING BARRIER
(0.62 IN^2 PER FT. TENSION STEEL)

34

Texas Transportation Institute

SUBJECT **Florida Bridge Deck Analysis**

Impact Within Wall Segment
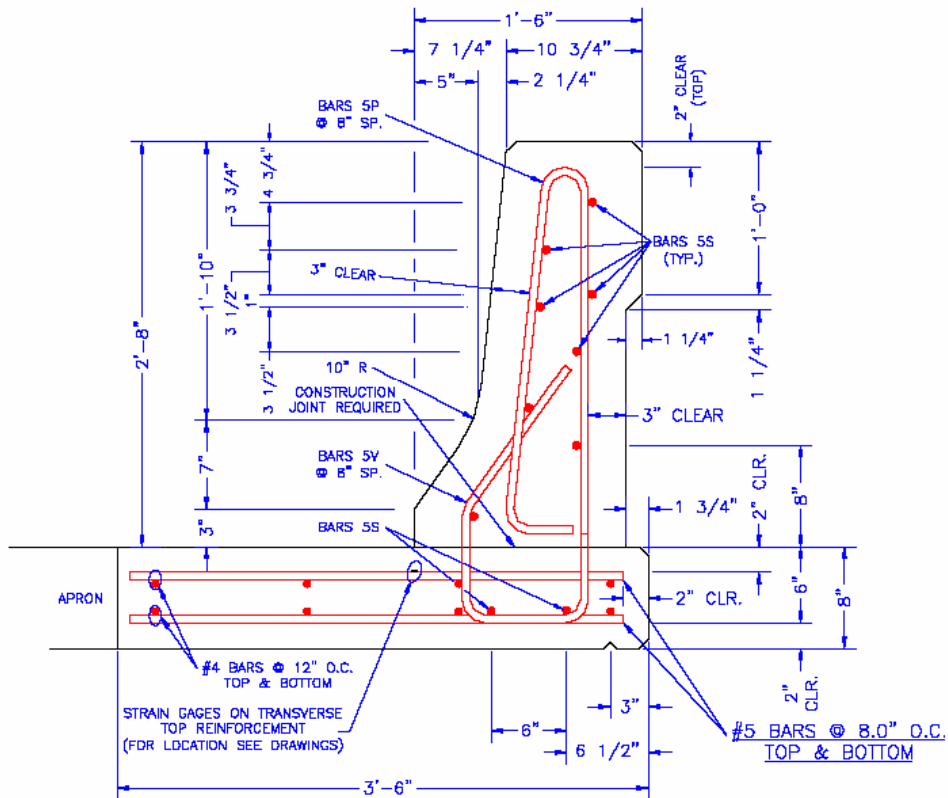
CLIENT **FDOT**

BY: W. Williams

CKD:

SECTION C—C
TYPICAL SECTION THRU TRAFFIC RAILING BARRIER
(0.47 IN^2 PER FT. TENSION STEEL)

SUBJECT __Florida Bridge Deck Analysis__
__Impact Within Wall Segment__

BY: __W. Williams__
CKD: _____

CLIENT __FDOT__

2.) Design Information & Material Properties:

$TL3_{load} := 54 kips$

$f'_{crail} := 4000 psi$ — Compressive strength of rail concrete

$f'_{cdeck} := 4500 psi$ — Compressive strength of deck concrete

$f_y := 60 ksi$ — Yield strength of reinforcing steel

$L_t := 3.5 ft$ — Distributed load length for Test Level 4 Conditions

$H_w := 32 in$ — ... Height of barrier

$Area_{rail} := 2.78 ft^2$ — Area of F-shape from AutoCad

$Area_{deck} := 8 in \times 42 in$ $Area_{deck} = 2.33\ ft^2$ — Area of the deck for volume purposes

$Slab_{thk} := 8 in$ — Thickness of the deck (inches)

$\gamma_{concrete} := 150 pcf$ — Unit weight of concrete (lb/ft^3)

36

Texas
Transportation
Institute

SUBJECT **Florida Bridge Deck Analysis**

Impact Within Wall Segment

BY: **W. Williams**

CKD: _____

CLIENT **FDOT**

3.) Calculate the Bending Capacity of the Rail about the Longitudinal Axis: M_c, (K-F/F):

$f_y = 60\,ksi$ Yield strength of rebar

$f'_{crail} = 4000\,psi$ Compressive strength of concrete

$A_{sc} := 0.31\,in^2 \times \dfrac{12}{8}$ Vertical #5 bars @ 8" O.C. $A_{sc} = 0.47\,in^2$

$b_c := 12\,in$ Unit width of wall, (in)

$a_c := \dfrac{A_{sc} \times f_y}{0.85 \times f'_{crail} \times b_c}$ $a_c = 0.68\,in$

$d_c := 11.75\,in - 3\,in - \dfrac{5}{16}\,in$ "Use parapet width at point of intersecting lines in the bottom portion of the barrier.

$d_c = 8.44\,in$

Calculate M_c:

with: $d_c = 8.44\,in$ $\phi := 1.0$

$A_{sc} = 0.47\,in^2$

$f_y = 60\,ksi$

$a_c = 0.68\,in$

Therefore:

$M_c := \dfrac{\left(\phi \times A_{sc} \times f_y\right) \times \left(d_c - \dfrac{a_c}{2}\right)}{ft}$ Ratio M_c to include 12-inch wall unit length.

$M_c = 18.82\,\dfrac{kips \times ft}{ft}$ Flexural resistance of 12-inch width specified in Article A13.4.2 (k-ft/ft)

37

**Texas
Transportation
Institute**

SUBJECT **Florida Bridge Deck Analysis**

Impact Within Wall Segment

CLIENT **FDOT**

BY: **W. Williams**

CKD: _____

4.) Calculate the Bending Capacity of the Wall about the Vertical Axis: M_w, (K-F):

$$A_{sw} := 0.2in^2 \times 5$$ ... Use 5 longitudinal tension bars in the over the full height of the barrier.

$$f_y = 60\,ksi$$

$$f'_{crail} = 4 \times 10^3\,psi$$

$$H_w = 32\,in$$

$$b_c = 1\,ft$$

$$a_w := \frac{A_{sw} \times f_y}{0.85 \times f'_{crail} \times b_c}$$

$$a_w = 1.47\,in$$

Determine d_w from weighted average from distances measured in AutoCad:

$$d_1 := 6in \qquad d_3 := 7.125in \qquad d_5 := 7.875in \qquad d_2 := 4in \qquad d_4 := 4in$$

$$d_w := \frac{d_1 + d_2 + d_3 + d_4 + d_5}{5}$$

..weighted average for the 5 longitudinal bars in parapet (neglect bar at the base on traffic side)

$$d_w = 5.8\,in$$

Calculate M_w: with:

$$d_w = 5.8\,in \qquad\qquad H_w = 2.67\,ft$$

$$A_{sw} = 1\,in^2$$

$$\phi = 1$$

$$f_y = 60\,ksi$$

$$a_w = 1.47\,in$$

Therefore:

$$M_w := \frac{\phi \times A_{sw} \times f_y \times \left(d_w - \frac{a_w}{2} \right)}{H_w}$$

$$M_w = 9.5\,ft^{-1}\,kips \times ft$$

Flexural resistance of wall in (k-ft/ft)

38

Texas Transportation Institute

SUBJECT **Florida Bridge Deck Analysis** BY: **W. Williams**

 Impact Within Wall Segment CKD: _____

CLIENT **FDOT**

5.) Additional Flexural Resistance from Beam Action in Top of Rail: M_b, (K-F):

$$A_{sw} := 0.0\text{in}^2 \times 0 \quad \text{... 0 longitudinal tension bars in beam}$$

$$f_y = 60\,\text{ksi}$$

$$a_w := \frac{A_{sw} \times f_y}{0.85 \times f'_{crail} \times b_c}$$

$$a_w = 0\,\text{in}$$

$$d_w := 19.5\text{in} - 2\text{in} \qquad \text{..distance to longitudinal tension steel in curb}$$

$$d_w = 17.5\,\text{in}$$

Calculate M_b:

with: $\quad d_w = 17.5\,\text{in} \qquad\qquad H_w = 32\,\text{in}$

$\qquad\qquad A_{sw} = 0\,\text{in}^2 \qquad\qquad \phi = 1$

$\qquad\qquad f_y = 60\,\text{ksi}$

$\qquad\qquad a_w = 0\,\text{in}$

Therefore:

$$M_b := \left[\phi \times A_{sw} \times f_y \times \left(d_w - \frac{a_w}{2} \right) \right]$$

$$M_b = 0\,\text{kips} \times \text{ft} \qquad \text{Flexural resistance of beam in top of curb in (k-ft)}$$

39

Texas Transportation Institute

SUBJECT **Florida Bridge Deck Analysis**

Impact Within Wall Segment

BY: **W. Williams**

CKD: _____

CLIENT **FDOT**

6.) Determine Ultimate Resistance of Wall at Mid-Span and at Joint/End:

Calculate Lc ~ Critical length of yield line failure pattern:

with: $M_b = 0 \, kips \times ft$ ... No additional beam action resistance

$M_c = 18.82 \dfrac{kips \times ft}{ft}$ $M_w = 9.5 \dfrac{kips \times ft}{ft}$ Calculated wall strengths

$H := H_w$ $H = 32 \, in$ ... Height of rail, (ft) $L_t = 3.5 \, ft$

Longitudinal length of distribution of impact force, (ft), TL-3 conditions

$$L_c := \frac{L_t}{2} + \sqrt{\left(\frac{L_t}{2}\right)^2 + \left[\frac{8 \times H \times (M_b + M_w \times H)}{M_c}\right]}$$ (Equation A13.3.1-2, pg. A13-6) $L_c = 7.39 \, ft$

$$R_w := \left(\frac{2}{2 \times L_c - L_t}\right) \times \left(8 \times M_b + 8M_w \times H + \frac{M_c \times L_c^2}{H}\right)$$ (Equation A13.3.1-1, pg. A13-6)

$R_w = 104.27 \, kips$ Total transverse resistance of rail, (kips) good ! > 54 kips

@ A Joint....

$$L_{cjoint} := \frac{L_t}{2} + \sqrt{\left(\frac{L_t}{2}\right)^2 + \left[\frac{H \times (M_b + M_w \times H)}{M_c}\right]}$$ $L_{cjoint} = 4.33 \, ft$

$$R_{wjoint} := \left(\frac{2}{2 \times L_c - L_t}\right) \times \left(M_b + M_w \times H + \frac{M_c \times L_c^2}{H}\right)$$ $R_{wjoint} = 72.81 \, kips$ Total transverse resistance of rail, (kips) good! > 54 kips

The Florida DOT F-shape 32-inch barrier meets the requirements of AASHTO, LRFD Specifications for Test Level 4
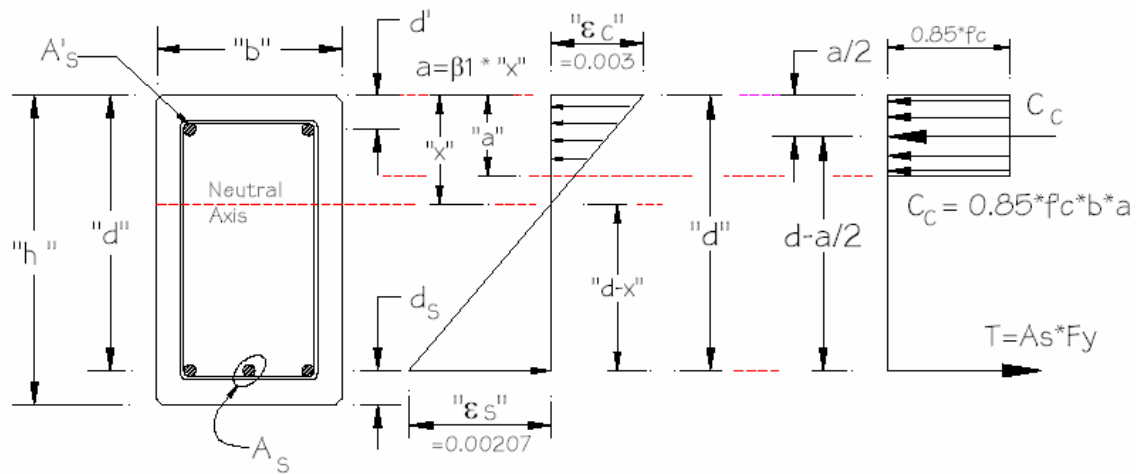
APPENDIX B. BARRIER AND DECK STRENGTH CALCULATIONS

Texas Transportation Institute

SUBJECT __Florida 32" F-Shape__

__Base Strength Mc @ Base__

BY: W. Williams

CKD: _____

CLIENT __FDOT__

1.) Given Design Data:

1.) Worksheet calculates Ultimate and Factored Moments (M_n & M_u) and plots a graph of these values versus area of steel (A_s) for the given geometry and properties of the concrete beam.

2.) Input Items in Yellow including range variable for area of steel (A_s).

3.) Review acceptable ranges of Rho for max and minimum reinforcement allowed.

4.) AutoCad Details: U:*\Beam22.dwg.

$f_c := 4500psi$ $f_y := 60000psi$ $d := 13in - .3125in$ $b := 1 \times ft$ $\phi := 0.9$

$d = 12.688\,in$

$A_s := \left(\dfrac{12}{8} \times 0.31 \times in^2\right)$ $f_{c1} := \dfrac{f_c}{psi}$ $f_{y1} := \dfrac{f_y}{psi}$

41

Texas Transportation Institute

SUBJECT   Florida 32" F-Shape

   Base Strength Mc @ Base

CLIENT   FDOT

BY: W. Williams

CKD: _____

2.) Calculate "ρ_{min}":

$$\rho_{min1} := \frac{3 \times \sqrt{f_{c1}}}{60000} \qquad \rho_{min1} = 3.354 \times 10^{-3}$$

$$\rho_{min2} := \frac{200}{60000} \qquad \rho_{min2} = 3.333 \times 10^{-3}$$

Take the lesser of ρ_{min1} & ρ_{min2}:

$$\rho_{min} := \begin{cases} \rho_{min1} & \text{if } \rho_{min2} > \rho_{min1} \\ \rho_{min2} & \text{otherwise} \end{cases}$$

$$\rho_{min} = 3.333 \times 10^{-3}$$

3.) Calculate "ρ" Actual for A_s:

$$A_s = 0.47 \, in^2$$

These are the calculated Rho for the given steel area provided in the range variable above:

$$\rho(A_s) := \frac{A_s}{b \times d}$$

$$\rho(A_s) = 0.0030542$$

$$kip \equiv 1000 lbf$$

4.) Calculate "ρ_{bal}":

$$lbf \equiv lb$$

Calculate β_1:

$$\beta_1 := if\left[f_c \leq 4000psi, 0.85, 0.85 - 0.05 \times \left(\frac{f_c - 4000psi}{1000psi} \right) \right]$$

$$\beta_1 = 0.825$$

$$\rho_{bal} := \beta_1 \times \left(\frac{0.85 \times f_{c1}}{f_{y1}} \right) \times \frac{87000}{87000 + f_{y1}}$$

$$\rho_{bal} = 0.031 \qquad \rho_{max} := 0.75 \times \rho_{bal}$$

$$\rho_{max} = 0.02335 \qquad \rho_{max} \text{ is 75\% of } \rho_{bal}$$

Texas Transportation Institute

SUBJECT   Florida 32" F-Shape

   Base Strength Mc @ Base

CLIENT   FDOT

BY: W. Williams

CKD: _____

5.) Is "ρ" Good Check:

$$\text{RhoCheck}\left(A_s\right) := \begin{cases} \text{"OK"} & \text{if } \rho_{min} < \rho\left(A_s\right) < \rho_{max} \\ \text{"NG!"} & \text{otherwise} \end{cases}$$

$$\xi\left(A_s\right) := \begin{cases} 1.0 & \text{if } \rho_{min} < \rho\left(A_s\right) < \rho_{max} \\ 0 & \text{otherwise} \end{cases}$$

$$A_s = 0.465 \, in^2$$

$$\rho_{actual} := \frac{A_s}{d \times b}$$

$$\rho_{actual} = 0.00305$$

$$\rho_{min} = 0.00333$$

$$\text{RhoCheck}\left(A_s\right) = \text{"NG!"} \qquad \xi\left(A_s\right) = 0$$

$$\xi\left(A_s\right) := 1$$

Do a "Rho Check" here... discard Rho values below ρ_{min} and above ρ_{max}!

For Values outside of these limits, we will multiply Mn by ζ to make the moment capacity go to "0".

6.) Calculate "a", Nominal Strength (M_n) & Factored Strength (M_u):

$$a\left(A_s\right) := \frac{A_s \times f_y}{0.85 \times f'_c \times b}$$

$$a\left(A_s\right) = 0.608 \, in$$

$$M_n\left(A_s\right) := A_s \times f_y \times \left(d - \frac{a\left(A_s\right)}{2}\right) \times \xi\left(A_s\right)$$

$$M_n\left(A_s\right) = 28.792 \, kip \times ft$$

$$\phi = 0.9$$

$$M_u\left(A_s\right) := M_n\left(A_s\right) \times \phi$$

$$M_u\left(A_s\right) = 25.913 \, kip \times ft$$

Texas Transportation Institute
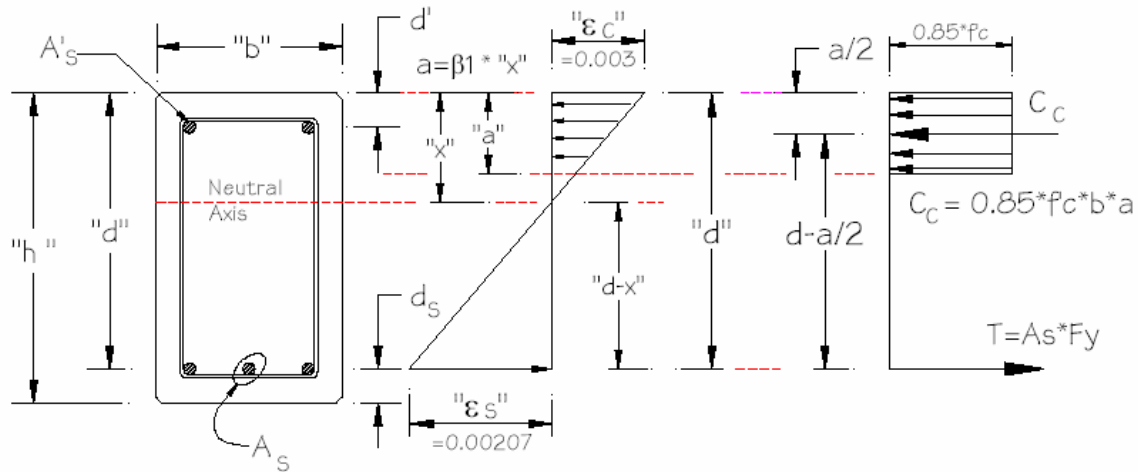
SUBJECT __Florida 32" F-Shape__

__Deck Strength ~ #5's @ 8.0 inches O.C.__

CLIENT __FDOT__

BY: W. Williams

CKD: _____

1.) Given Design Data:

1.) Worksheet calculates Ultimate and Factored Moments (M_n & M_u) and plots a graph of these values versus area of steel (A_s) for the given geometry and properties of the concrete beam.

2.) Input Items in Yellow including range variable for area of steel (A_s).

3.) Review acceptable ranges of Rho for max and minimum reinforcement allowed.

4.) AutoCad Details: U:*\Beam22.dwg.

$f_c := 4500 \text{psi}$ $f_y := 60000 \text{psi}$ $d := 8\text{in} - 2\text{in} - 0.3125\text{in}$ $b := 1 \times \text{ft}$ $\phi := 0.9$

$d = 5.688 \text{ in}$

$A_s := \left(\dfrac{12}{4.5} \times 0.31 \times \text{in}^2 \right)$ ... #5's @ 4.5 inches O.C.

$f_{c1} := \dfrac{f_c}{\text{psi}}$ $f_{y1} := \dfrac{f_y}{\text{psi}}$

Texas Transportation Institute

SUBJECT Florida 32" F-Shape
          Deck Strength

CLIENT   FDOT

BY: W. Williams
CKD: _____

2.) Calculate "ρ_{min}":

$$\rho_{min1} := \frac{3 \times \sqrt{f_{c1}}}{60000} \qquad \rho_{min1} = 3.354 \times 10^{-3}$$

$$\rho_{min2} := \frac{200}{60000} \qquad \rho_{min2} = 3.333 \times 10^{-3}$$

Take the lesser of ρ_{min1} & ρ_{min2}:

$$\rho_{min} := \begin{vmatrix} \rho_{min1} & \text{if } \rho_{min2} > \rho_{min1} \\ \rho_{min2} & \text{otherwise} \end{vmatrix}$$

$$\rho_{min} = 3.333 \times 10^{-3}$$

3.) Calculate "ρ" Actual for A_s:

$$A_s = 0.83 \, in^2$$

These are the calculated Rho for the given steel area provided in the range variable above:

$$\rho(A_s) := \frac{A_s}{b \times d}$$

$$\rho(A_s) = 0.0121123$$

$$kip \equiv 1000lbf$$

$$lbf \equiv lb$$

4.) Calculate "ρ_{bal}":

Calculate β_1:

$$\beta_1 := if\left[f_c \le 4000psi, 0.85, 0.85 - 0.05 \times \left(\frac{f_c - 4000psi}{1000psi} \right) \right]$$

$$\beta_1 = 0.825$$

$$\rho_{bal} := \beta_1 \times \left(\frac{0.85 \times f_{c1}}{f_{y1}} \right) \times \frac{87000}{87000 + f_{y1}}$$

$$\rho_{bal} = 0.031 \qquad \rho_{max} := 0.75 \times \rho_{bal}$$

$$\rho_{max} = 0.02335 \qquad \rho_{max} \text{ is 75\% of } \rho_{bal}$$

SUBJECT Florida 32" F-Shape
         Deck Strength

CLIENT   FDOT

5.) Is "ρ" Good Check:

$$\text{RhoCheck}(A_s) := \begin{vmatrix} "OK" & \text{if} & \rho_{min} < \rho(A_s) < \rho_{max} \\ "NG!" & \text{otherwise} \end{vmatrix}$$

$$\xi(A_s) := \begin{vmatrix} 1.0 & \text{if} & \rho_{min} < \rho(A_s) < \rho_{max} \\ 0 & \text{otherwise} \end{vmatrix}$$

$$A_s = 0.827 \, in^2$$

$$\text{RhoCheck}(A_s) = "OK"$$

$$\xi(A_s) = 1$$

$$\xi(A_s) := 1$$

Do a "Rho Check" here... discard Rho values below ρ_{min} and above ρ_{max}!

For Values outside of these limits, we will multiply Mn by ζ to make the moment capacity go to "0".

6.) Calculate "a", Nominal Strength (M_n) & Factored Strength (M_u):

$$a(A_s) := \frac{A_s \times f_y}{0.85 \times f'_c \times b}$$

$$a(A_s) = 1.081 \, in$$

$$M_n(A_s) := A_s \times f_y \times \left(d - \frac{a(A_s)}{2}\right) \times \xi(A_s)$$

$$M_n(A_s) = 21.275 \, kip \times ft$$

$$\phi = 0.9$$

$$M_u(A_s) := M_n(A_s) \times \phi$$

$$M_u(A_s) = 19.148 \, kip \times ft$$

46

Texas Transportation Institute

PAGE 1 of 3
JOB NO.: 421323
Date: 05-14-04

BY: W. Williams
CKD: _____

SUBJECT   Florida 32" F-Shape
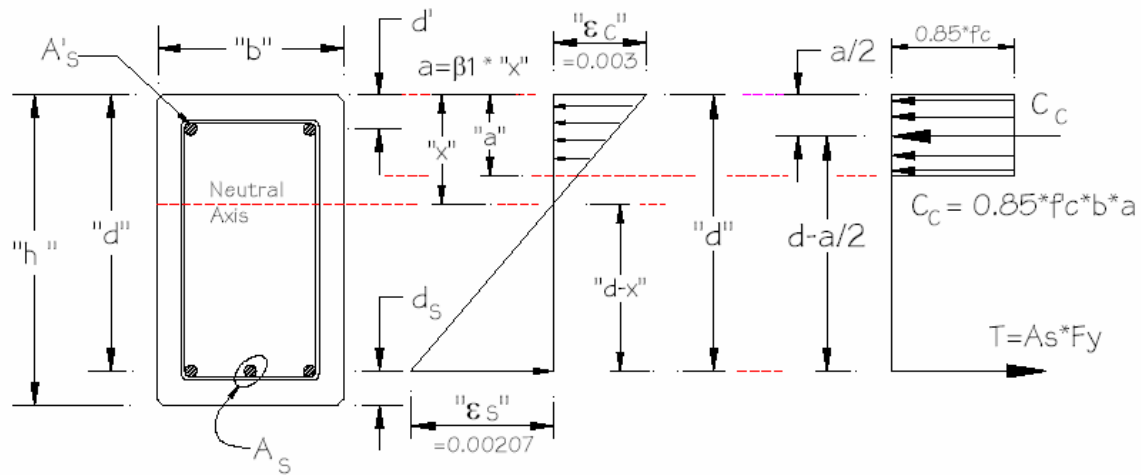
         Deck Strength ~ #5's @ 8.0 inches O.C.

CLIENT   FDOT

1.) Given Design Data:

1.) Worksheet calculates Ultimate and Factored Moments (M_n & M_u) and plots a graph of these values versus area of steel (A_s) for the given geometry and properties of the concrete beam.

2.) Input Items in Yellow including range variable for area of steel (A_s).

3.) Review acceptable ranges of Rho for max and minimum reinforcement allowed.

4.) AutoCad Details: U:*\Beam22.dwg.

$$f_c := 4500 \text{psi} \qquad f_y := 60000 \text{psi} \qquad d := 8\text{in} - 2\text{in} - 0.3125\text{in} \qquad b := 1 \times \text{ft} \qquad \phi := 0.9$$

$$d = 5.688 \text{ in}$$

$$A_s := \left(\frac{12}{8.0} \times 0.31 \times \text{in}^2 \right) \qquad \text{... #5's @ 8.0 inches O.C.}$$

$$f_{c1} := \frac{f_c}{\text{psi}} \qquad f_{y1} := \frac{f_y}{\text{psi}}$$

**Texas
Transportation
Institute**

SUBJECT   Florida 32" F-Shape

          Deck Strength

CLIENT   FDOT

2.) Calculate "ρ_{min}":

$$\rho_{min1} := \frac{3 \times \sqrt{f_{c1}}}{60000} \qquad \rho_{min1} = 3.354 \times 10^{-3}$$

$$\rho_{min2} := \frac{200}{60000} \qquad \rho_{min2} = 3.333 \times 10^{-3}$$

Take the lesser
of ρ_{min1} & ρ_{min2}:
$$\rho_{min} := \begin{vmatrix} \rho_{min1} & \text{if} \ \rho_{min2} > \rho_{min1} \\ \rho_{min2} & \text{otherwise} \end{vmatrix}$$

$$\rho_{min} = 3.333 \times 10^{-3}$$

3.) Calculate "ρ" Actual for A_s: $\qquad A_s = 0.47\,in^2$

These are the calculated Rho
for the given steel area provided in
the range variable above:

$$\rho(A_s) := \frac{A_s}{b \times d}$$

$$\rho(A_s) = 0.0068132$$

$$kip \equiv 1000lbf$$

4.) Calculate "ρ_{bal}":

$$lbf \equiv lb$$

Calculate β_1: $\quad \beta_1 := if\left[f_c \le 4000psi, 0.85, 0.85 - 0.05 \times \left(\frac{f_c - 4000psi}{1000psi}\right)\right]$

$$\beta_1 = 0.825$$

$$\rho_{bal} := \beta_1 \times \left(\frac{0.85 \times f_{c1}}{f_{y1}}\right) \times \frac{87000}{87000 + f_{y1}}$$

$$\rho_{bal} = 0.031 \qquad \rho_{max} := 0.75 \times \rho_{bal}$$

$$\rho_{max} = 0.02335 \qquad \rho_{max} \text{ is } 75\% \text{ of } \rho_{bal}$$

Texas
Transportation
Institute

SUBJECT   Florida 32" F-Shape

    Deck Strength

CLIENT   FDOT

BY: W. Williams
CKD: _____

5.) Is "ρ" Good Check:

$$\text{RhoCheck}(A_s) := \begin{vmatrix} \text{"OK"} & \text{if} & \rho_{min} < \rho(A_s) < \rho_{max} \\ \text{"NG!"} & \text{otherwise} \end{vmatrix}$$

$$\xi(A_s) := \begin{vmatrix} 1.0 & \text{if} & \rho_{min} < \rho(A_s) < \rho_{max} \\ 0 & \text{otherwise} \end{vmatrix}$$

$$A_s = 0.465 \, in^2$$

$$\text{RhoCheck}(A_s) = \text{"OK"}$$

$$\xi(A_s) = 1$$

$$\xi(A_s) := 1$$

Do a "Rho Check" here... discard Rho values below ρ_{min} and above ρ_{max}!

For Values outside of these limits, we will multiply Mn by ζ to make the moment capacity go to "0".

6.) Calculate "a", Nominal Strength (M_n) & Factored Strength (M_u):

$$a(A_s) := \frac{A_s \times f_y}{0.85 \times f_c \times b}$$

$$a(A_s) = 0.608 \, in$$

$$M_n(A_s) := A_s \times f_y \times \left(d - \frac{a(A_s)}{2}\right) \times \xi(A_s)$$

$$M_n(A_s) = 12.517 \, kip \times ft$$

$$\phi = 0.9$$

$$M_u(A_s) := M_n(A_s) \times \phi$$

$$M_u(A_s) = 11.265 \, kip \times ft$$

49