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16. Abstract

In 2003, there were 38 metal truss bridges 50 years of age or older remaining on the State of Texas highway system. Of these 38 bridges, 33 are listed in the National Register of Historic Places. Many of these bridges do not meet current design criteria for rehabilitation due to narrow deck widths, low vertical clearance, and substandard load capacity. In addition, the existing bridge railing systems on these bridges have not been shown to meet the current requirements for safety and strength.

This project addressed the design and performance of acceptable traffic railings for existing and new truss bridges in Texas. Specific objectives were to:

- design/develop a retrofit railing for low-speed application on the Roy B. Inks Bridge in Llano, Texas;
- design/develop a retrofit railing for high-speed application on the U.S. 281 Bridge over the Brazos River in Palo Pinto County, Texas;
- identify criteria that can serve as a basis for design exceptions; and

design/develop a traffic railing for new truss bridges.

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RETROFIT RAILINGS FOR TRUSS BRIDGES

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Project 0-4419
Project Title: Retrofit Railing for Existing Truss Bridges

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DISCLAIMER

The contents of this report reflect the views of the authors, who are responsible for the facts and the accuracy of the data, 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), 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 C. Eugene Buth, P.E. (Texas, #27579).

ACKNOWLEDGMENTS

This research project was conducted under a cooperative program between the Texas Transportation Institute, the Texas Department of Transportation, and the U.S. Department of Transportation, Federal Highway Administration. The TxDOT program coordinator for this research was David P. Hohmann, P.E.; the project director was Charles E. Walker, P.E.; and the project monitor was Mark J. Bloschock, P.E. The valuable guidance and assistance of Mr. Hohmann, Mr. Walker, and Mr. Bloschock throughout the course of this project are acknowledged and appreciated.

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CHAPTER 1. INTRODUCTION

BACKGROUND

The Texas Department of Transportation (TxDOT) Environmental Affairs Division has developed a bridge project coordination process to ensure that bridge replacement projects comply with preservation laws and regulations and to facilitate project coordination with the State Historic Preservation Officer (SHPO). From this process, the TxDOT Historic Bridge Task Force was formed in 1996 for the purpose of developing a methodology to evaluate preservation options for on-system truss bridges that are listed or are eligible for listing in the National Register of Historic Places. Section 106 of the National Historic Preservation Act of 1966, as amended, requires TxDOT, acting as an agent for the Federal Highway Administration (FHWA), to coordinate all federally funded, licensed, or permitted bridge projects involving bridges 50 years of age or older with the staff of the SHPO. In Texas, the SHPO is the Executive Director of the Texas Historic Commission.

In 2003, there were 38 metal truss bridges 50 years of age or older remaining on the State of Texas highway system. Of these 38 bridges, 33 are listed in the National Register of Historic Places. Many of these bridges do not meet current design criteria for rehabilitation because they have narrow deck widths, low vertical clearance, and substandard load capacity. In addition, the existing bridge railing systems on these bridges have not been shown to meet the current requirements for safety and strength.

OBJECTIVES/SCOPE OF RESEARCH

This project addressed the design and performance of acceptable traffic railings for existing and new truss bridges in Texas. Specific objectives were to

- design/develop a retrofit railing for low-speed application on the Roy B. Inks Bridge in Llano, Texas;
- design/develop a retrofit railing for high-speed application on the U.S. 281 Bridge over the Brazos River in Palo Pinto County, Texas;
- identify criteria that can serve as a basis for design exceptions; and
- design/develop a traffic railing for new truss bridges.

The Roy B. Inks Bridge carries State Highway 16 over the Llano River in Llano, Texas. This bridge was constructed in the early 1930s, is classified as a historic structure, and is listed in the National Register of Historic Places. Its four main spans are Parker thru-truss structures, and the concrete roadway is 24 ft 0 inch (7.32 m) wide face-to-face of curbs. The existing bridge consists of four spans each measuring 198 ft 6-3/4 inch (60.5 m), for a total length of 794 ft 3 inches (242.0 m) between abutments. Each bridge span consists of nine panels, 22 ft 3/4 inches (6.7 m) in length. The curb is 1 ft 0 inch (305 mm) tall, and a C12×20.7 (C310×31) traffic rail is mounted directly to the truss members at a height to the top of rail of 3 ft 1 inch (940 mm) above the roadway. The face of the channel is set back approximately

6 inches (152 mm) behind the top face of the curb. The existing configuration does not provide a high level of protection to the truss members from errant vehicular impacts. The bridge is to be rehabilitated by TxDOT, and the crashworthiness of the existing traffic railing is considered inadequate by current standards. The posted speed limit on the bridge is 40 mi/h (64 km/h). National Cooperative Highway Research Program *(NCHRP) Report 350* Test Level 2 (TL-2) is appropriate for this posted speed limit *(1)*.The bridge needed a TL-2 retrofit railing that would be compatible with the appearance of the existing bridge and require minimum structural modifications to the existing bridge superstructure.

The U.S. 281 Bridge over the Brazos River in Palo Pinto County, Texas, is a three-span, steel Warren-type truss bridge with verticals. This bridge was constructed in the early 1930s, is classified as a historic structure, and is listed in the National Register of Historic Places. Two of the three spans measure 202 ft (61.6 m), and the longer middle span measures 252 ft 6 inches (77 m). The total length of the truss bridge is 656 ft 6 inches (200 m). The 202-ft spans have eight 25 ft 3 inch (7.7 m) panels, and the middle span, which is 252 ft 6 inches (77.0 m) in length, has ten 25 ft 3 inch (7.7 m) panels. The clear roadway width is 24 ft 0 inch (7.3 m) between the top faces of the curbs. The total length of the bridge including the approach spans is 1138 ft 4 inches (347 m). The existing curb is 1 ft 0 in (305 mm) tall and 1 ft 7-1/2 inches (495 mm) in width. A C12×20.7 (C310×31) traffic rail is mounted directly to the truss members; height to the top of rail from the roadway is 3 ft 1 inch (940 mm). The face of the channel is set back approximately 1 ft 6 inches (457 mm) behind the top face of the curb. The existing configuration does not provide a high level of protection to the truss members from errant vehicular impacts. The bridge is to be rehabilitated by TxDOT, and the crashworthiness of the existing traffic railing is considered inadequate by current standards. The posted speed limit on the bridge is 60 mi/h (97 km/h). *NCHRP Report 350* Test Level 3 (TL-3) is appropriate for this posted speed limit. The bridge needed a TL-3 retrofit railing that would be compatible with the appearance of the existing bridge and require minimum structural modifications.

Many existing historic through-truss bridges are located on highways with posted speed limits greater than 45 mi/h (72 km/h), and a TL-3 bridge railing would be indicated. Some of these bridges are narrow, and the impact speed and angle combination for TL-3 might not be appropriate. For these bridges, direct application of design loads for a TL-3 condition may lead to extensive and unnecessary alteration of original truss members. The objective of this portion of the project was to investigate methods for evaluating the response of existing truss bridge members to impact forces resulting from mounting a TL-3 retrofit rail directly to an existing historic truss.

TxDOT plans several new truss bridges throughout the state. The typical new truss is assumed to be a Warren-type or Pratt-type pony truss with vertical truss web members at each panel point. Currently, the bridge railing proposed for these structures is a standard TxDOT railing, the T101, which is supported by a cast-in-place concrete deck. TxDOT would prefer to have the option to support a bridge rail system from the truss members in lieu of supporting the railing from the concrete deck. The primary advantage of using a truss-supported bridge rail is to allow alternate types of deck. One disadvantage to using a truss-supported bridge rail is that the bridge structure must be adequately designed to resist the crash loads imparted from the bridge rail directly to the truss members. A truss-mounted bridge railing system provides the bridge designer with more options and greater flexibility in designing steel truss bridges. The objective of this phase of the project was to design a truss-mounted bridge rail system for new Pratt-type or Warren-type trusses that have vertical truss members rigidly connected to transverse floorbeams. The railing was to be designed for installation by bolted connection to vertical members spaced 20 ft (6.1 m) or less apart.

CHAPTER 2. TESTING AND EVALUATION

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 a U.S. 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 construction of the Llano Truss Bridge is along a wide out-of-service apron. The apron consists of an unreinforced jointed concrete pavement in 12.5 ft by 15 ft (3.8 m by 4.6 m) blocks nominally 8 to 12 inches (203 to 305 mm) deep. The aprons and runways are about 50 years old, and the joints have some displacement, but are otherwise flat and level.

CRASH TEST CONDITIONS

Crash testing procedures for evaluating the performance of bridge rails and other highway safety structures are based on the assumption that the errant vehicle is tracking straight ahead with no side-slip and no yaw velocity. Recommended test conditions include the vehicle type/mass, speed, and approach angle. Lateral placement of the vehicle with respect to the device being tested is also included for guardrail terminals, sign supports, and other similar devices.

Evaluation of longitudinal barriers, such as the Llano Truss Bridge Rail, to TL-2 of *NCHRP Report 350* requires two tests:

> ***NCHRP Report 350* test designation 2-10**: An 1806-lb (820 kg) passenger car impacts the bridge rail at the critical impact point (CIP) along the length of need (LON) at a nominal speed and angle of 43.5 mi/h (70 km/h) and 20 degrees, respectively, to evaluate occupant risk and post-impact trajectory.
>
> ***NCHRP Report 350* test designation 2-11**: A 4404-lb (2000 kg) pickup truck impacts the bridge rail at the CIP along the LON at a nominal speed and angle of 43.5 mi/h (70 km/h) and 25 degrees, respectively, to evaluate strength of the section in containing and redirecting the 4404-lb (2000 kg) vehicle.

The test reported herein on the Llano Truss Bridge Rail corresponds to *NCHRP Report 350* test designation 2-11. Researchers performed this test to evaluate the ability of the bridge rail to safely contain and redirect the pickup truck as it impacted the bridge rail at a speed of 43.5 mi/h (70 km/h). Information and tables contained in the guidelines of *NCHRP Report 350* were used to select the CIP for this test. The target impact point for this test was 2.6 ft (0.8 m) upstream of post 5.

TL-3 of *NCHRP Report 350* also requires two redirection tests for a bridge rail. They are:

> ***NCHRP Report 350* test designation 3-10:** An 1806-lb (820 kg) passenger car impacts the CIP in the LON of the longitudinal barrier at a nominal speed and angle of 62 mi/h (100 km/h) and 20 degrees to evaluate the overall performance of the LON section in general and occupant risks in particular.
>
> ***NCHRP Report 350* test designation 3-11:** A 4405-lb (2000 kg) pickup truck impacts the CIP in the LON of the longitudinal barrier at a nominal speed and angle of 62 mi/h (100 km/h) and 25 degrees to evaluate the strength of the section for containing and redirecting the pickup truck.

The test reported herein on the U.S. 281 Truss Bridge Rail corresponds to *NCHRP Report 350* test designation 3-11. This test evaluates the strength of the section to safely contain and redirect the pickup truck as it impacts the bridge rail at a speed of 62 mi/h (100 km/h). Information and tables contained in the guidelines of *NCHRP Report 350* were used to select the CIP for this test. The target impact point for this test was 1.3 m (4.26 ft) upstream of the splice located mid-span between posts 4 and 5.

The crash test and data analysis procedures were in accordance with guidelines presented in *NCHRP Report 350*. Appendix A presents brief descriptions of these procedures.

EVALUATION CRITERIA

Researchers evaluated the crash tests in accordance with *NCHRP Report 350*. As stated in *NCHRP Report 350*, “Safety performance of a highway appurtenance cannot be measured directly but can be judged on the basis of three factors: structural adequacy, occupant risk, and vehicle trajectory after collision.” Accordingly, researchers used the safety evaluation criteria from Table 5.1 of *NCHRP Report 350* to evaluate the crash tests reported herein.

CHAPTER 3. LLANO TRUSS BRIDGE RAIL

ANALYSIS AND DESIGN

TxDOT engineers worked closely with researchers at the Texas Transportation Institute (TTI) to develop a retrofit bridge railing for the Roy B. Inks Bridge over the Llano River in Llano, Texas. Their goal was to develop a crashworthy design for low-speed application that preserves the historical character of the bridge. To meet this objective, a truss-mounted rail system was desired. Structural evaluation included: design of the new rail, design of energy-absorbing mountings, and evaluation of the existing structures response to rail impact loading.

Initially, TxDOT personnel and the TTI researchers decided that safety performance could be improved by lowering the bridge rail from a height of 37 inches (940 mm) above the existing pavement surface to a height of 32 inches (813 mm) to provide better geometric interaction with passenger vehicles. A TS8×4×1/2 (TS203×103×13) tube was used to increase the flexural capacity of the rail. This tube helps distribute the collision load to more intermediate support posts and truss members. To preserve the visual appearance of the original rail, the tube was placed behind the existing C12×20.7 (C310×31) rail member. Researchers noted during on-site inspection of the bridge that several truss members had been damaged due to vehicular impact. Researchers decided to offset the rail to reduce the potential for direct impact of the truss members. Upon consultation with TxDOT, the rail face was blocked out to the top edge of the existing concrete curb, permitting the existing clear roadway of 24 ft (7.3 m) to be maintained, thus eliminating the need for a design exception.

TTI researchers and the TxDOT project team worked closely to develop a conceptual design for analysis. After a conceptual design was developed, calculations were performed on the new retrofit design to determine if it had sufficient structural capacity to meet the requirements for TL-2 impact conditions as stated in the current American Association of State Highway Transportation Officials (AASHTO) *Load Resistance Factor Design* (LRFD) *Bridge Design Specifications* *(2)*. The capacity of the new retrofit design developed by TTI researchers did meet the minimum *NCHRP Report 350* TL-2 strength requirements. Next, researchers investigated the ability of the existing structure to resist impact forces resulting from the retrofit of the bridge rail.

Impacts at existing intermediate posts transmit a torsion force to the supporting W18×50 (W460×74) exterior stringer. An analysis was performed to determine if the torsion force on the exterior stringers would cause failure of the stringer. The intermediate posts are supported by two 3/4-inch (19 mm) diameter anchor rods embedded into the concrete curb as well as by segments of C12×20.7 (C310×31) attached to the exterior stringers. Based on the plastic strength of the post, the torsion moment applied to the stringer could be as high as 11 klb-ft (kip-ft) (14.93 kN-m). The lateral force applied to the stringer by the C12×20.7 (C310×31) is approximately 14.3 kips (63.6 kN). This load is applied to the W18×50 (W460×74) stringer from the collision load applied to the rail.

A finite element analysis was performed for a W18×50 (W460×74) stringer, 22 ft (6.7 m) in length. The top edges of the flange were fixed to simulate the embedment of the top flange into the deck concrete. Based on this analysis, some localized stresses exceeded 36 kips/in^2 (ksi) (248 MPa) near the applied load from the C12×20.7 (C310×31). These stresses might cause localized yielding of the stringer but should not cause a catastrophic failure of the stringer. Loads and torsion stresses in the exterior stringers were within acceptable limits from the TL-2 collision loads applied to the posts. No additional modifications were required for the exterior stringers.

The test installation constructed for this project included a simulated portion of the actual bridge superstructure supporting the concrete deck and bridge rail. Since the bridge rail is supported by the exterior W18×50 (W460×74) stringers, the effect of the collision load into the W18×50 (W460×74) stringer was included in the testing. No structural distress was observed in the simulated exterior stringers after the crash test.

Researchers performed an analysis of the pullout capacity of the 3/4-inch (19 mm) diameter rods that are anchored in the curb concrete and used to support the intermediate posts. This analysis included the magnitude of the tension force in the 3/4-inch (19 mm) anchor rods from the ultimate strength of the intermediate post used in the strength analysis of the retrofit design. In summary, for an AASHTO *LRFD* TL-2 crash load applied directly at a post, the maximum applied force to the rods is approximately 28 kips (125 kN) based on the plastic strength of the post. Based on the tensile strength of the concrete and the assumed mode of failure, the calculated force to fail the concrete around the rods is approximately 57 kips (254 kN). Based on this analysis, some localized yielding would likely occur during a TL-2 collision along with some spalling of concrete. Bearing force between the rods and the concrete would be very high in the area where the upper end of the rod projects into the concrete curb. However, a global failure of concrete supporting the anchors is not likely and was not observed in the crash test.

The retrofit rail is supported by the truss members and intermediate posts located between the truss members. The new retrofit rail incorporated W8×18 (W200×27) steel blockouts at the intermediate posts. Blockouts (of some type) are required at the truss members. To limit the magnitude of the impact force transmitted from the rail to the truss members, crushable steel pipe blockouts were used for the blockouts at the truss members. Analyses were performed to determine the crush strength of 12-inch (305 mm) long lengths of 5-inch (127 mm) and 6 inch (152 mm) schedule 40 steel pipe loaded transverse to the longitudinal axis of the pipe. These sizes closely matched the blockout distances required for the various sizes and shapes of truss members. Based on the analyses for a 5-inch (127 mm) and 6-inch (152 mm) schedule 40 pipe, the crush strength for 12-inch (305 mm) long pieces of each pipe size was approximately 8 kips (36 kN) for each pipe size. Thus, provided the pipe blockout does not completely collapse, the force transmitted from the rail to the truss members was limited to approximately 8 kips (36 kN). Researchers incorporated these pipe blockouts into the retrofit rail.

In summary, TTI researchers recommended that the prototype bridge rail be 32 inches (813 mm) in height. They also recommended using TS8×4×1/2 (TS203×102×13) tube behind the C12×20.7 (C310×31) rail, W8×18 (W200×27) blockouts at the intermediate posts, and 5-inch (127 mm) and 6-inch (152 mm) schedule 40 pipe blockouts at the truss members. This prototype bridge rail was constructed and subjected to full-scale crash testing. The retrofit bridge rail strength calculations are presented in Appendix B, and strength analyses of bridge truss members and additional information on the crush strength for the pipe blockouts is provided in Appendix C.

TEST INSTALLATION PROTOTYPE

TTI received detailed drawings from TxDOT entitled “198 [ft] 6-3/4 [inches] Steel Truss Span, Llano River Bridge Hwy. 29 Llano County,” dated October 1935. Details from these drawings were used to prepare construction and fabrication drawings for the project test installation, shown in Figures 1 through 23. The existing State Highway 16 bridge over the Llano River consists of four spans each measuring 198 ft 6-3/4 inches (60.5 m), for a total length of 794 ft 3 inches (242.0 m) between abutments. Each bridge span consists of nine panels, 22 ft 3/4 inches (6.7 m) in length. For this project TTI constructed a full-scale test installation consisting of three panels, each measuring approximately 22 ft 3/4 inches (6.7 m). The total length of the installation was approximately 70 ft (21.3 m). The post spacing in the installation closely matched a segment of the actual bridge structure. The bridge superstructure supporting the concrete deck and curb consisted of two rows of W18×50 (W460×74) stringers spaced 4 ft 6 inches (1.4 m) apart (see Figures 13 through 16). These stringers were supported by W33×130 (W840×193) support beams spaced 22 ft 3/4 inches (6.7 m) on center (see Figure 16). The stringers attached to the support beams with two L6×4×3/8 (L152×102×9) clip angles, 1 ft 3 inches (0.38 m) in length, and connected with ten 3/4-inch (19 mm) diameter A325 bolts, 2-1/2 inches (64 mm) in length (see Figure 12). The stringers were constructed at the same top-of-steel elevation as the top of the W33×130 (W840×193) support beams. The W33×130 (W840×193) support beams were supported by an 8-inch (203 mm) thick concrete slab constructed adjacent to the concrete apron at our testing facility (see Figures 2 and 3). All remaining features, with the exception of the truss members, were constructed similarly to the actual details used in the bridge.

The bridge rail posts were constructed from two L5×3-1/2×3/8 (L127×89×9) angles (long legs back-to-back) with a 5/16-inch (8 mm) thick steel plate (sandwich plate) located between the angles (see Figures 17 and 18). Several 3/4-inch (19 mm) diameter A325 bolts connected the post angles and sandwich plate together. The posts were supported by segments of C12×20.7 (C310×31) channel (see Figure 18), which connected to the exterior W18×50 (W460×74) stringers with a single L6×4×3/8 (L152×102×9) clip angle, bolted to the W18×50 (W460×74) stringer with four 3/4-inch (19 mm) diameter A325 bolts (see Figure 12). These posts were also supported by two 3/4-inch (19 mm) diameter A36 threaded rods, embedded into the concrete curb and deck and also connected to the top of the W18×50 (W460×74) stringers (see Figure 10). Cross sections of the posts are shown in Figures 4 through 8.

The concrete curb and deck were cast on top of the support beams and stringers with the tops of the stringers and support beams extending into the deck concrete approximately 1 inch (25 mm). The concrete curb was 12 inches (305 mm) high and 6 inches (152.4 mm) wide at the top and sloped on the traffic side face to a thickness of 8 inches (203 mm) at the gutter line. The concrete deck was 7-1/2 inches (191 mm) thick and extended beyond the centerline of the exterior stringer 1 ft 3 inches (0.38 m) (see Figure 9). Transverse reinforcement in the deck consisted of #5 (#16) hooked bars “A” on 1 ft 3 inch (0.38 m) centers and #5 (#16) hooked bars “B” on 1 ft 3 inch (0.38 m) centers (see Figure 11). In the top layer of reinforcement in the deck, the effective transverse bar spacing was approximately 7-1/2 inches (191 mm). At the time the bridge was constructed, the yield strength of concrete reinforcing steel was typically

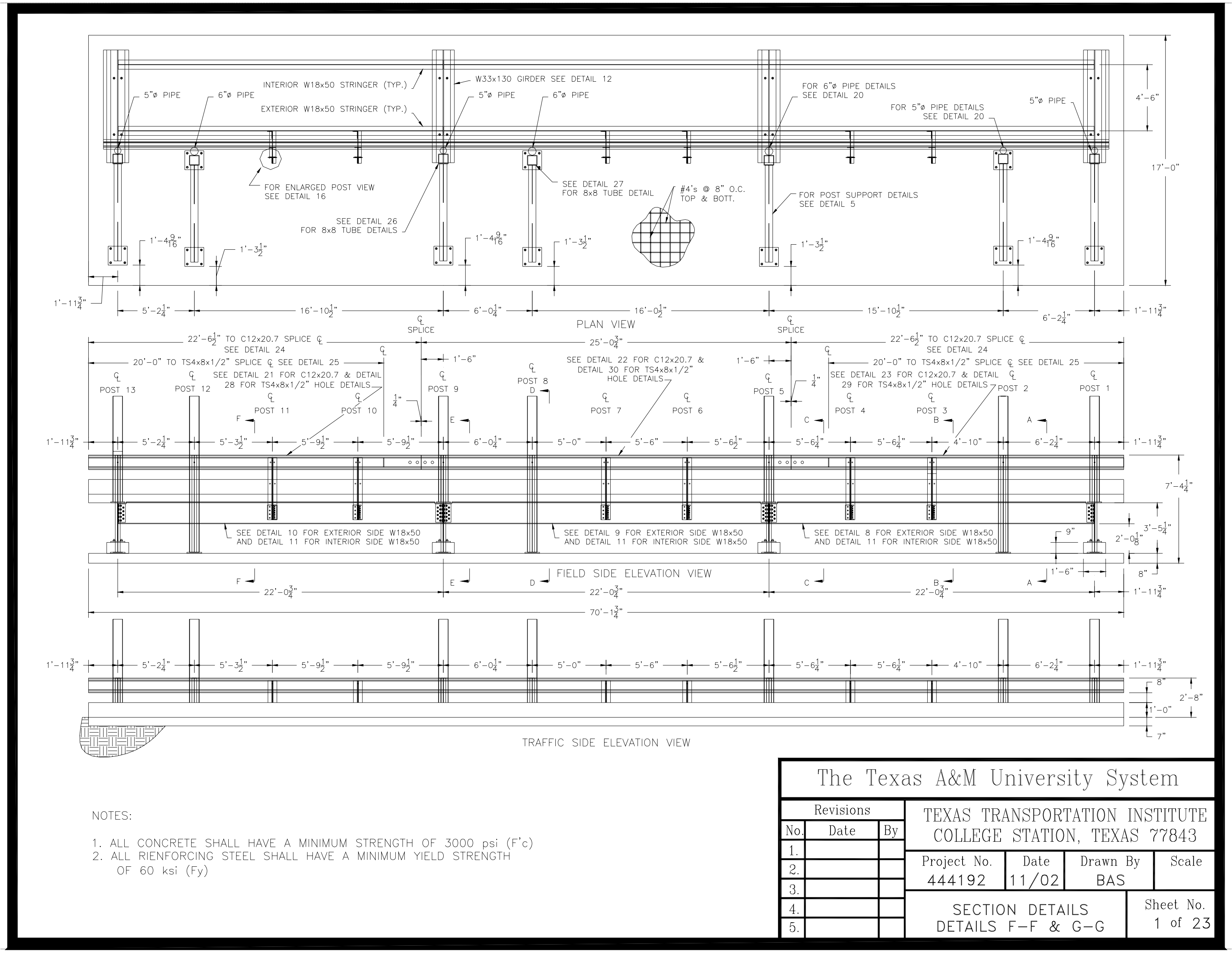


Figure 1. Overall Layout of the Llano Truss Bridge Installation.

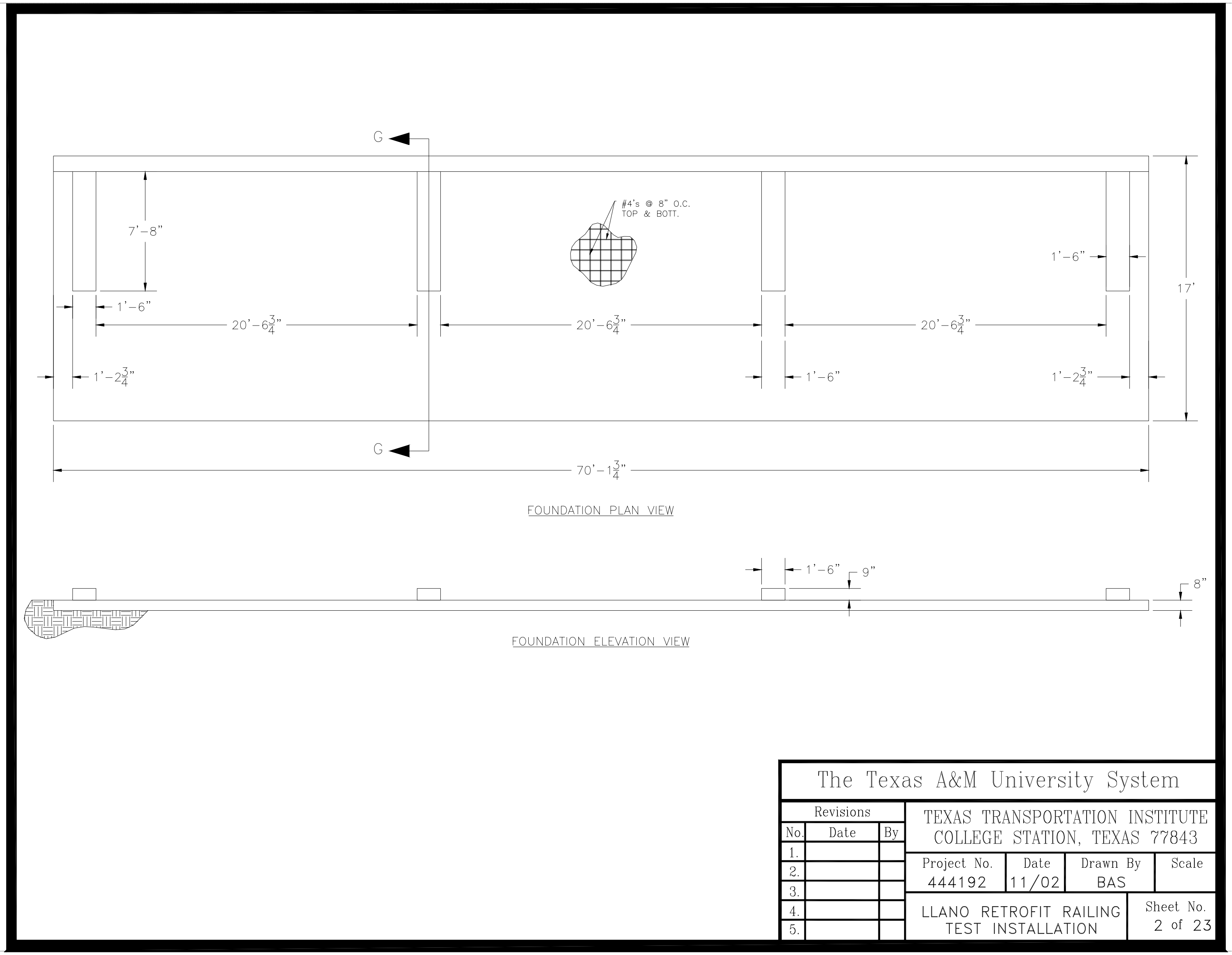


Figure 2. Layout of the Foundation for Llano Truss Bridge Installation.

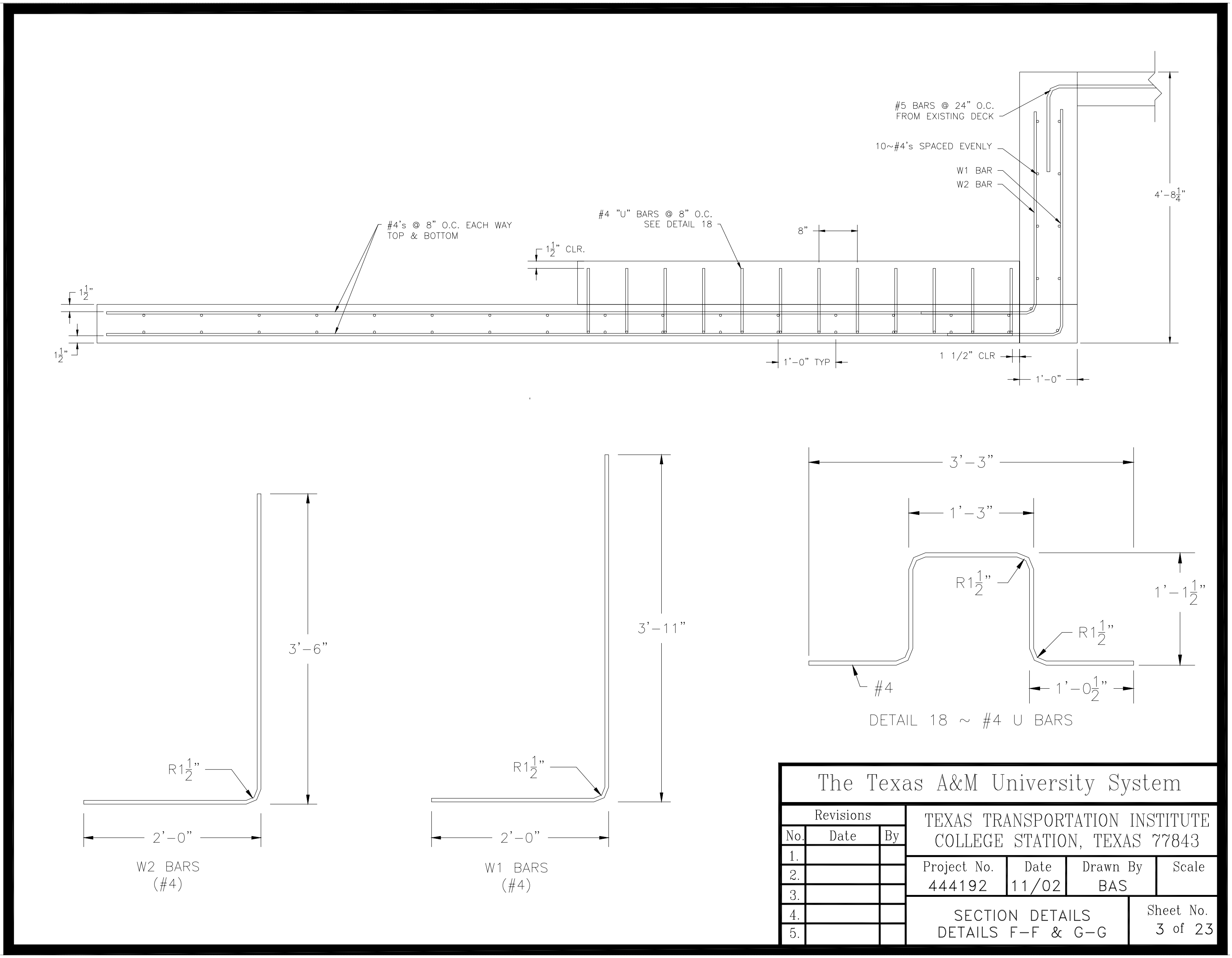


Figure 3. Rebar Details of the Foundation for Llano Truss Bridge Installation.

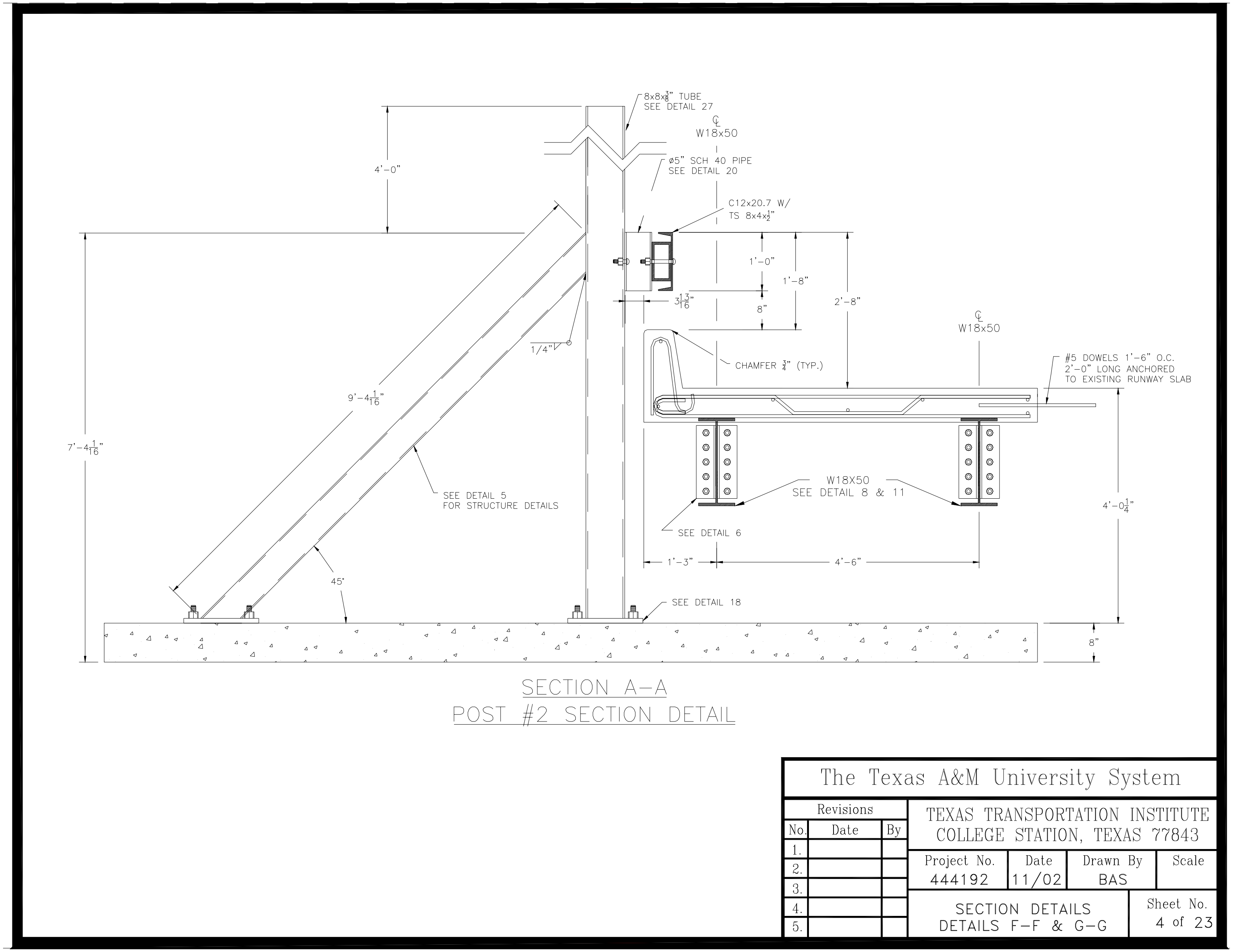


Figure 4. Cross Section of Post 2.

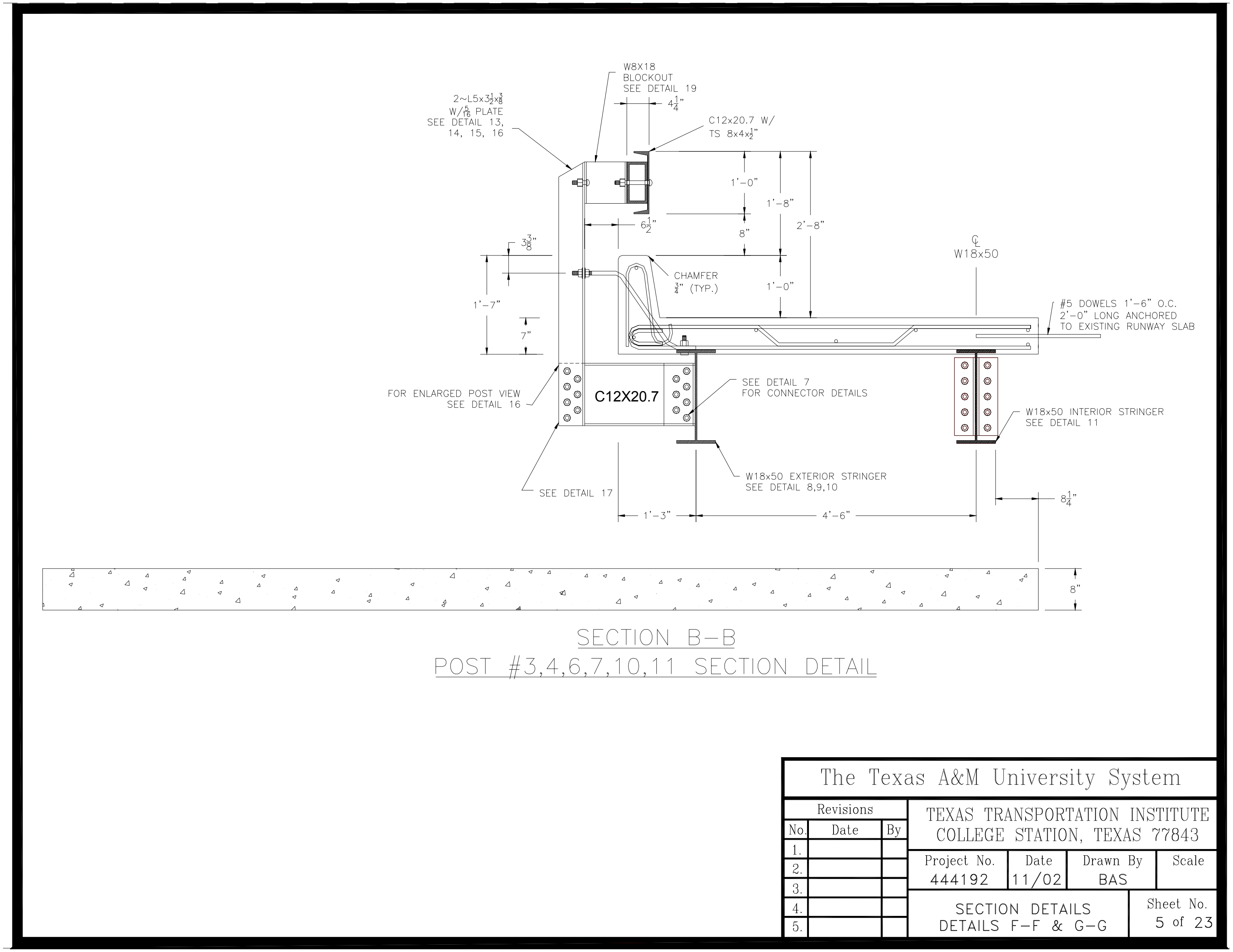


Figure 5. Cross Section of Posts 3, 4, 6, 7, 10, and 11.

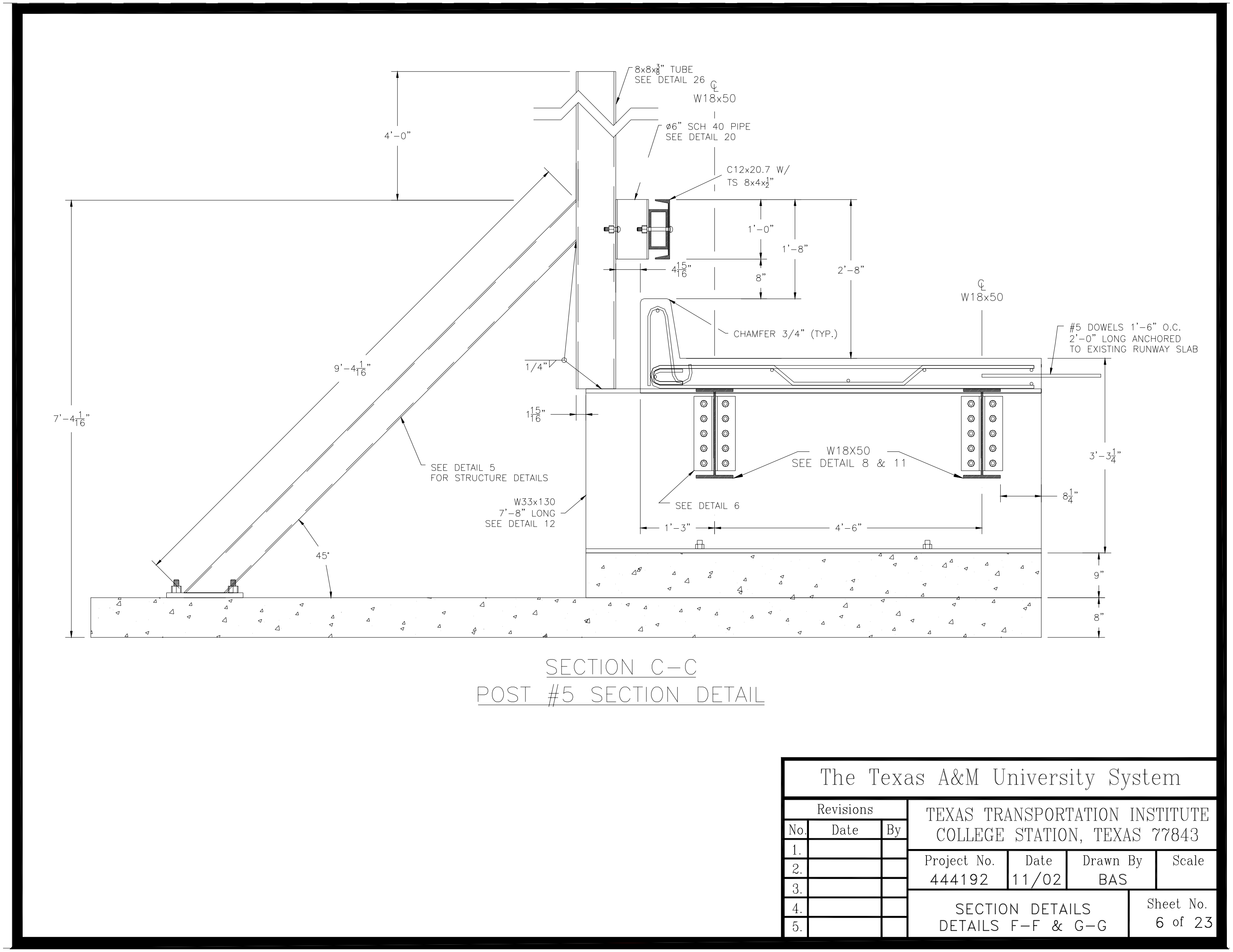


Figure 6. Cross Section of Post 5.

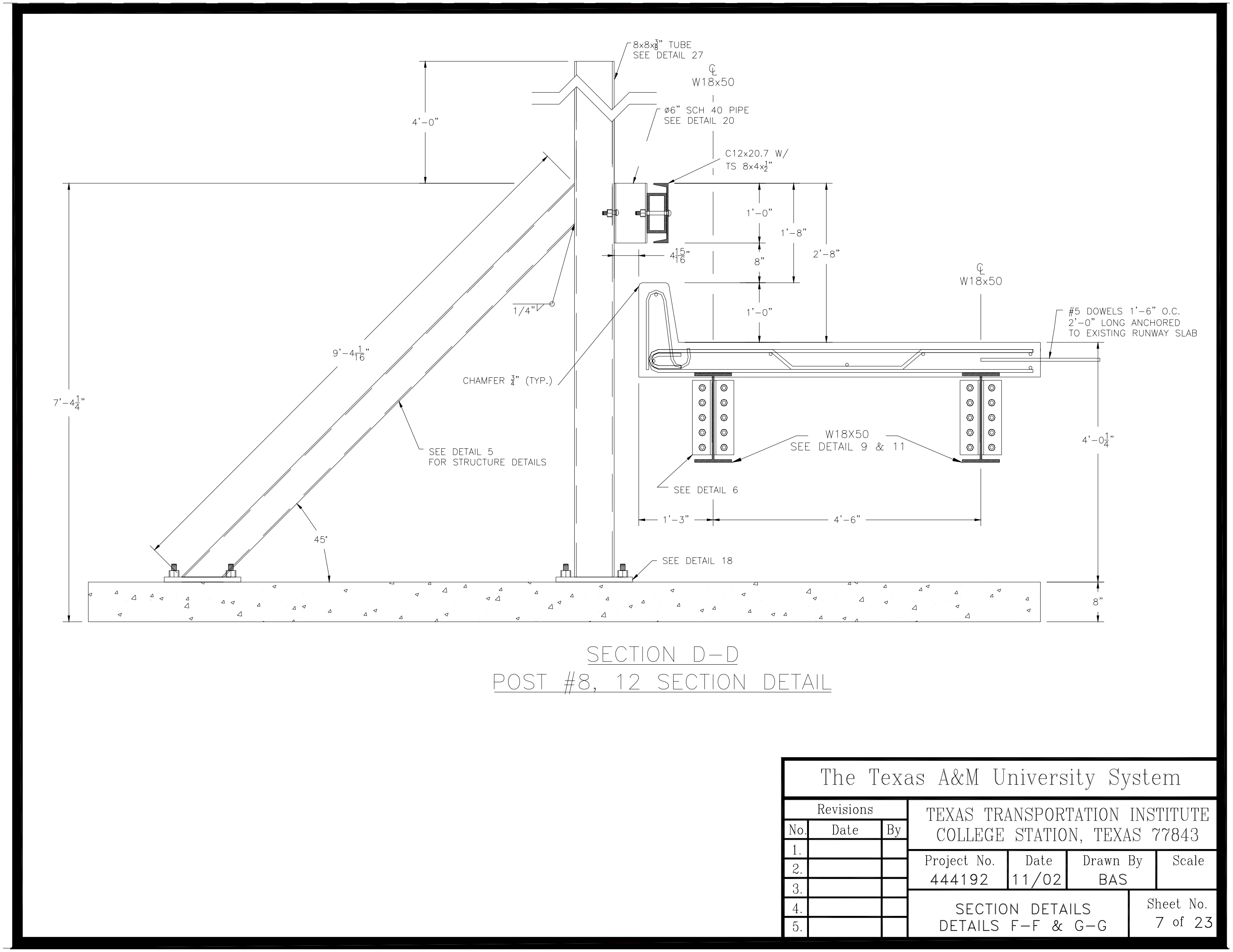
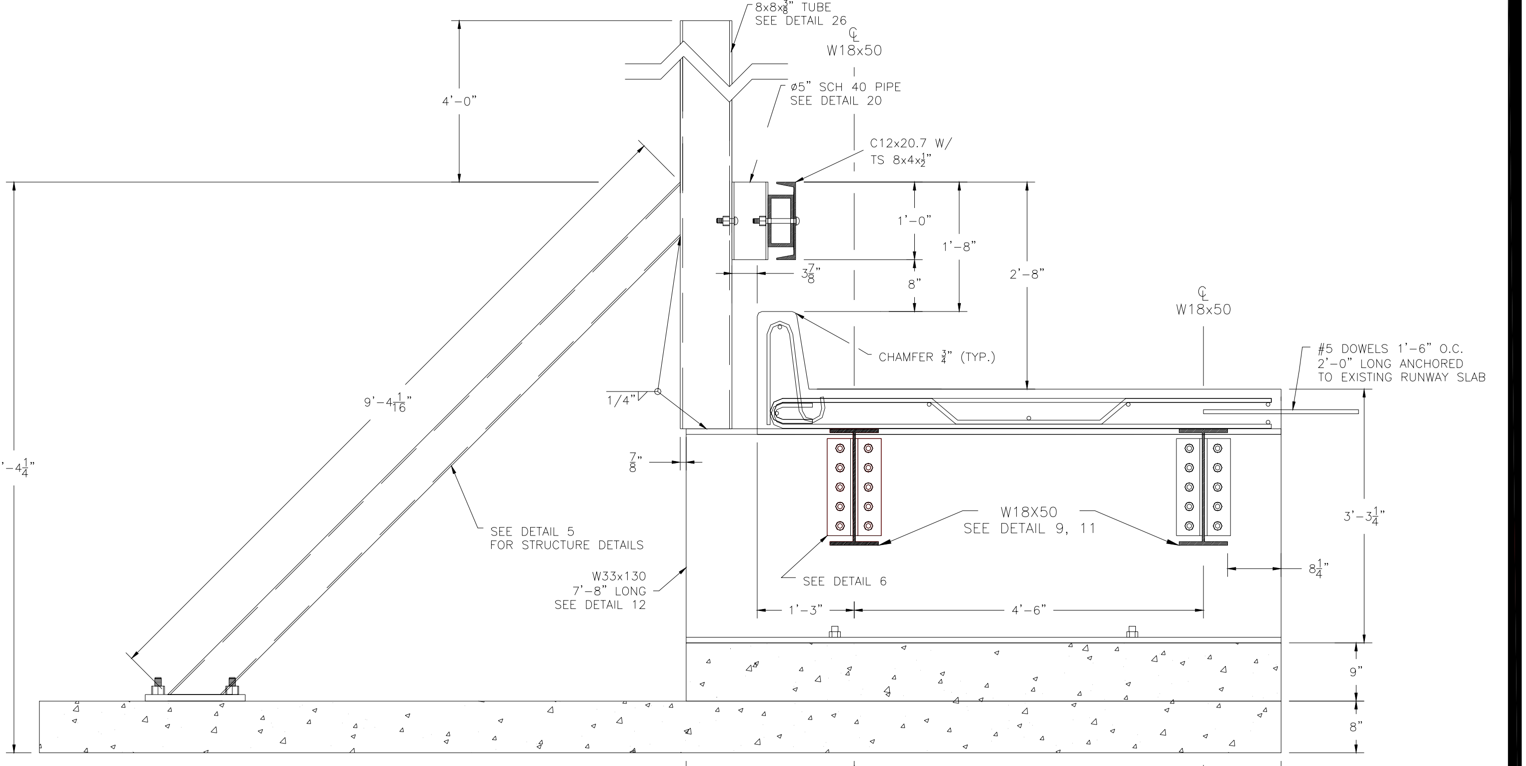


Figure 7. Cross Section of Posts 8 and 12.



SECTION E-E

POST #1, 9, 13 SECTION DETAIL

The Texas A&M University System

TEXAS TRANSPORTATION INSTITUTE
COLLEGE STATION, TEXAS 77843

Revisions		
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Figure 8. Cross Section of Posts 1, 9, and 13.

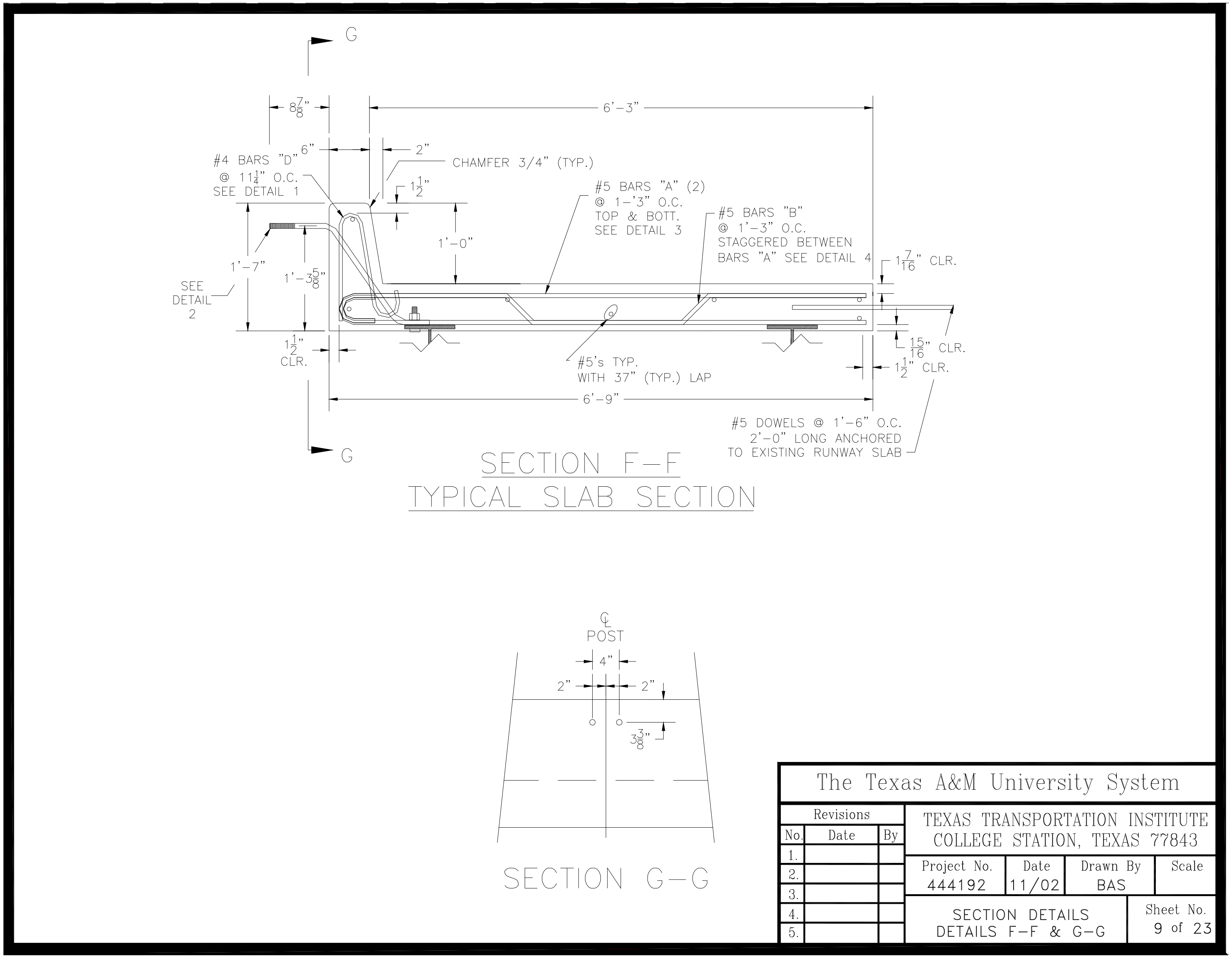


Figure 9. Details of Concrete Curb and Deck.

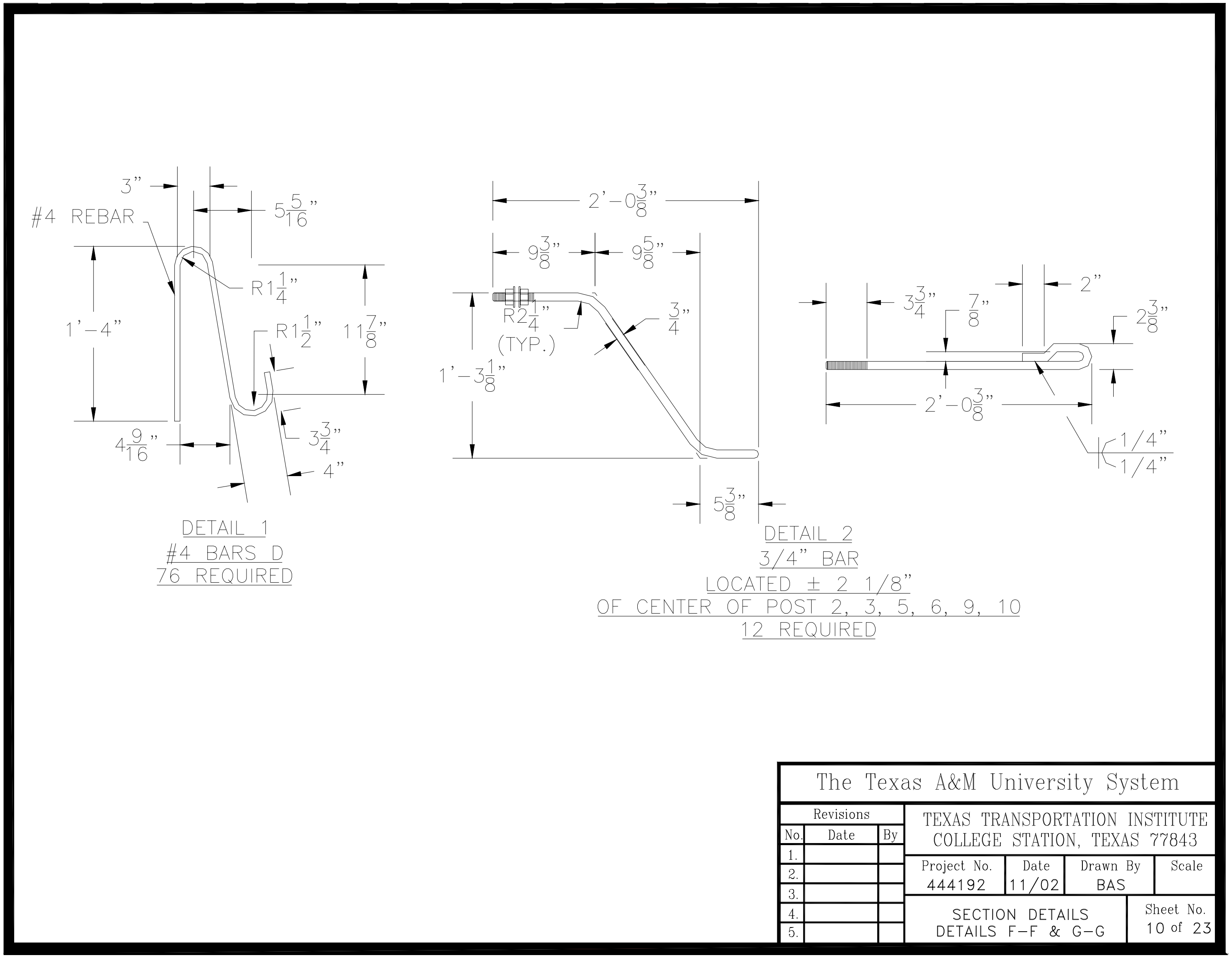


Figure 10. Rebar Details for Concrete Curb and Bridge Rail Posts.

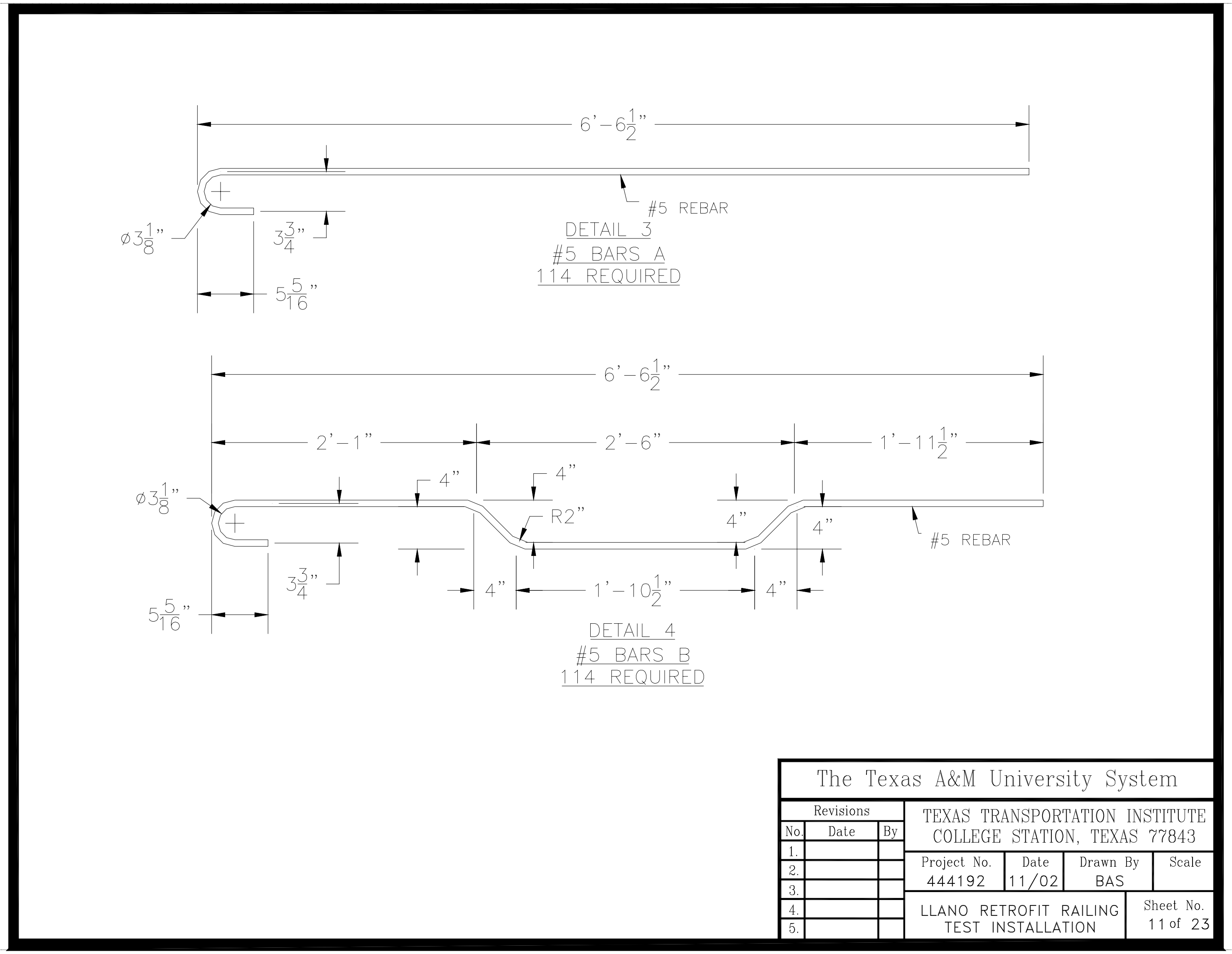


Figure 11. Rebar Details for Concrete Curb and Deck.

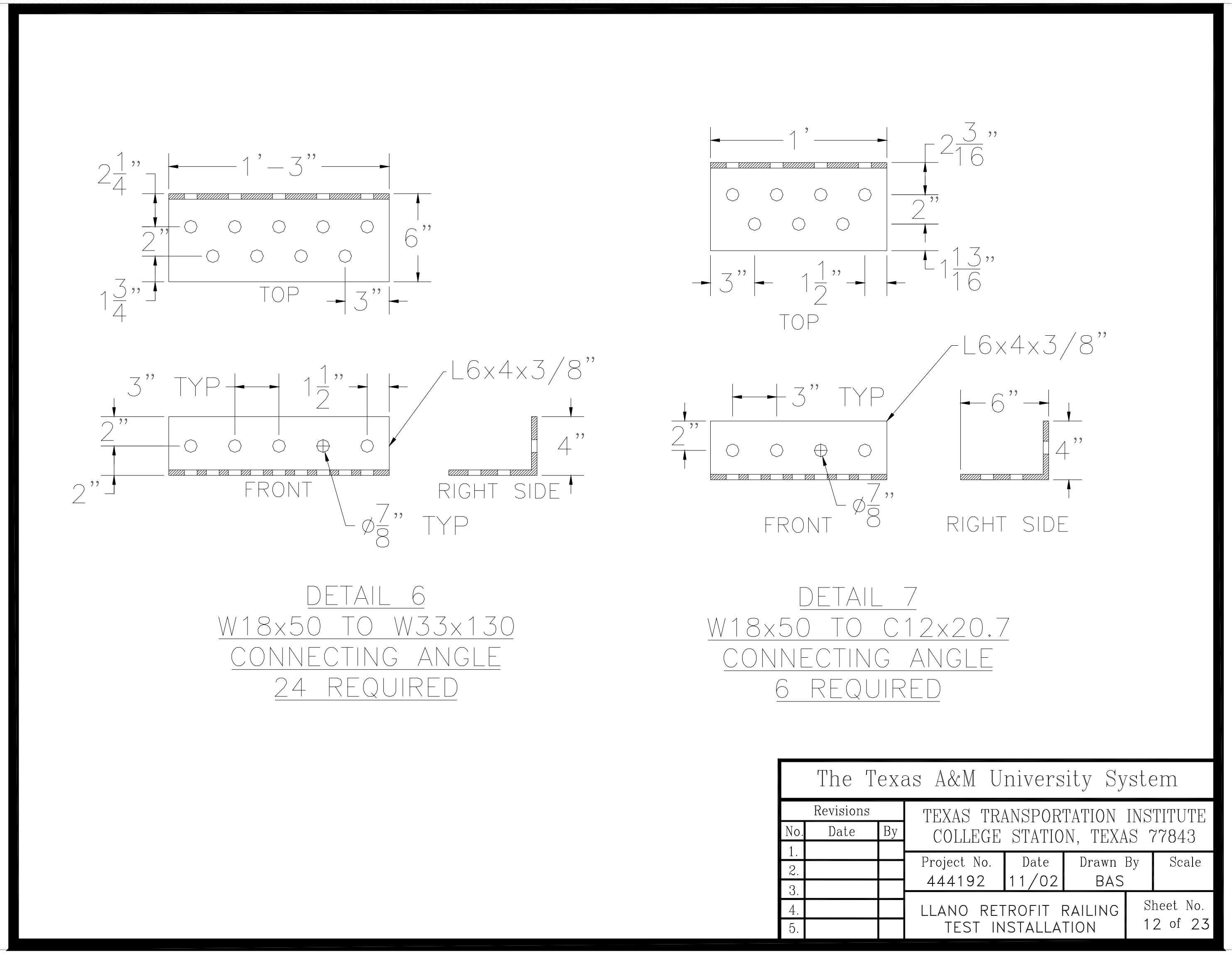


Figure 12. Details of Connecting Angles.

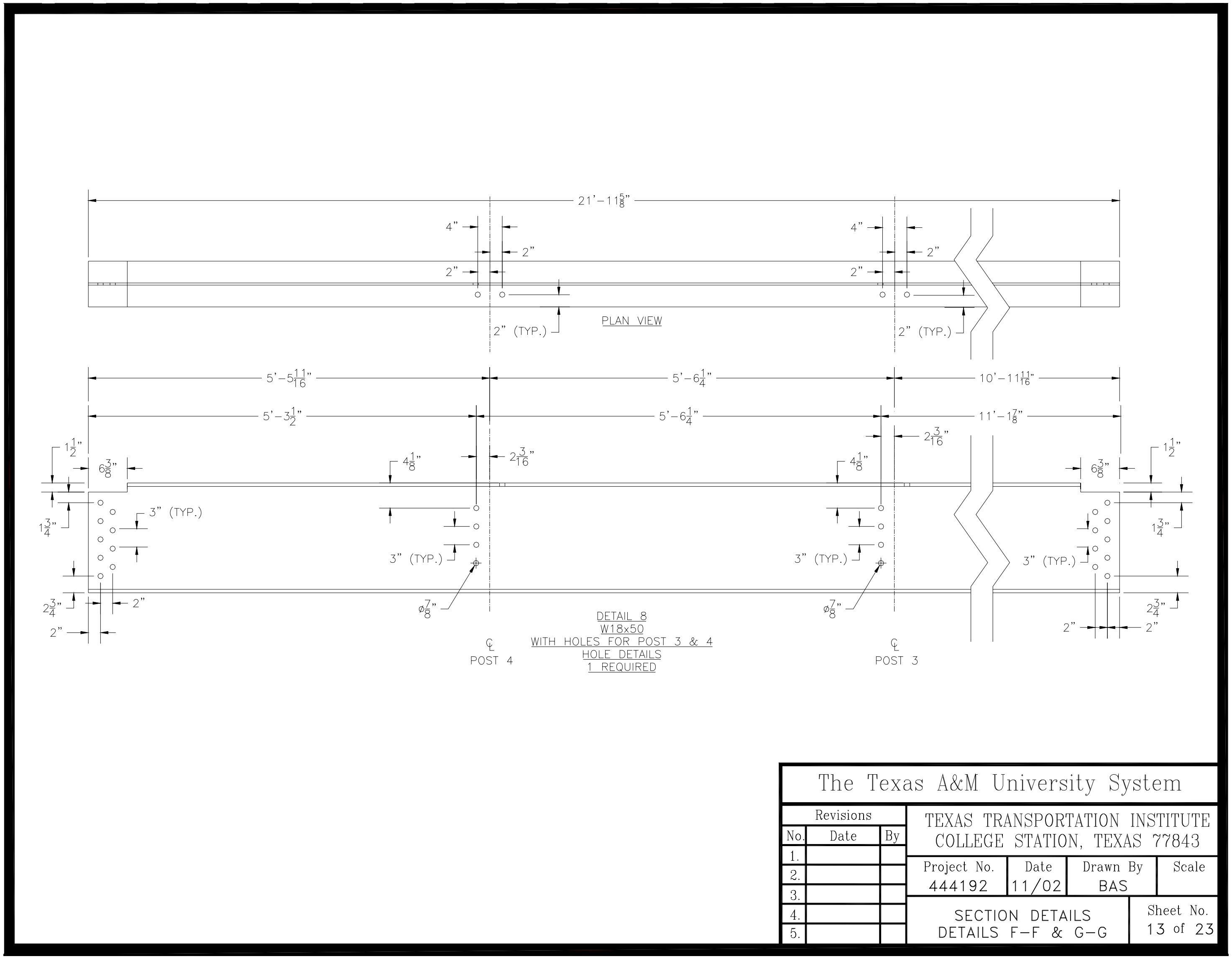


Figure 13. Stringer Details for Posts 3 and 4.

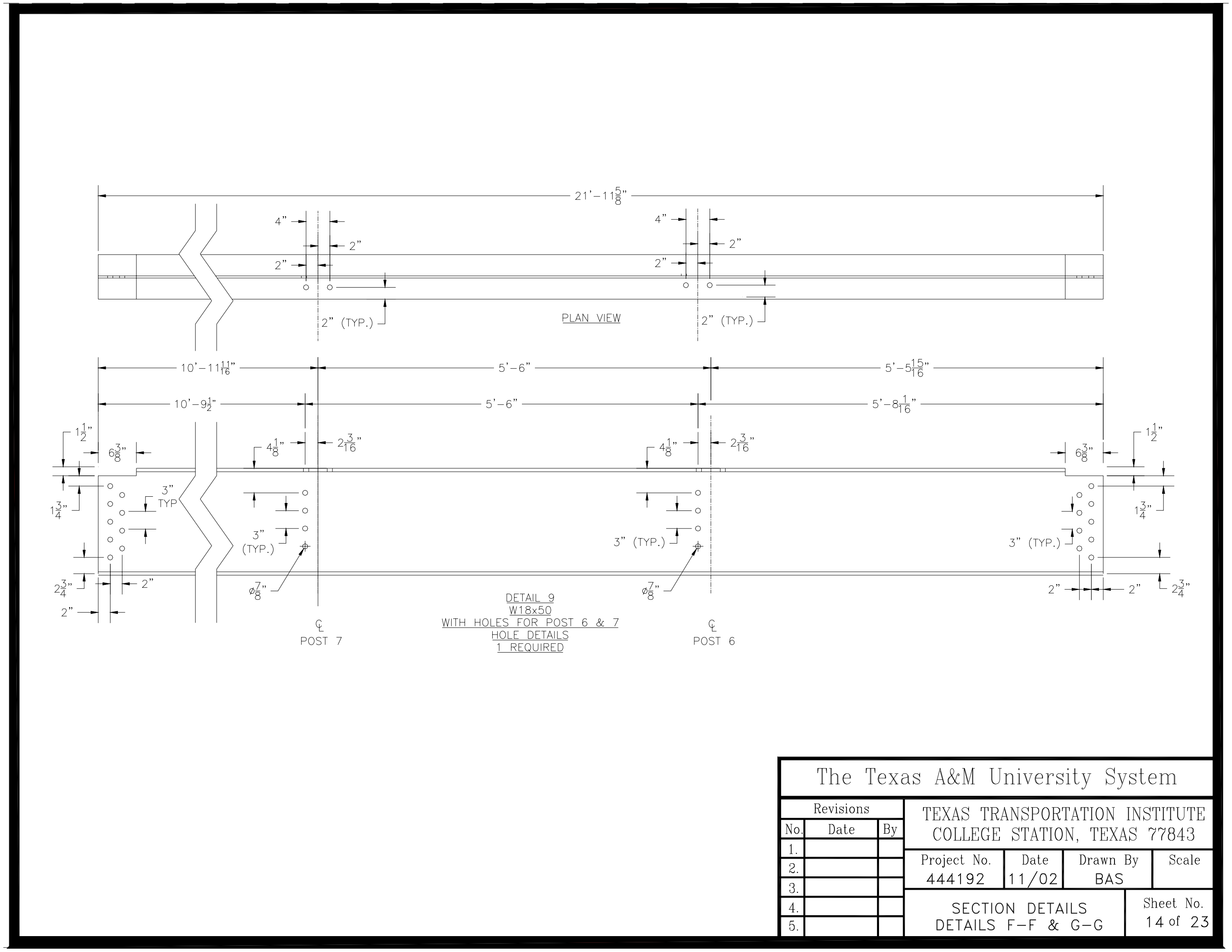


Figure 14. Stringer Details for Posts 6 and 7.

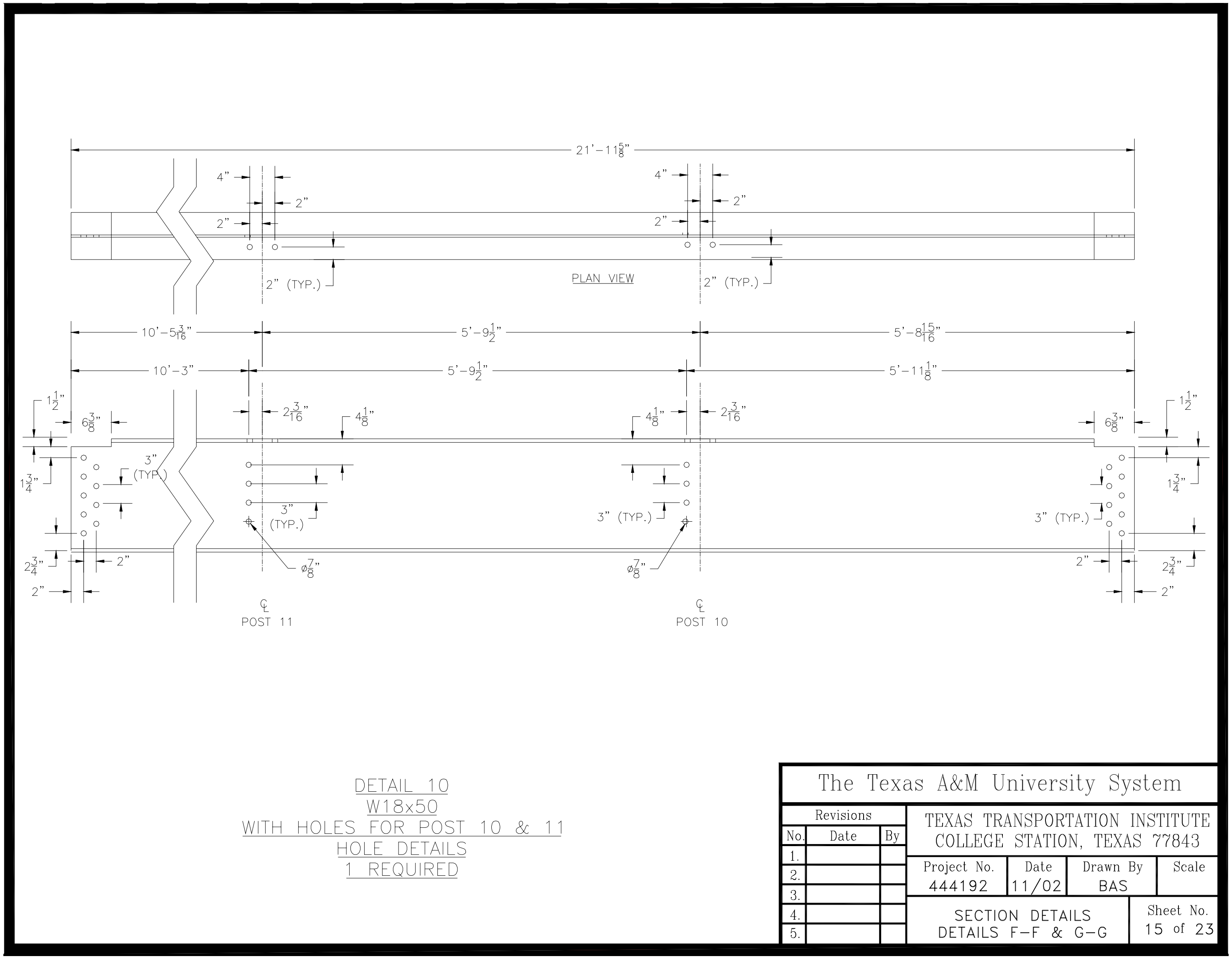


Figure 15. Stringer Details for Posts 10 and 11.

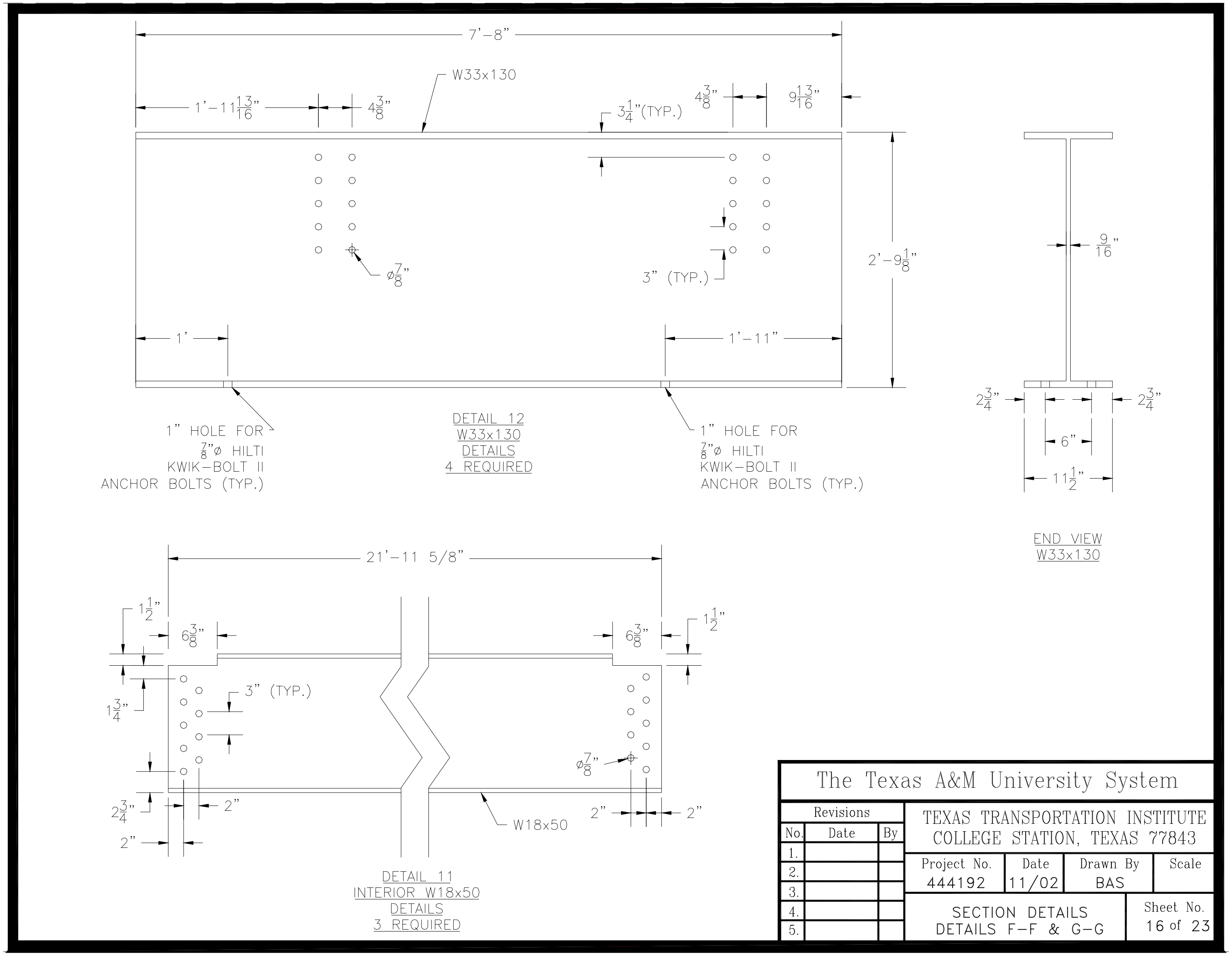
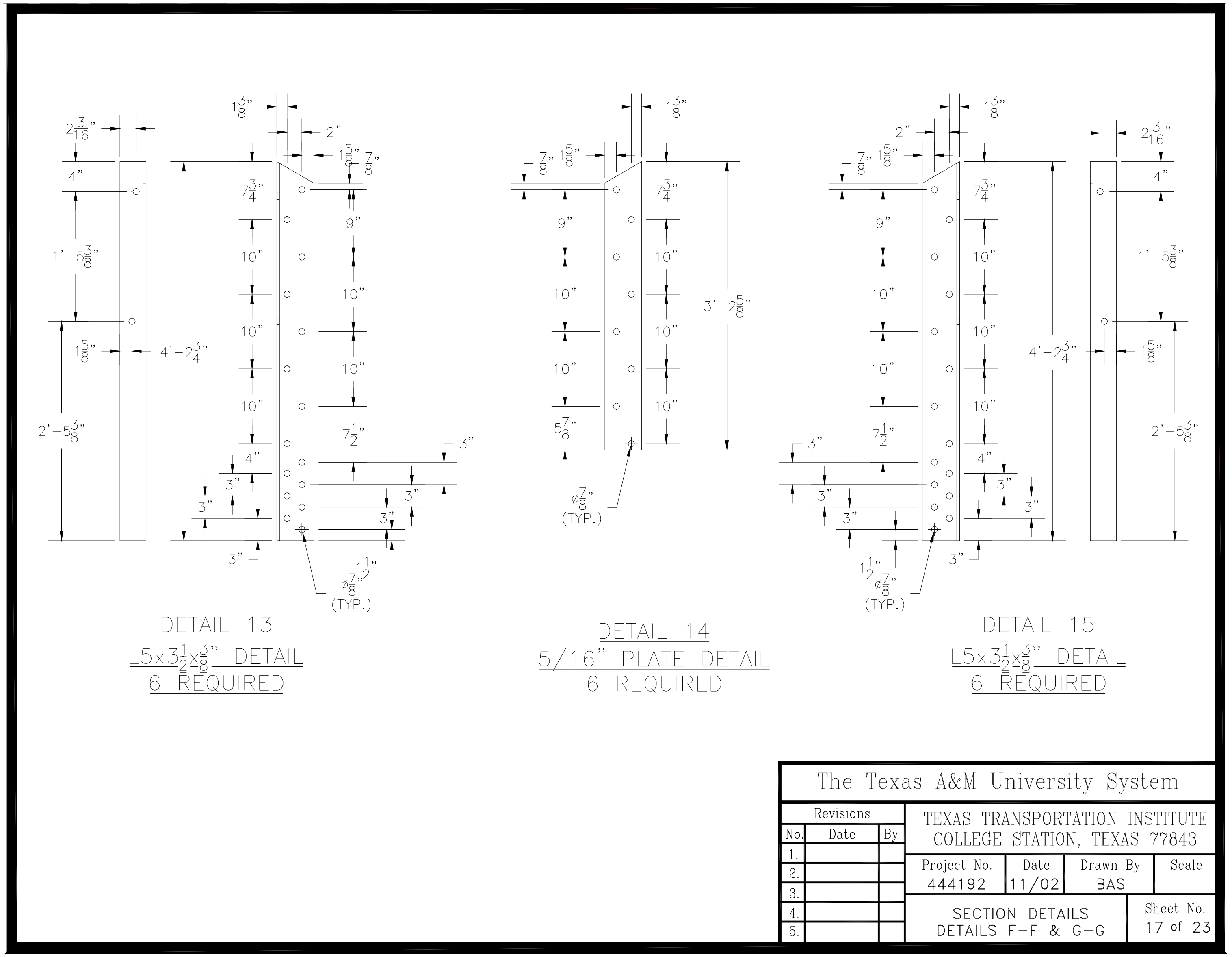


Figure 16. Details of Girder and Interior Stringer.



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4.			SECTION DETAILS DETAILS F-F & G-G			Sheet No. 17 of 23
5.						

Figure 17. Details of Bridge Rail Posts.

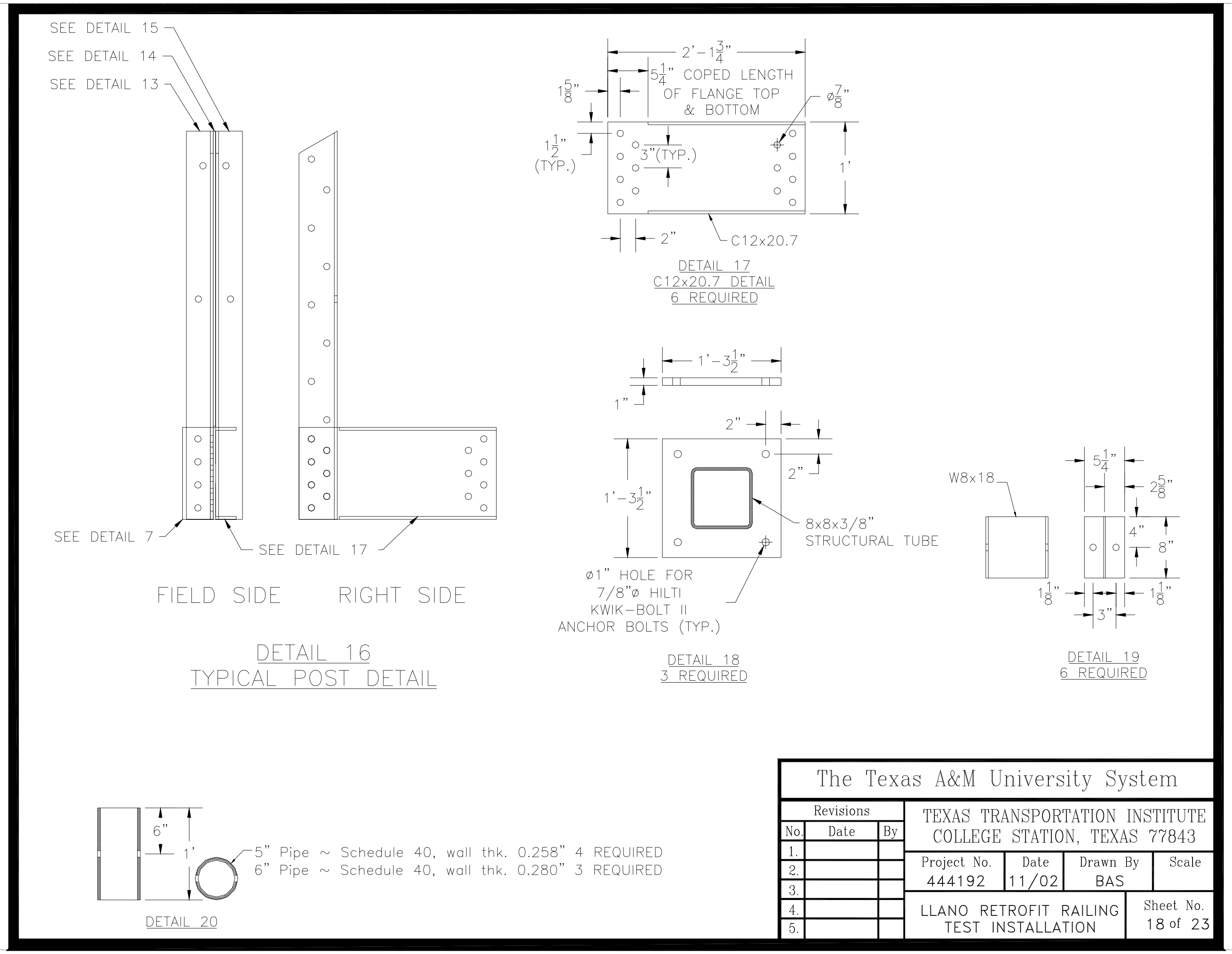


Figure 18. Details of Bridge Rail Posts, Baseplate, Blockouts, and Crushable Steel Pipe Tube.

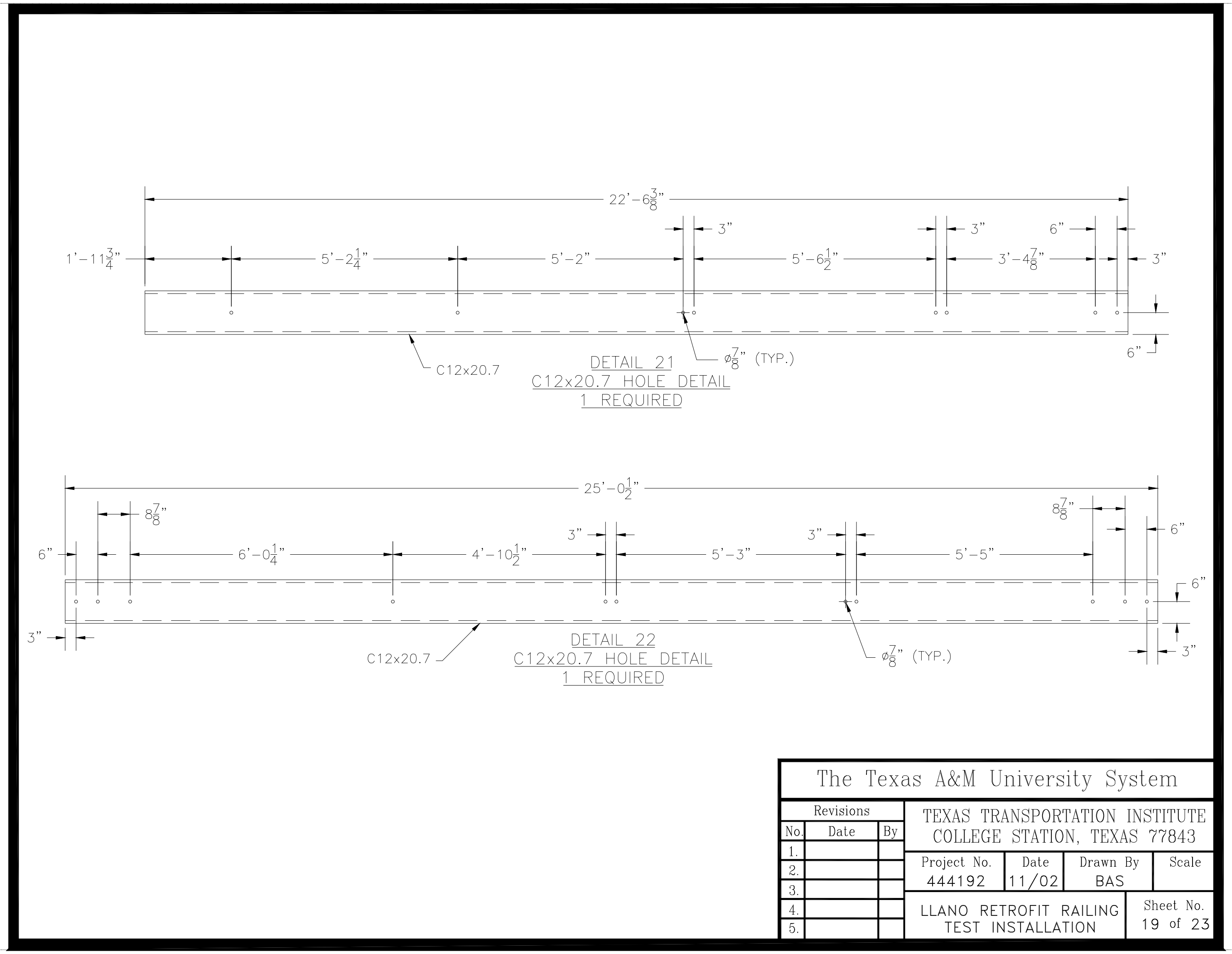


Figure 19. Details of C12×20.7 Bridge Rail.

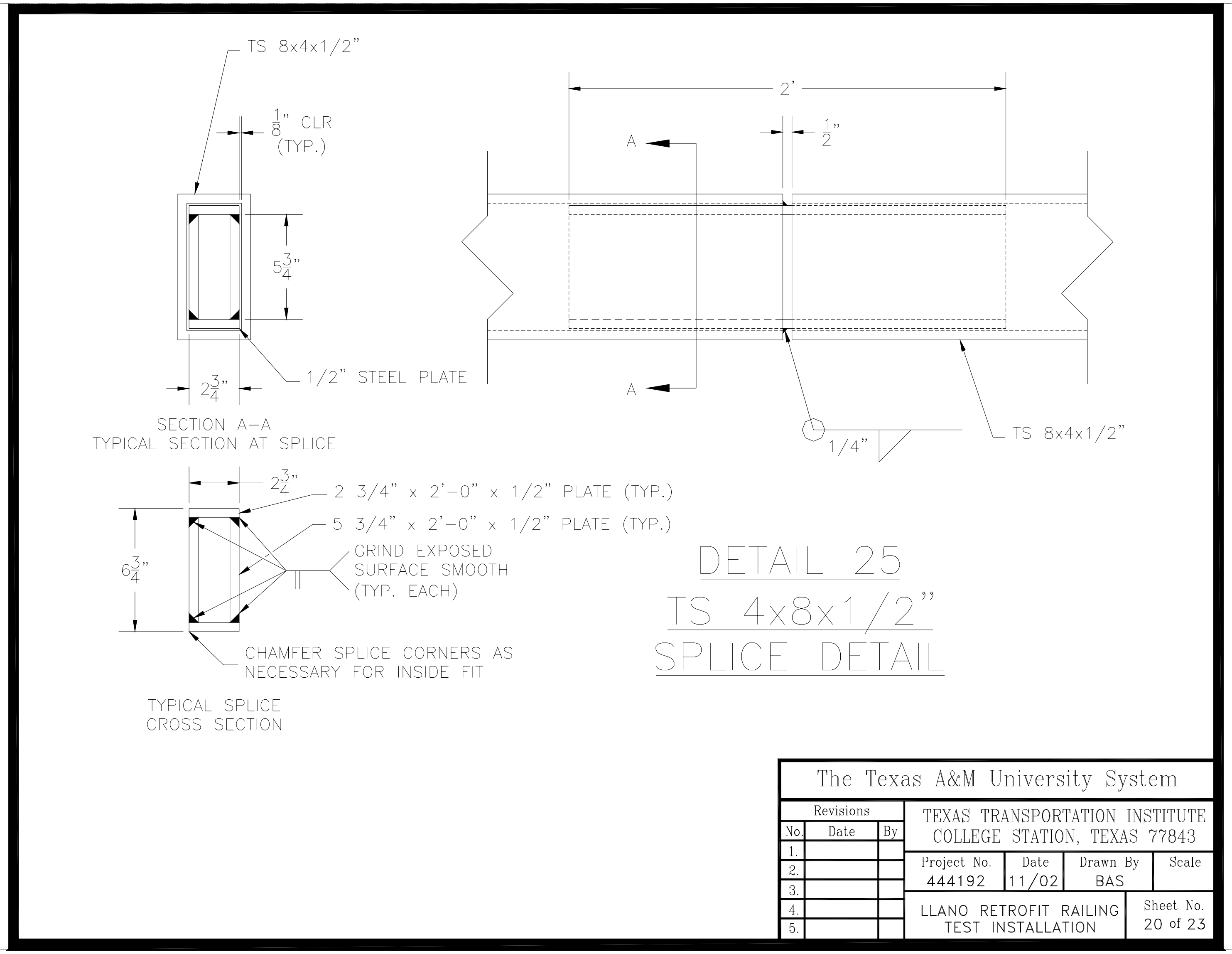


Figure 20. Details of TS 4×8×1/2 Splice.

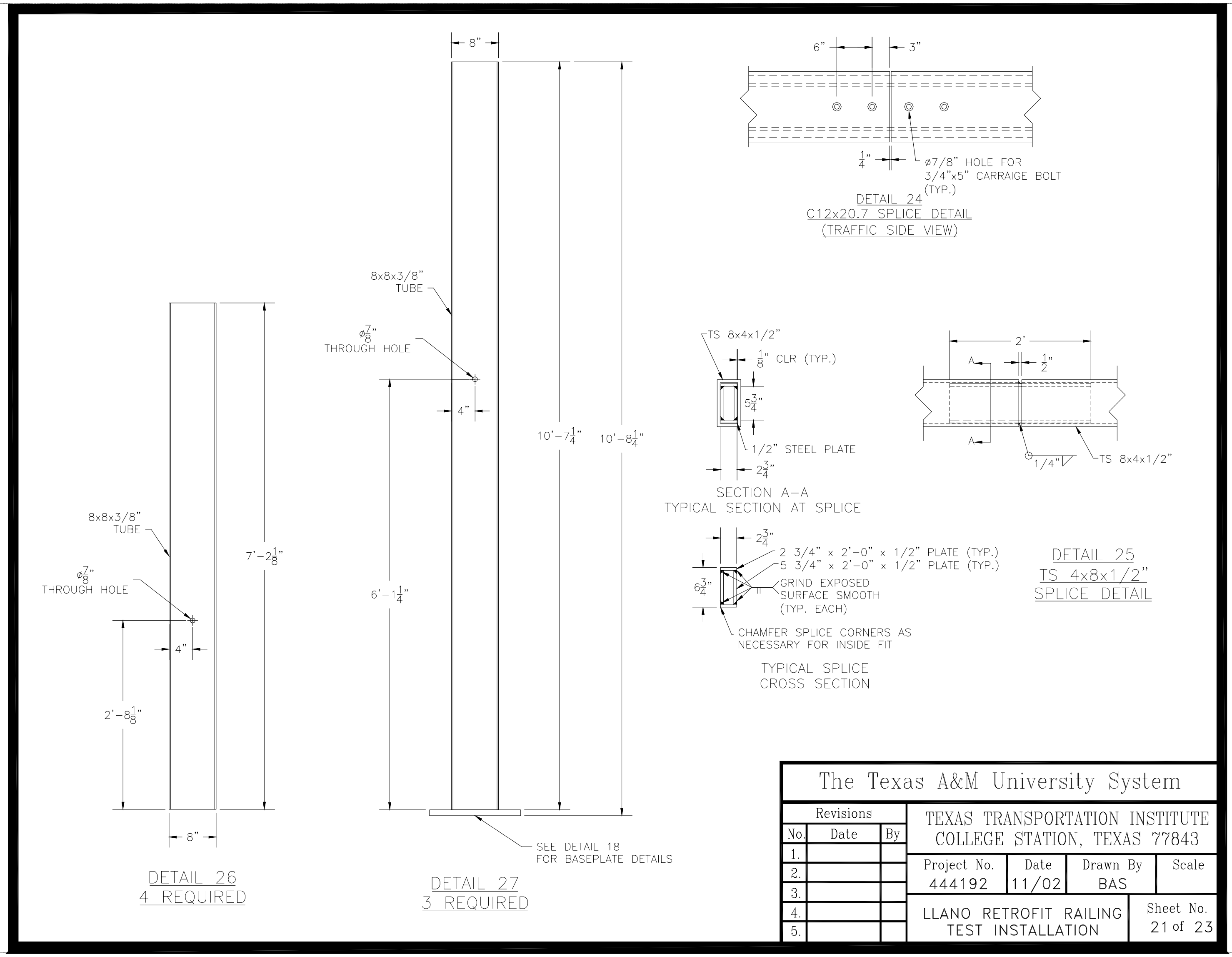
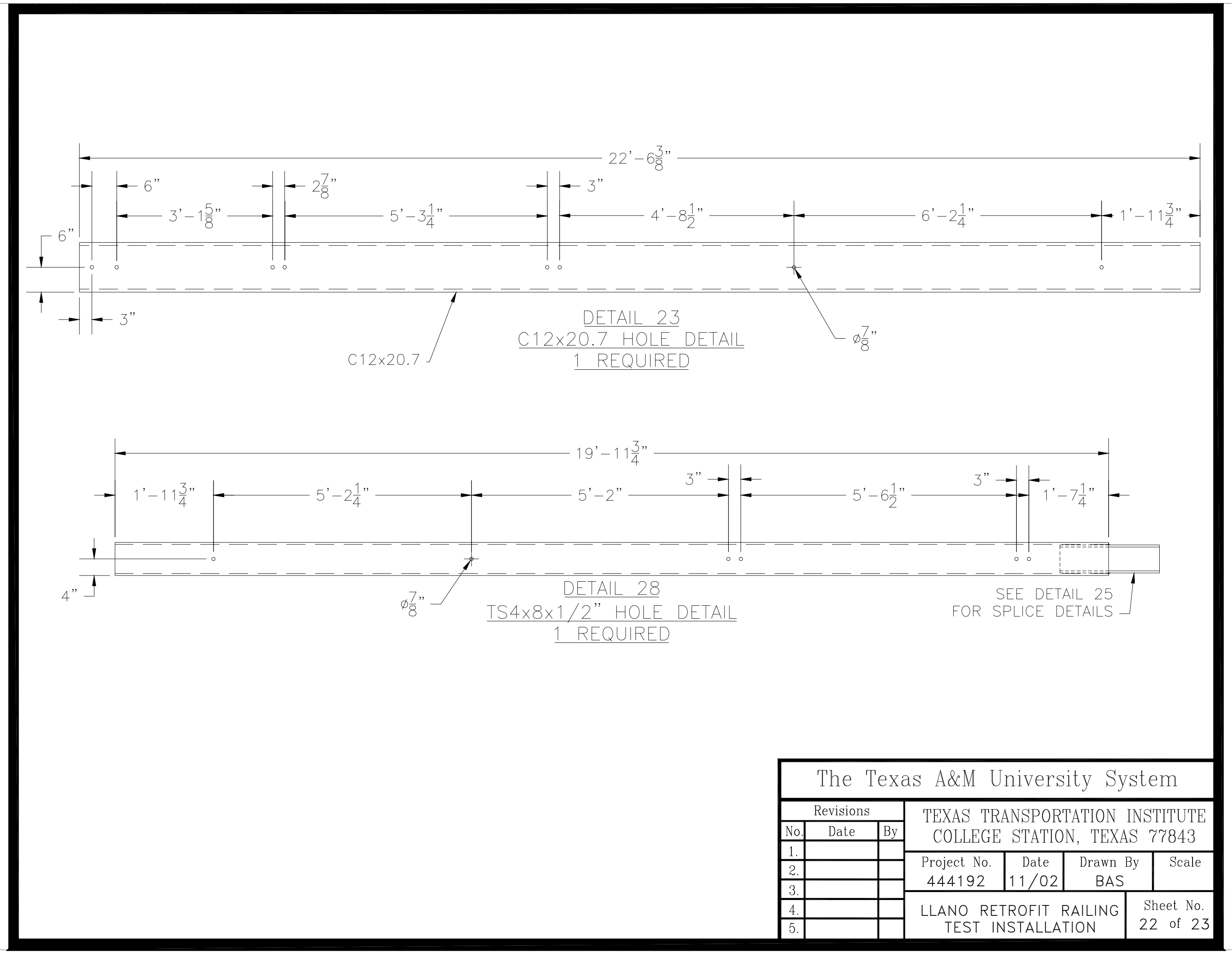


Figure 21. Details of 8×8×3/8 Tube and C12×20.7 Splice.



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444192	11/02	BAS	

LLANO RETROFIT RAILING
TEST INSTALLATION

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Figure 22. Details of C12×20.7 and TS 4×8×1/2 Bridge Rail.

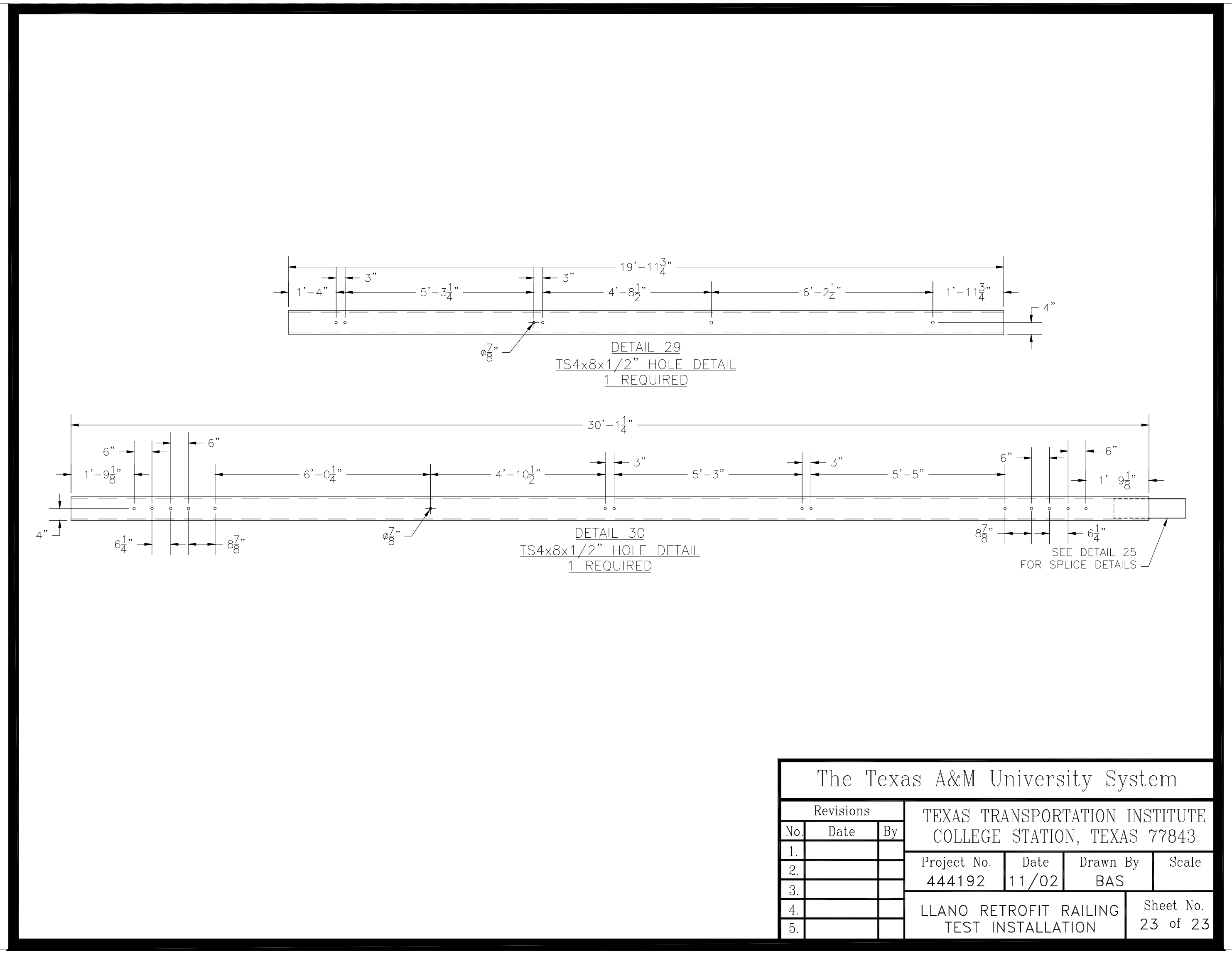


Figure 23. Details of TS 4×8×1/2 Bridge Rail.

40 (ksi) (275 MPa). At present, this grade of reinforcing steel is uncommon and difficult to obtain. To account for the lower grade of reinforcing steel, 60 ksi (413 MPa) yield strength reinforcing steel was used and spaced at a greater distance according to the ratio of strength between the two grades of reinforcement. In the bottom layer of reinforcement in the deck, #5 (#16) bars "A" were spaced 1 ft 3 inches (0.38 m) on centers (see Figure 11). Transverse reinforcement in the curb consisted of #4 (#16) bars "D" on 11-1/4-inch (286 mm) centers (see Figure 10). Longitudinal reinforcement in the curb consisted of one #6 (#19) bar on the inside and at the top of the transverse bars "D" in the curb.

The retrofit bridge railing consisted of a TS8×4×1/2 (TS203×102×13) tube with a C12×20.7 (C310×31) attached to the traffic side face (see Figures 19, 22, and 23). Rectangular splices for the TS8×4×1/2 (TS203×102×13) were fabricated from 1/2-inch (13 mm) thick steel plates, 2 ft 0 inches (0.6 m) in length (see Figure 20). The C12×20.7 (C310×31), which was attached to the traffic side face of the TS8×4×1/2 (TS203×102×13), was spliced at the joint locations (see Figure 21). The rail was blocked out at the post locations using a piece of W8×18 (W200×27), 8 inches (203.2 mm) in length (see Figure 18). The height to the top of the bridge rail was 2 ft 8 inches (0.81 m).

To reduce the collision loads into the truss members, crushable steel pipes were used in lieu of rigid steel blocks. At the truss member locations, 5-inch (127 mm) and 6-inch (152 mm) diameter schedule 40 steel pipe blocks were designed to have a "crush" strength of approximately 8 kips (35.60 kN). In the test installation, steel tubes were used at all truss member locations to represent the truss members (see Figures 18 and 21). Depending on the geometry of the truss member, 5-inch (127 mm) diameter or 6-inch (152 mm) diameter crushable steel pipe tubes, 12 inches (3.6 m) in length, were used between the rigid steel tubes that served as a surrogate for the truss members and the bridge rail (see Figure 18). For an overall view of the test installation, please refer to Figure 1. Photographs of the completed installation are shown in Figure 24.

TEST NO. 444193-1 (*NCHRP REPORT 350* TEST NO. 2-11)

Test Vehicle

A 1998 Chevrolet Cheyenne 2500 pickup truck, shown in Figures 25 and 26, was used for the crash test. Test inertia weight of the vehicle was 4579 lb (2079 kg), and its gross static weight was 4579 lb (2079 kg). The height to the lower edge of the vehicle bumper was 16.3 inches (415 mm), and the height to the upper edge of the bumper was 25.0 inches (636 mm). Additional dimensions and information on the vehicle are given in Figure 55 of Appendix D. The vehicle was directed into the installation using the cable reverse tow and guidance system and was released to be freewheeling and unrestrained just prior to impact.

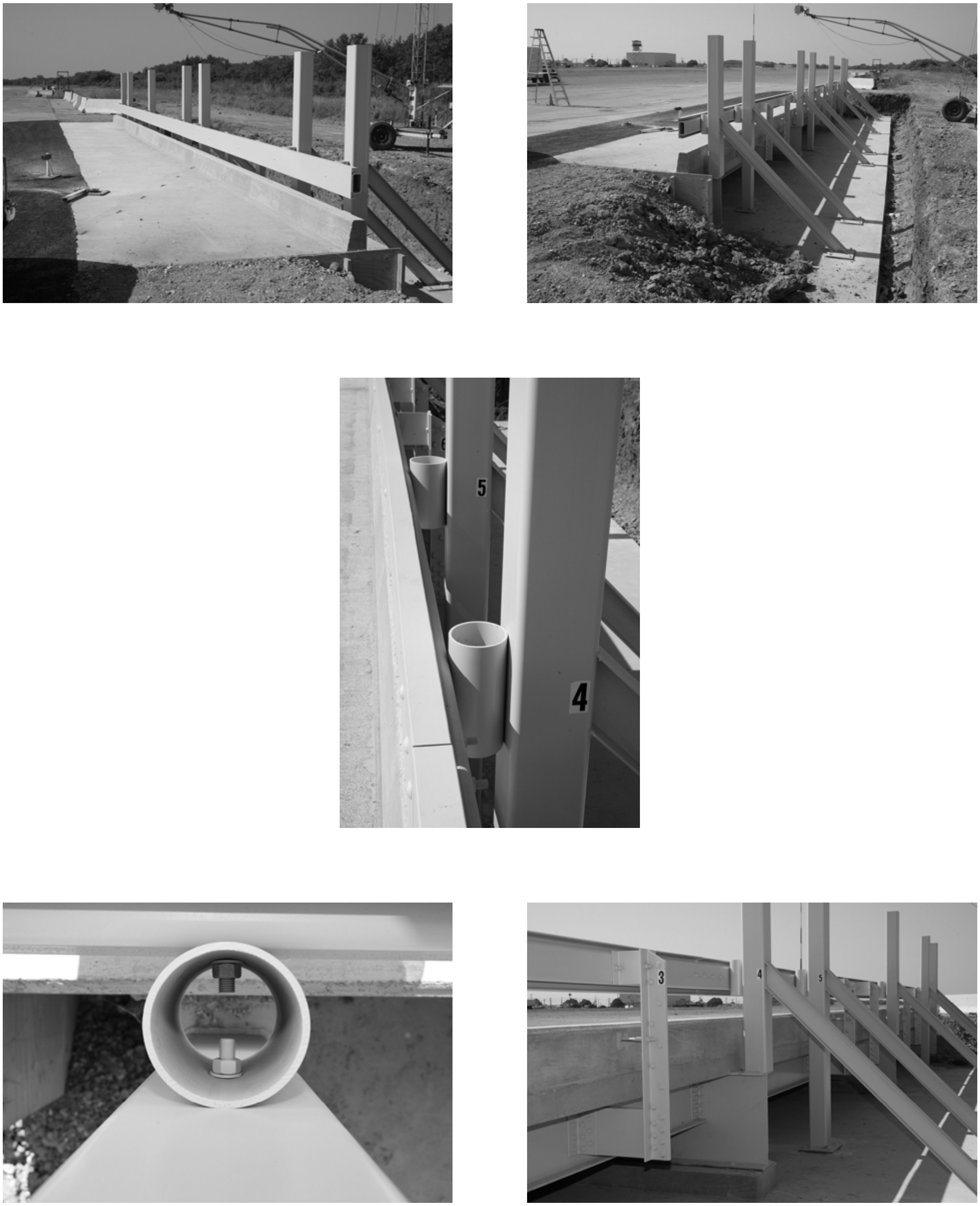


Figure 24. Llano Truss Bridge Installation before Test 444193-1.

Figure 25. Vehicle/Installation Geometrics for Test 444193-1.

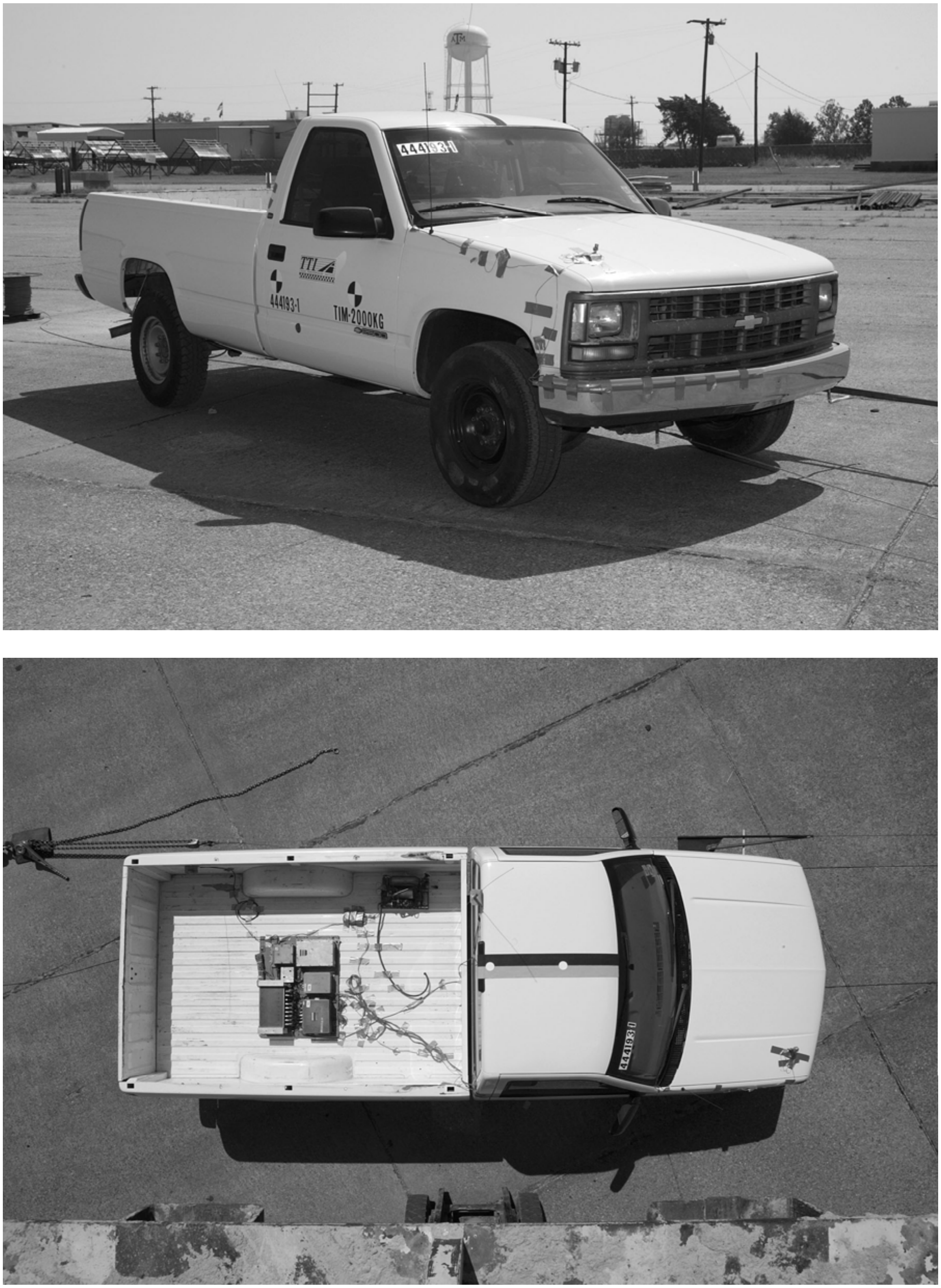


Figure 26. Vehicle before Test 444193-1.

Soil and Weather Conditions

The test was performed on the morning of May 30, 2003. Rainfall of 2 mm (0.078 inches) was recorded nine days prior to the test. Weather conditions at the time of testing were as follows: wind speed: 6 mi/h (9 km/h); wind direction: 350 degrees with respect to the vehicle (vehicle was traveling in a southwesterly direction); temperature: 91ºF (33ºC); and relative humidity: 46 percent.

Test Description

The vehicle, traveling at 44.4 mi/h (71.5 km/h), impacted the Llano Truss Bridge Rail 0.74 m (2.43 ft) upstream of post 5 at an impact angle of 25.5 degrees. Shortly after impact, the right front tire contacted the curb, and by 0.032 s, the right front tire reached the rail element. The steel tube blockout at post 5 began to deform at 0.035 s, and the right front tire blew out at 0.045 s. The vehicle began to redirect at 0.048 s, and a fine crack on the rear of the deck at post 4 began to form at 0.055 s. At 0.221 s, the vehicle was traveling parallel with the bridge rail at a speed of 39.9 mi/h (64.2 km/h). The rear of the vehicle contacted the rail element at 0.242 s, and post 4 began to deflect toward the field side at 0.247 s. The concrete around the anchor bolts at post 4 began to spall at 0.254 s, and post 6 began to deflect toward the field side at 0.264 s. At 0.372 s, the vehicle lost contact with the bridge rail while traveling at a speed of 38.6 mi/h (62.2 km/h) and an exit angle of 5.0 degrees. Brakes on the vehicle were applied 3.5 s after impact, and the vehicle subsequently yawed clockwise, contacted a protective barrier, and came to rest adjacent to this barrier 172.6 ft (52.6 m) downstream of impact and 6.2 ft (1.9 m) forward of the traffic face of the bridge rail. Figures 57 and 58 in Appendix E present sequential photographs of the test period.

Damage to Test Installation

Damage to the Truss Bridge Rail is shown in Figures 27 and 28. There were tire marks on the face of the rail and the curb beginning 28.8 inches (740 mm) upstream of post 5 and continuing for a distance of 9.8 ft (2.99 m), which was the length of contact of the vehicle with the bridge rail. Post 4 deflected toward field side 0.4 inches (11 mm), and the concrete around the anchor bolts on the rear side of the deck spalled. The steel tube blockout at post 5 was crushed 0.8 inches (21 mm), and the blockout at post 6 was crushed 0.2 inches (6 mm). Maximum dynamic deflection during the test was 1.8 inches (47 mm).

Figure 27. After Impact Trajectory Path for Test 444193-1.

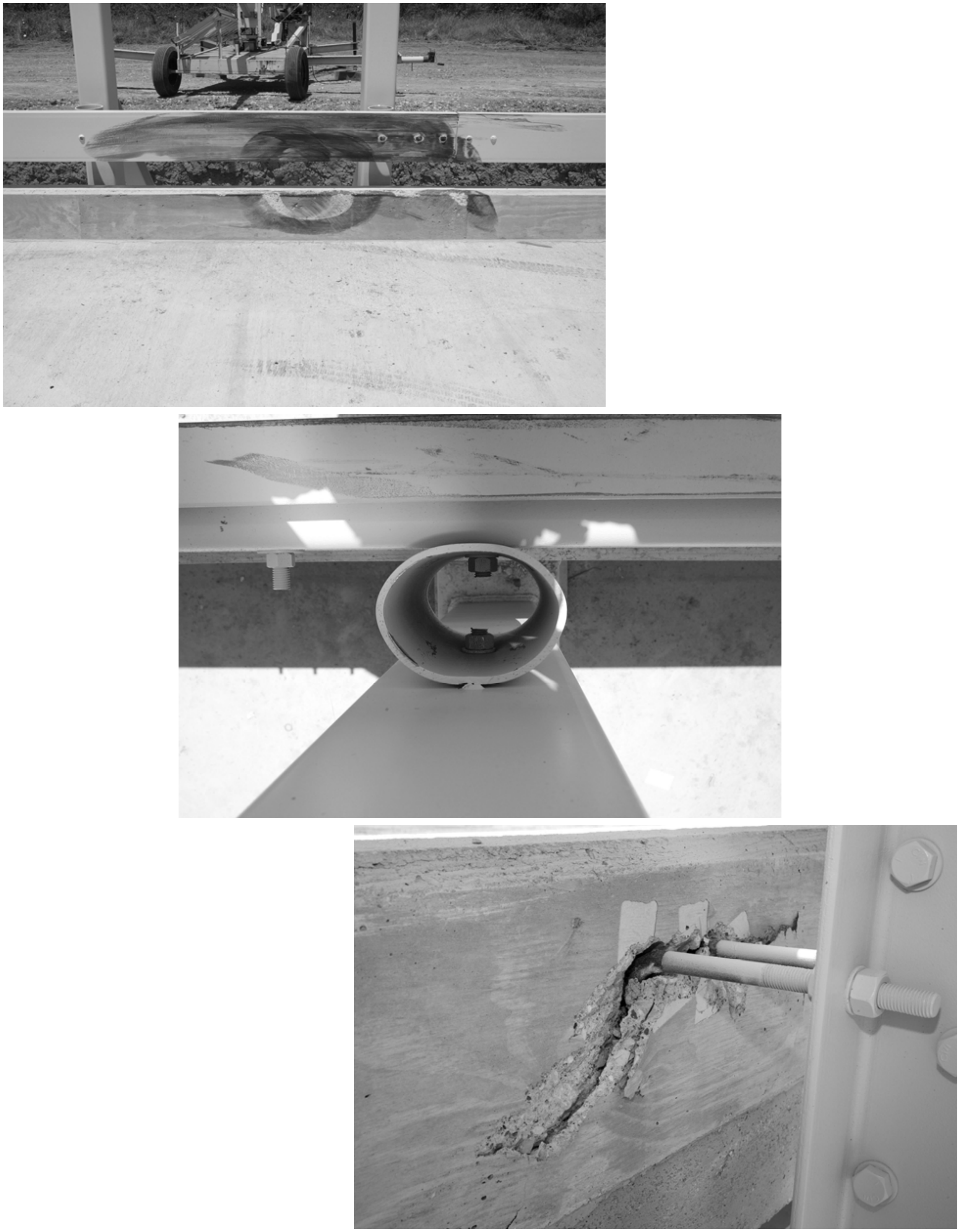


Figure 28. Installation after Test 444193-1.

Vehicle Damage

The pickup sustained moderate damage, as shown in Figure 29. Structural damage was imparted to the right upper and lower A-arms, right outer tie rod end, and floor pan, and the right front frame rail was deformed. Also damaged were the front bumper, hood, grill, right front quarter panel, right front tire and wheel rim, right door, right rear exterior bed, right rear tire, and rear bumper. Maximum exterior crush to the vehicle was 15.7 inches (400 mm) in the side plane at the right front corner at bumper height. Maximum occupant compartment deformation was 0.4 inches (10 mm) in the lateral kick panel area near the passenger's feet. Photographs of the interior of the vehicle are shown in Figure 30. Tables 7 and 8 in Appendix D show exterior crush and occupant compartment deformations.

Occupant Risk Factors

Data from the triaxial accelerometer, located at the vehicle center of gravity, were digitized to compute occupant impact velocity and ridedown accelerations. Only the occupant impact velocity and ridedown accelerations in the longitudinal axis are required from these data for evaluation of criterion L of *NCHRP Report 350*. In the longitudinal direction, the occupant impact velocity was 13.8 ft/s (4.2 m/s) at 0.114 s, the highest 0.010-s occupant ridedown acceleration was -7.0 g's from 0.267 to 0.277 s, and the maximum 0.050-s average acceleration was -6.7 g's between 0.058 and 0.108 s. In the lateral direction, the occupant impact velocity was 20.3 ft/s (6.2 m/s) at 0.114 s, the highest 0.010-s occupant ridedown acceleration was -11.5 g's from 0.269 to 0.279 s, and the maximum 0.050-s average was -9.6 g's between 0.054 and 0.104 s. These data and other pertinent information from the test are summarized in Figure 31. Figures 61 through 67 in Appendix F present vehicle angular displacements and accelerations versus time traces.

ASSESSMENT OF TEST RESULTS

An assessment of the test based on the applicable *NCHRP Report 350* safety evaluation criteria is provided below.

Structural Adequacy

A. Test article should contain and redirect the vehicle; the vehicle should not penetrate, underride, or override the installation although controlled lateral deflection of the test article is acceptable.

Results: The Truss Bridge Rail contained and redirected the 2000P pickup truck. The 2000P pickup truck did not penetrate, underride, or override the installation. Maximum dynamic deflection during the test was 1.8 inches (47 mm). (PASS)

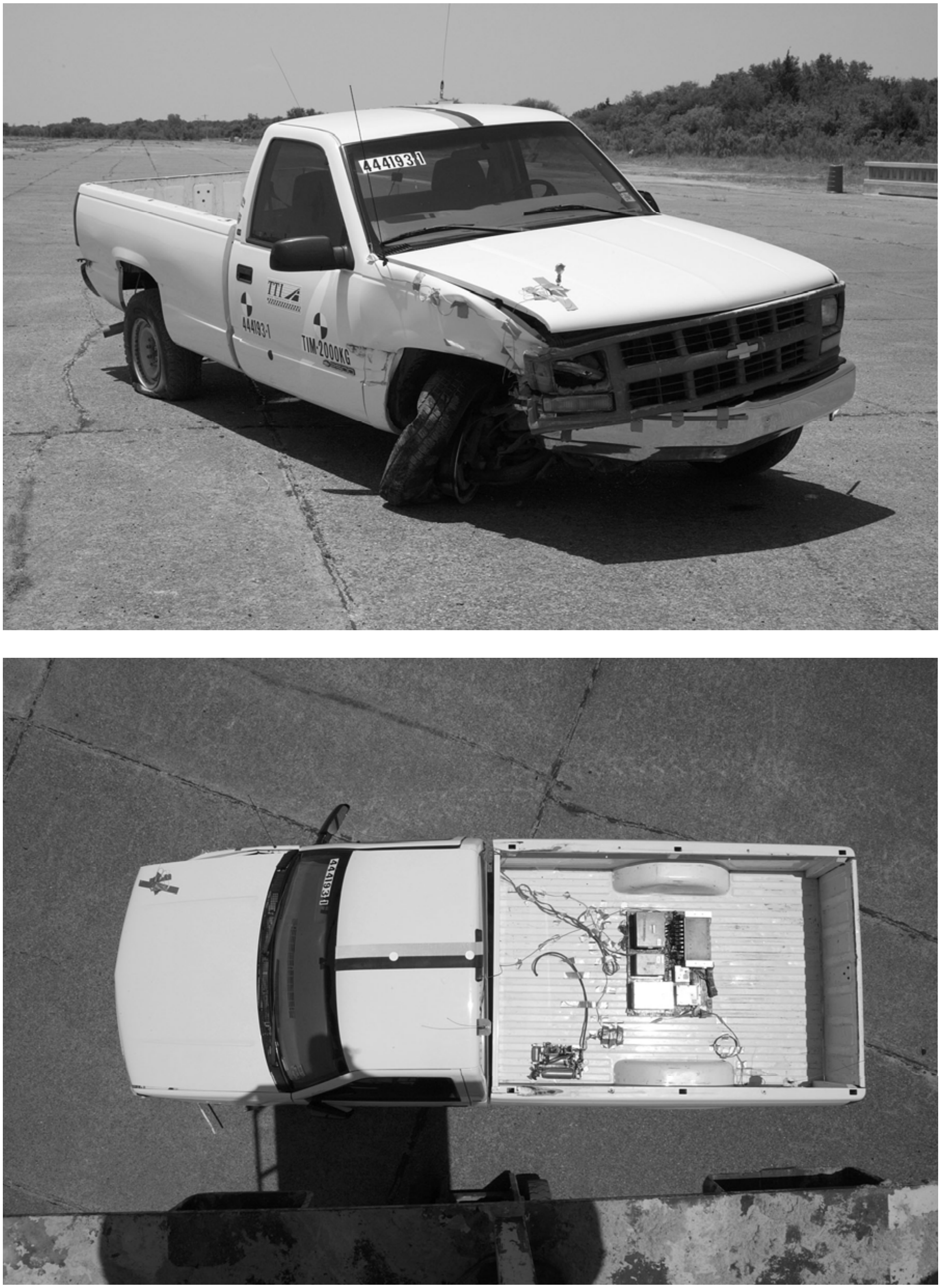
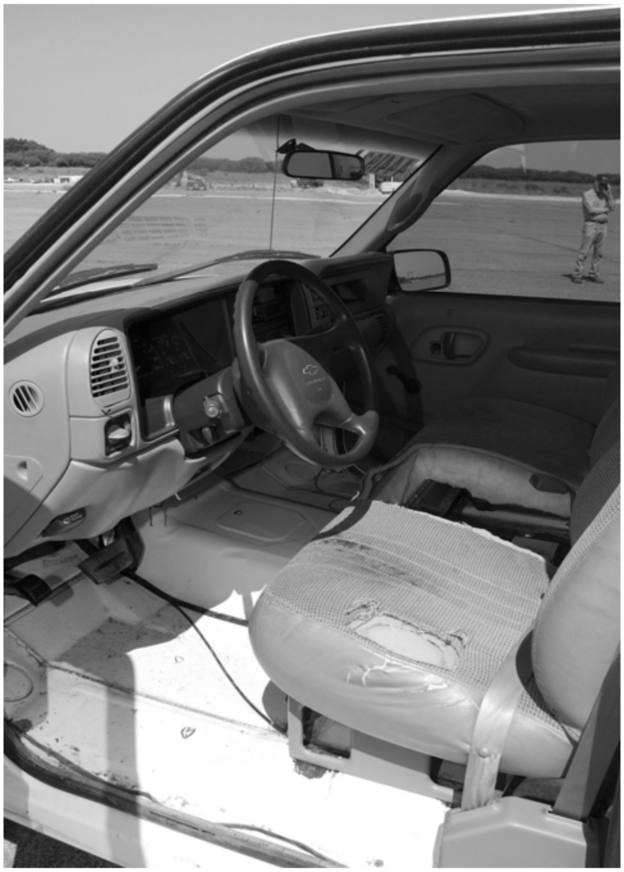


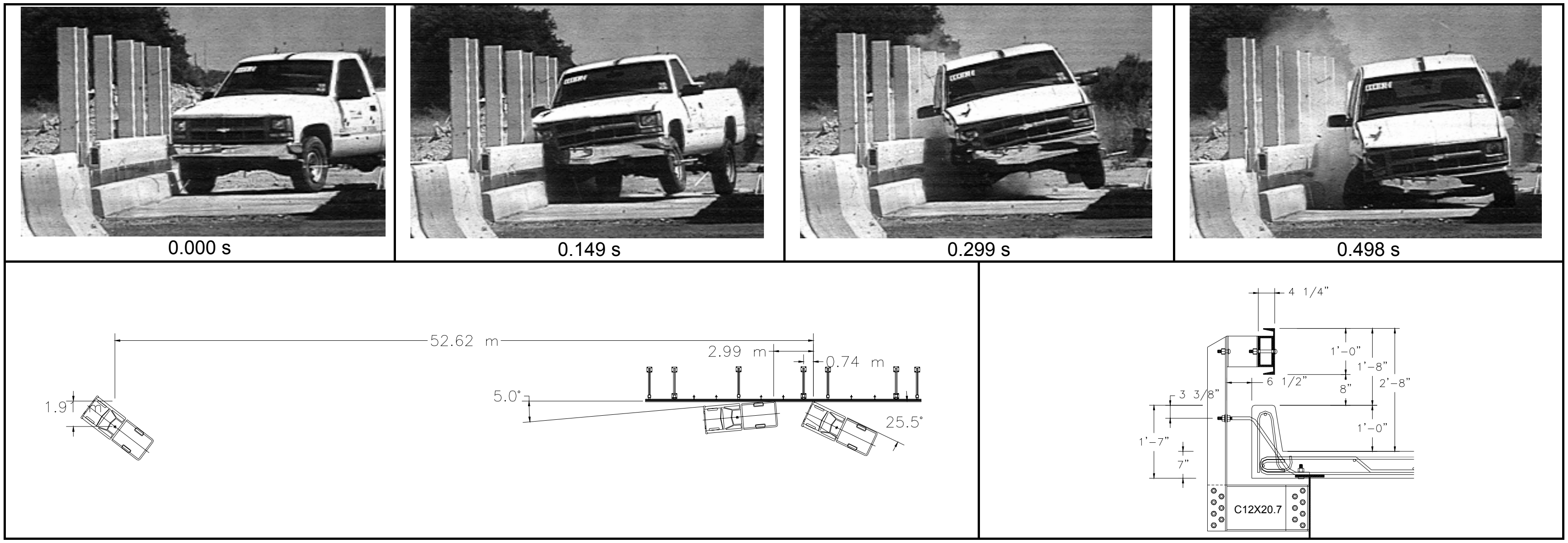
Figure 29. Vehicle after Test 444193-1.



Before Test

After Test

Figure 30. Interior of Vehicle for Test 444193-1.


0.000 s 0.149 s 0.299 s 0.498 s

General Information	
Test Agency	Texas Transportation Institute
Test No.	444193-1
Date	05/30/03
Test Article	
Type	Bridge Rail Retrofit
Name	Llano Truss Bridge Rail Retrofit
Installation Length (ft)	70 (21.3 m)
Material or Key Elements	Steel Tube Rail with Steel Posts and Blockouts
Soil Type and Condition	Concrete Deck, Dry
Test Vehicle	
Type	Production
Designation	2000P
Model	1998 Chevrolet Cheyenne 2500 Pickup
Weight (lb)	
Curb	4441 (2017 kg)
Test Inertial	4579 (2079 kg)
Dummy	N/A
Gross Static	4579 (2079 kg)

Impact Conditions	
Speed (mi/h)	44.4 (71.5 km/h)
Angle (deg)	25.5
Exit Conditions	
Speed (mi/h)	38.6 (62.2 km/h)
Angle (deg)	5.0
Occupant Risk Values	
Impact Velocity (ft/s)	
x-direction	13.8 (4.2 m/s)
y-direction	20.3 (6.2 m/s)
THIV (mi/h)	4.6 (7.4 km/h)
Ridedown Acceleration (g's)	
x-direction	-7.0
y-direction	-11.5
PHD (g's)	13.0
ASI	1.19
Max. 0.050-s Average (g's)	
x-direction	-6.7
y-direction	-9.6
z-direction	2.7

Test Article Deflection (inches)	
Dynamic	1.8 (47 mm)
Permanent	0.8 (21 mm)
Working Width	13.6 (349 mm)
Vehicle Damage	
Exterior	
VDS	01FR2
CDC	01FREW2
Maximum Exterior Vehicle Crush (inches)	15.6 (400 mm)
Interior	
OCDI	RF0001000
Max. Occ. Compart. Deformation (inches)	0.4 (10 mm)
Post-Impact Behavior	
(during 1.0 s after impact)	
Max. Yaw Angle (deg)	-30.0
Max. Pitch Angle (deg)	12.6
Max. Roll Angle (deg)	9.0

Figure 31. Summary of Results for *NCHRP Report 350* Test 2-11 on the Llano Truss Bridge Rail.

Occupant Risk

D. *Detached elements, fragments, or other debris from the test article should not penetrate or show potential for penetrating the occupant compartment, or present an undue hazard to other traffic, pedestrians, or personnel in a work zone. Deformation of, or intrusions into, the occupant compartment that could cause serious injuries should not be permitted.*

Results: No detached elements, fragments, or other debris was present to penetrate or to show potential for penetrating the occupant compartment or to present undue hazard to others in the area. Maximum occupant compartment deformation was 0.4 inches (10 mm) in the kick panel area near the passenger's feet, laterally across the cab. (PASS)

F. *The vehicle should remain upright during and after collision although moderate roll, pitching, and yawing are acceptable.*

Results: The vehicle remained upright during and after the collision event. (PASS)

Vehicle Trajectory

K. *After collision, it is preferable that the vehicle's trajectory not intrude into adjacent traffic lanes.*

Results: The vehicle came to rest upright 172.6 ft (52.6 m) downstream of impact and 6.2 ft (1.9 m) forward of the traffic face of the rail. (PASS)

L. *The occupant impact velocity in the longitudinal direction should not exceed 12 m/s and the occupant ridedown acceleration in the longitudinal direction should not exceed 20 g's.*

Results: Longitudinal occupant impact velocity was 13.8 ft/s (4.2 m/s) and longitudinal ridedown acceleration was -7.0 g's. (PASS)

M. *The exit angle from the test article preferably should be less than 60 percent of the test impact angle, measured at time of vehicle loss of contact with the test device.*

Results: Exit angle at loss of contact was 5.0 degrees, which was 20 percent of the impact angle. (PASS)

The following supplemental evaluation factors and terminology, as presented in the FHWA memo entitled "Action: Identifying Acceptable Highway Safety Features," were used for visual assessment of test results. Factors underlined below pertain to the results of the crash test reported herein.

Passenger Compartment Intrusion

1. *Windshield Intrusion*
    - *[✓ a. No windshield contact]*
    - *b. Windshield contact, no damage*
    - *c. Windshield contact, no intrusion*
    - *d. Device embedded in windshield, no significant intrusion*
    - *e. Complete intrusion into passenger compartment*
    - *f. Partial intrusion into passenger compartment*
2. *Body Panel Intrusion* — *[✓ yes] or no*

Loss of Vehicle Control

- *[✓ 1. Physical loss of control]*
- *2. Loss of windshield visibility*
- *3. Perceived threat to other vehicles*
- *4. Debris on pavement*

Physical Threat to Workers or Other Vehicles

1. *Harmful debris that could injure workers or others in the area*
2. *Harmful debris that could injure occupants in other vehicles*

No debris was present.

Vehicle and Device Condition

1. *Vehicle Damage*
    - *a. None*
    - *b. Minor scrapes, scratches or dents*
    - *c. Significant cosmetic dents*
    - *d. Major dents to grill and body panels*
    - *[✓ e. Major structural damage]*
2. *Windshield Damage*
    - *[✓ a. None]*
    - *b. Minor chip or crack*
    - *c. Broken, no interference with visibility*
    - *d. Broken or shattered, visibility restricted but remained intact*
    - *e. Shattered, remained intact but partially dislodged*
    - *f. Large portion removed*
    - *g. Completely removed*
3. *Device Damage*
    - *a. None*
    - *b. Superficial*
    - *[✓ c. Substantial, but can be straightened]*
    - *d. Substantial, replacement parts needed for repair*
    - *e. Cannot be repaired*

CHAPTER 4. U.S. 281 TRUSS BRIDGE RAIL

ANALYSIS AND DESIGN

TxDOT engineers worked closely with researchers at TTI to develop a retrofit bridge railing for the U.S 281 Bridge over the Brazos River in Palo Pinto County, Texas. Their goal was to develop a crashworthy design for high-speed application that preserves the historical character of the bridge. Initially, TxDOT personnel and the TTI researchers decided to improve safety performance by lowering the bridge rail from a height of 37 inches (940 mm) above the existing pavement surface to a height of 30 inches (762 mm). Maintaining the visual appearance of the existing C12×20.7 (C310×31) bridge rail by adding another rail element behind this rail was also preferred. Researchers decided to maintain the clear roadway width of 24 ft (7.3 m). Several truss members on existing truss bridges have been damaged by vehicular collision. The goal of this research was to develop a crashworthy design with sufficient structural capacity to meet the *NCHRP Report 350* TL-3 requirements. Several different conceptual designs were developed for this project and are presented in Appendix G. A variation of Option #4 was selected by the TxDOT project team. TTI researchers and the TxDOT project team worked closely to develop a conceptual design for analysis. After a conceptual design was developed, calculations were performed on the new retrofit design to determine if it had sufficient structural capacity to meet the requirements for TL-3 impact conditions as stated in the current *AASHTO LRFD Bridge Design Specifications*. The capacity of the new retrofit design developed by TTI researchers did meet the minimum AASHTO TL-3 strength requirements.

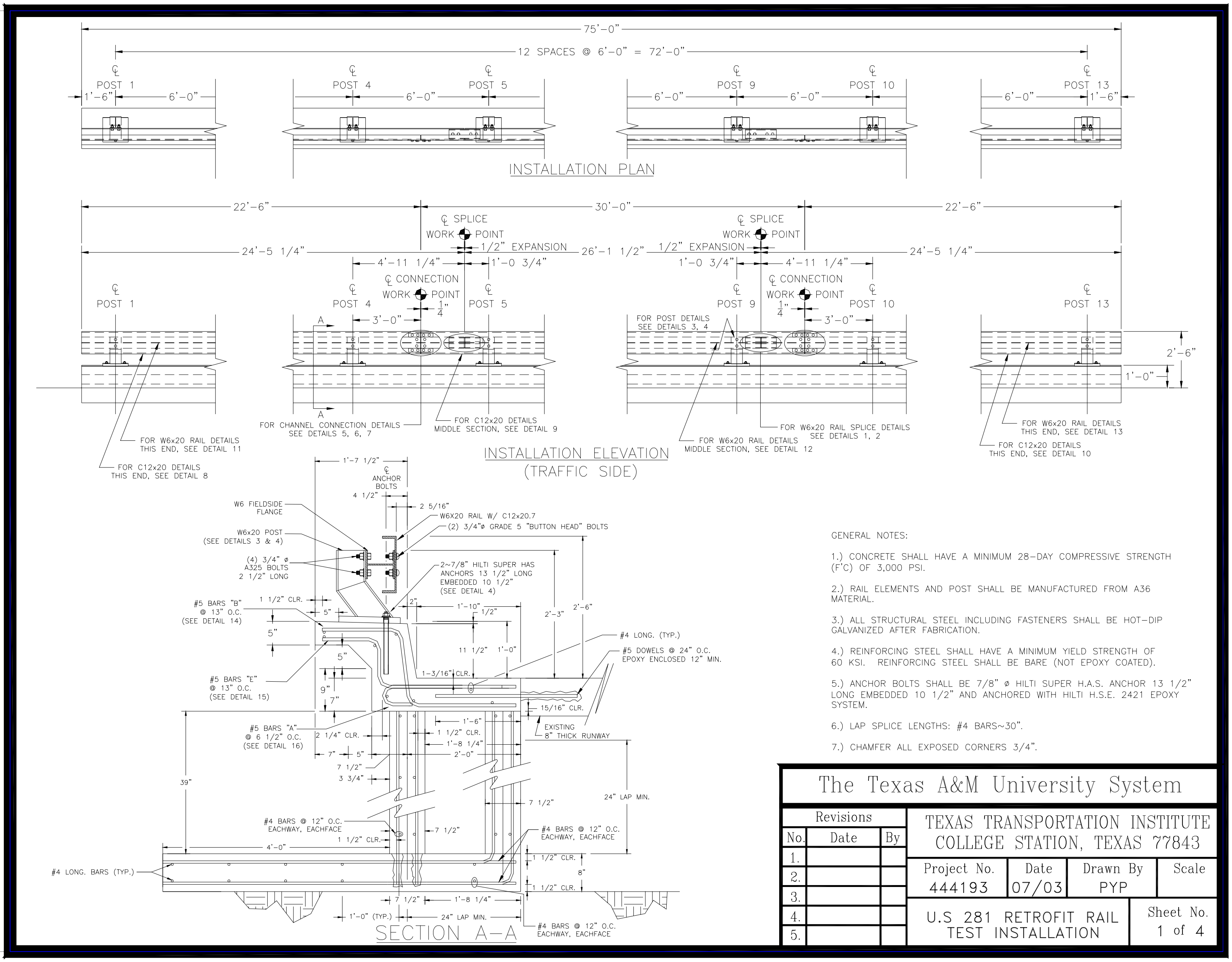
The conceptual design selected for testing utilized a new rail system mounted on top of the existing concrete curb. This design relocates the bridge rail away from the truss members and provides a clear space between the bridge rail and the truss members. An analysis was performed to determine the structural adequacy of the curb to support a new rail system. The test installation constructed for this project included a simulated portion of the actual bridge superstructure supporting the concrete deck and curb. The new retrofit rail was attached directly to the top of the curb using chemical epoxy anchor bolts, which were also incorporated into the design of the new rail system. Finite element analyses were performed on the post to determine stresses in the post and baseplate and to determine forces in the bolts. Based on these analyses, TTI researchers recommended that the prototype bridge rail be 30 inches (762 mm) in height and be supported separately from the truss members by the concrete curb. To maintain the existing structural appearance of the bridge, the existing C12×20.7 (C310×31) rail was mounted in front of a W6×20 (W150×30) rail in the new retrofit design. The new rail is supported by fabricated W6×20 (W150×30) steel posts anchored into the existing concrete curb using adhesive anchors. The retrofit bridge railing strength calculations are presented in Appendix G.

TEST INSTALLATION PROTOTYPE

TTI received drawings from TxDOT entitled "Brazos River Bridge, U.S. Highway No. 281, Palo Pinto County" dated July 13, 1937. Details from these drawings were used to prepare construction and fabrication drawings for the test installation for this project, shown as Figures 32 through 35. The existing U.S. 281 Bridge consists of three steel spans with two spans measuring 202 ft 0 inches (61.6 m) and the third middle span measuring 253 ft 6 inches (77.3 m). The shorter spans have eight panels, each measuring 25 ft 3 inches (7.7 m), and the longer middle span has 10 panels, each measuring also 25 ft 3 inches (7.7 m). The total length of the bridge including the approach structures is 1138 ft 4 inches (347 m). For this project, TTI constructed approximately 75 ft (22.8 m) of concrete curb and deck similar to the actual deck and curb on the existing bridge structure. The width of the concrete deck constructed for this project was 2 ft 0 inches (610 mm). The existing concrete curb and deck is supported by five longitudinal stringers spaced 6 ft (1.8 m) on centers spanning between the W36×150 (W920×223) panel beams. The exterior stringers are W18×55 (W460×82) and the interior stringers are W21×68 (W530×101). In lieu of constructing W18×55 (W460×82) exterior stringers to support the deck and curb, a 7-1/2-inch (191 mm) wide concrete wall was constructed and used to support the simulated concrete deck and curb.

The simulated concrete deck constructed for this project was 7-1/2 inches (191 mm) thick. The curb was 12 inches (305 mm) high, 1 ft 7-1/2 inches (495 mm) wide at the top, and 9-1/2 inches (241 mm) wide at the gutter line. The top of the curb sloped downward 1/2 inch (12 mm) over the full top width of the curb. Transverse reinforcement in the deck consisted of #5 (#16) "hooked" bars at 6-1/2 inches (165 mm) on centers. Transverse reinforcement in the curb consisted of #5 (#16) "Z"-shaped bars at 13 inches (330 mm) on centers on both the traffic and field faces of the curb. Longitudinal reinforcement in the deck and curb consisted of #4 (#12) bars spaced within the transverse "hooked" bars, at bends in the curb reinforcement, and at representative locations within the actual deck. Concrete compressive strength tests were performed on representative samples of the deck and curb concrete just prior to testing. The concrete compressive strength on the deck and curb concrete was 3908 psi (27 MPa) and 4158 psi (29 MPa), respectively.

A new retrofit steel bridge rail was designed and constructed for the U.S. 281 Bridge over the Brazos River in Palo Pinto County, Texas. The bridge rail consisted of W6×20 (W150×30) steel posts and bridge railing with a C12×20.7 (C310×31) attached to the face of the W6 (W150) bridge rail. The C12×20.7 (C310×31) was attached to the flange of the W6×20 (W150×30) to maintain the historical appearance of the new retrofit design. The W6×20 (W150×30) steel posts were 1 ft 3-1/4 inches (387 mm) in height and spaced 6 ft 0 inches (1.83 m) on centers. The posts were attached to 1 ft 1-1/2 inch (343 mm) × 1 ft 1 inch (330 mm) × 1 inch (25 mm) thick steel baseplates. The posts were fabricated such that the top of the post was offset 6-3/8 inches from the base of the post to provide additional roadway clearance for traffic. The C12×20.7 (C310×31) was attached to the front flange of the W6×20 (W150×30) rail using 3/4-inch (19 mm) diameter Grade 8, "button-head" machine bolts, 3 inches (75 mm) in length.



GENERAL NOTES:

1.) CONCRETE SHALL HAVE A MINIMUM 28-DAY COMPRESSIVE STRENGTH (F'C) OF 3,000 PSI.

2.) RAIL ELEMENTS AND POST SHALL BE MANUFACTURED FROM A36 MATERIAL.

3.) ALL STRUCTURAL STEEL INCLUDING FASTENERS SHALL BE HOT-DIP GALVANIZED AFTER FABRICATION.

4.) REINFORCING STEEL SHALL HAVE A MINIMUM YIELD STRENGTH OF 60 KSI. REINFORCING STEEL SHALL BE BARE (NOT EPOXY COATED).

5.) ANCHOR BOLTS SHALL BE 7/8" ø HILTI SUPER H.A.S. ANCHOR 13 1/2" LONG EMBEDDED 10 1/2" AND ANCHORED WITH HILTI H.S.E. 2421 EPOXY SYSTEM.

6.) LAP SPLICE LENGTHS: #4 BARS~30".

7.) CHAMFER ALL EXPOSED CORNERS 3/4".

The Texas A&M University System

TEXAS TRANSPORTATION INSTITUTE
COLLEGE STATION, TEXAS 77843

Revisions		
No.	Date	By
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Project No.	Date	Drawn By	Scale
444193	07/03	PYP	

U.S 281 RETROFIT RAIL TEST INSTALLATION

Sheet No. 1 of 4

Figure 32. Overall Details of the U.S. 281 Truss Bridge Installation.

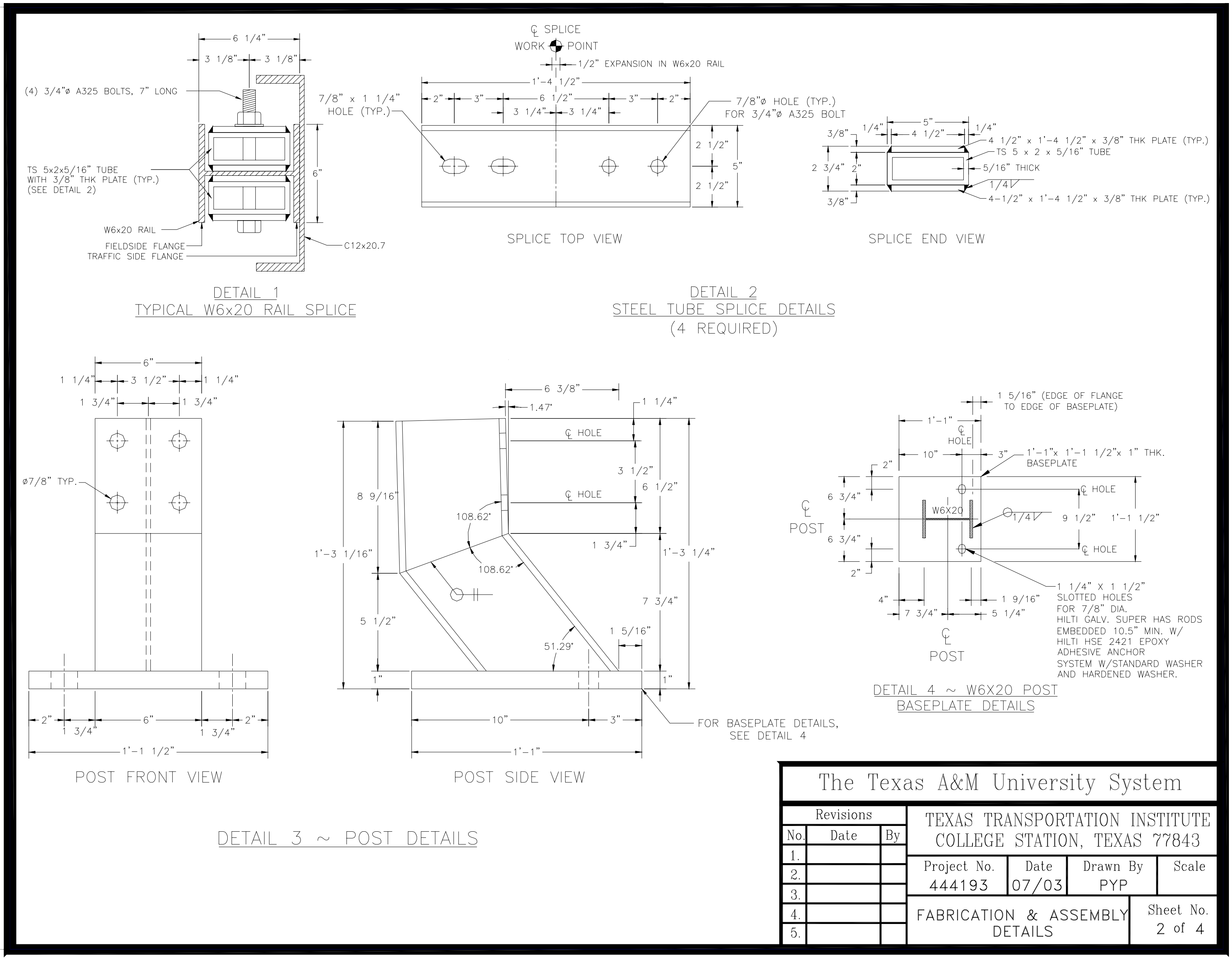


Figure 33. Fabrication and Assembly Details for U.S. 281 Truss Bridge Installation.

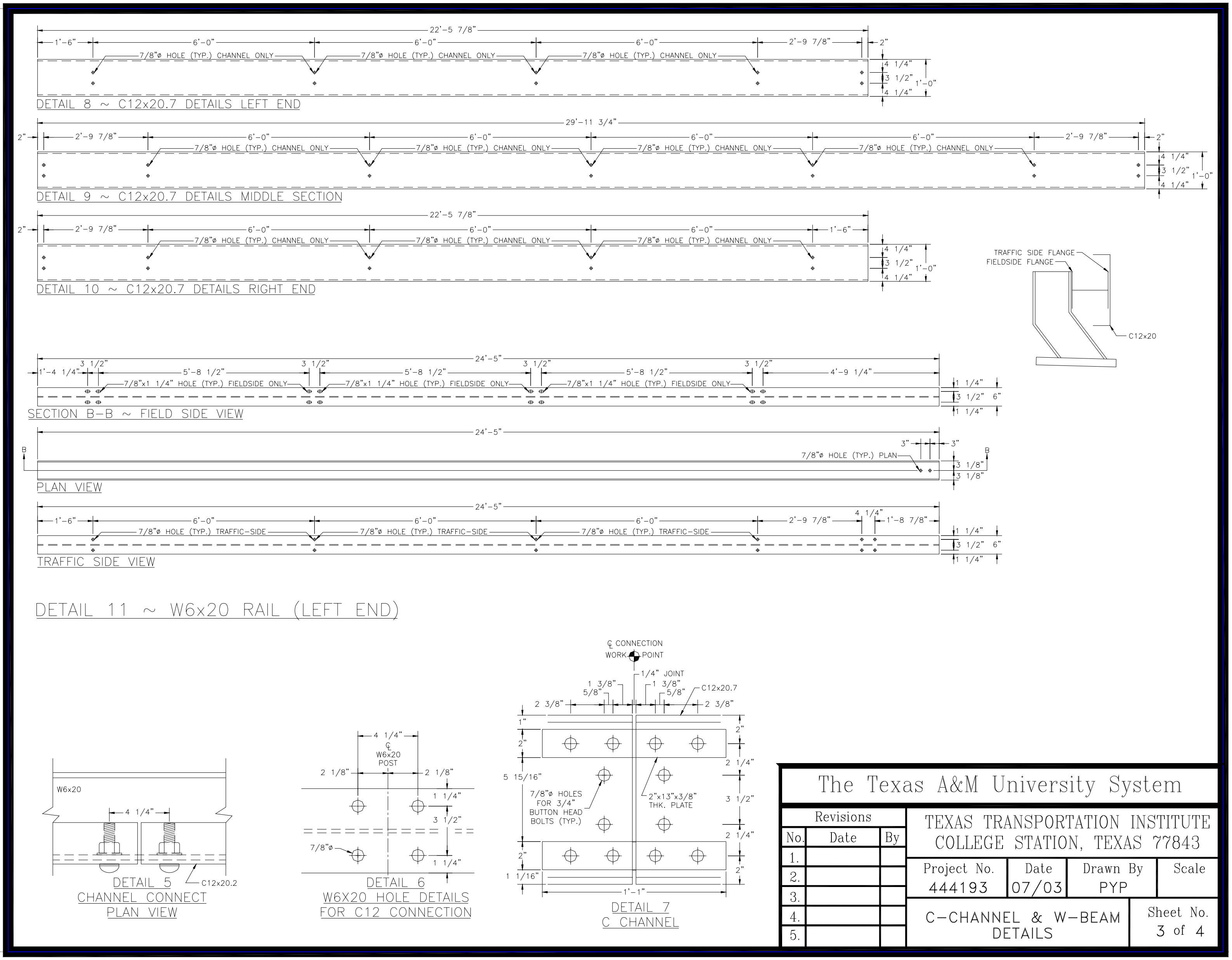


Figure 34. C-Channel and W-Beam Details for U.S. 281 Truss Bridge Installation.

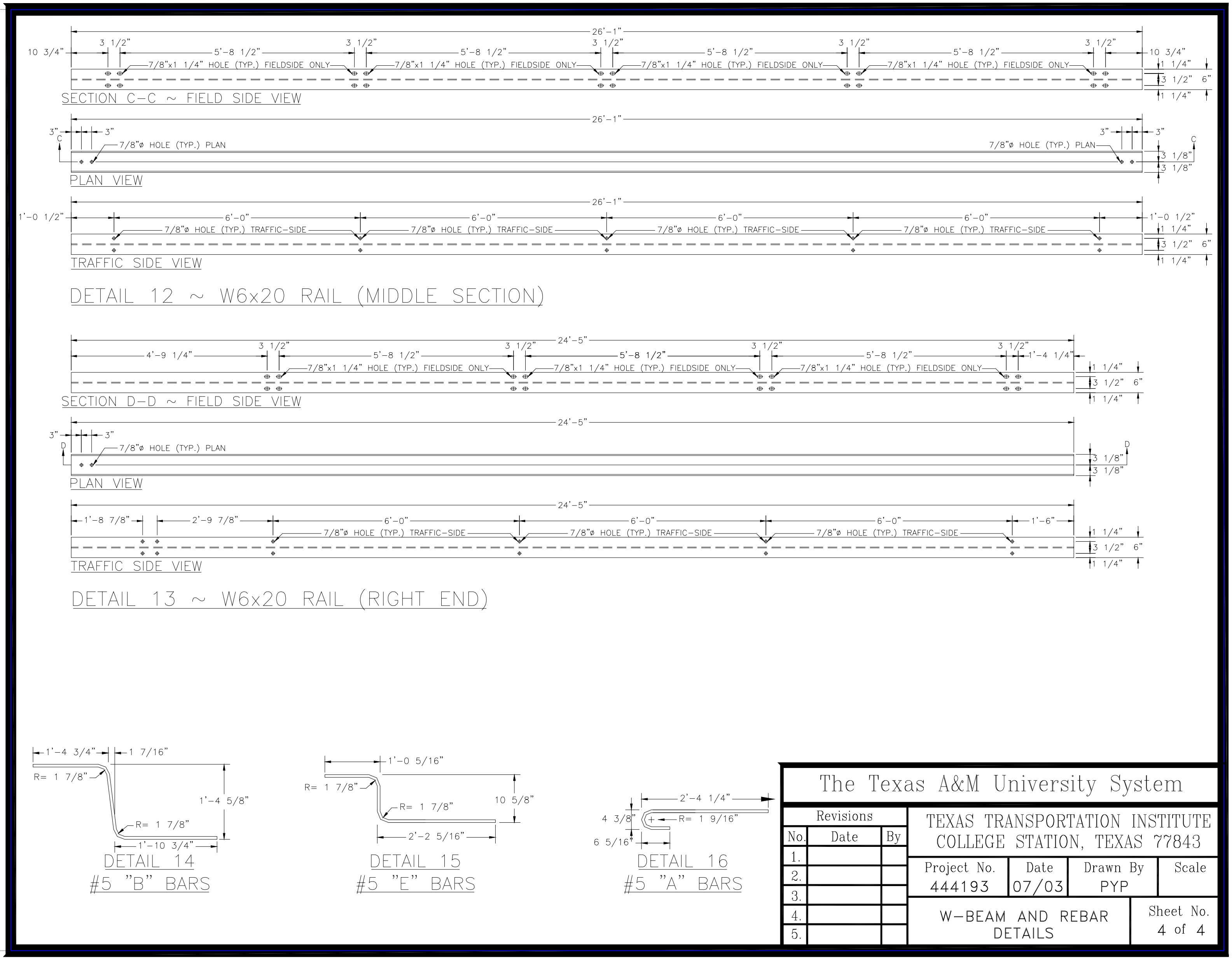


Figure 35. W-Beam and Rebar Details for U.S. 281 Truss Bridge Installation.

The W6 (W150) rail was attached to each post using four 3/4-inch (19 mm) diameter A325 bolts, 2-1/2 inches (64 mm) in length. The W6×20 (W150×30) rail was spliced together using TS5×2×5/16 (TS127×51×8) tube shapes fabricated from 5/16-inch (8 mm) thick bent plate with two 3/8-inch (10 mm) thick steel plates welded to the top and bottom of each tube splice. Two fabricated tube splices were used at each W6 (W150) rail splice location. The splices were 1 ft 4-1/2 inches (419 mm) in length and bolted to the rail using four 3/4-inch (19 mm) diameter A325 bolts, 7 inches (178 mm) in length. The C12×20.7 (C310×31) rail attached directly to the W6×20 (W150×30) rail elements and was spliced using two 2 inch (51 mm) × 1 inch (25 mm) × 3/8 inch (10 mm) thick splice plates that bolted on each side of the underlying W6×20 (W150×30) flange. Grade 8 "button head" bolts 3/4 inch (19 mm) diameter were used at the C12×20.7 (C310×31) splice locations. Each post was attached to the top of the curb using two 7/8-inch (22 mm) diameter Hilti Super HAS anchors, 13-1/2 inches (343 mm) long and embedded 10-1/2 inches (267 mm). These bolts were anchored using the Hilti HSE 2421 epoxy anchoring system. All structural steel used for this project was specified as A36 material. For an overall view of the test installation, please refer to Figures 32 through 35. Photographs of the completed installation are shown in Figure 36.

TEST NO. 444193-2 (*NCHRP REPORT 350* TEST NO. 3-11)

Test Vehicle

A 1999 Chevrolet Cheyenne 2500 pickup truck, shown in Figures 37 and 38, was used for the crash test. Test inertia weight of the vehicle was 4535 lb (2059 kg) and its gross static weight was 4535 lb (2059 kg). The height to the lower edge of the vehicle bumper was 16.3 inches (415 mm), and the height to the upper edge of the bumper 25.0 inches (635 mm). Figure 56 in Appendix D gives additional dimensions and information on the vehicle. The vehicle was directed into the installation using the cable reverse tow and guidance system and was released to be freewheeling and unrestrained just prior to impact.

Soil and Weather Conditions

The test was performed on the morning of July 30, 2003. Rainfall of 0.10 inches (3 mm) was recorded seven days prior to the test. Weather conditions at the time of testing were as follows: wind speed: 3 mi/h (4 km/h); wind direction: 0 degrees with respect to the vehicle (vehicle was traveling in a southwesterly direction); temperature: 91ºF (33ºC); and relative humidity: 54 percent.

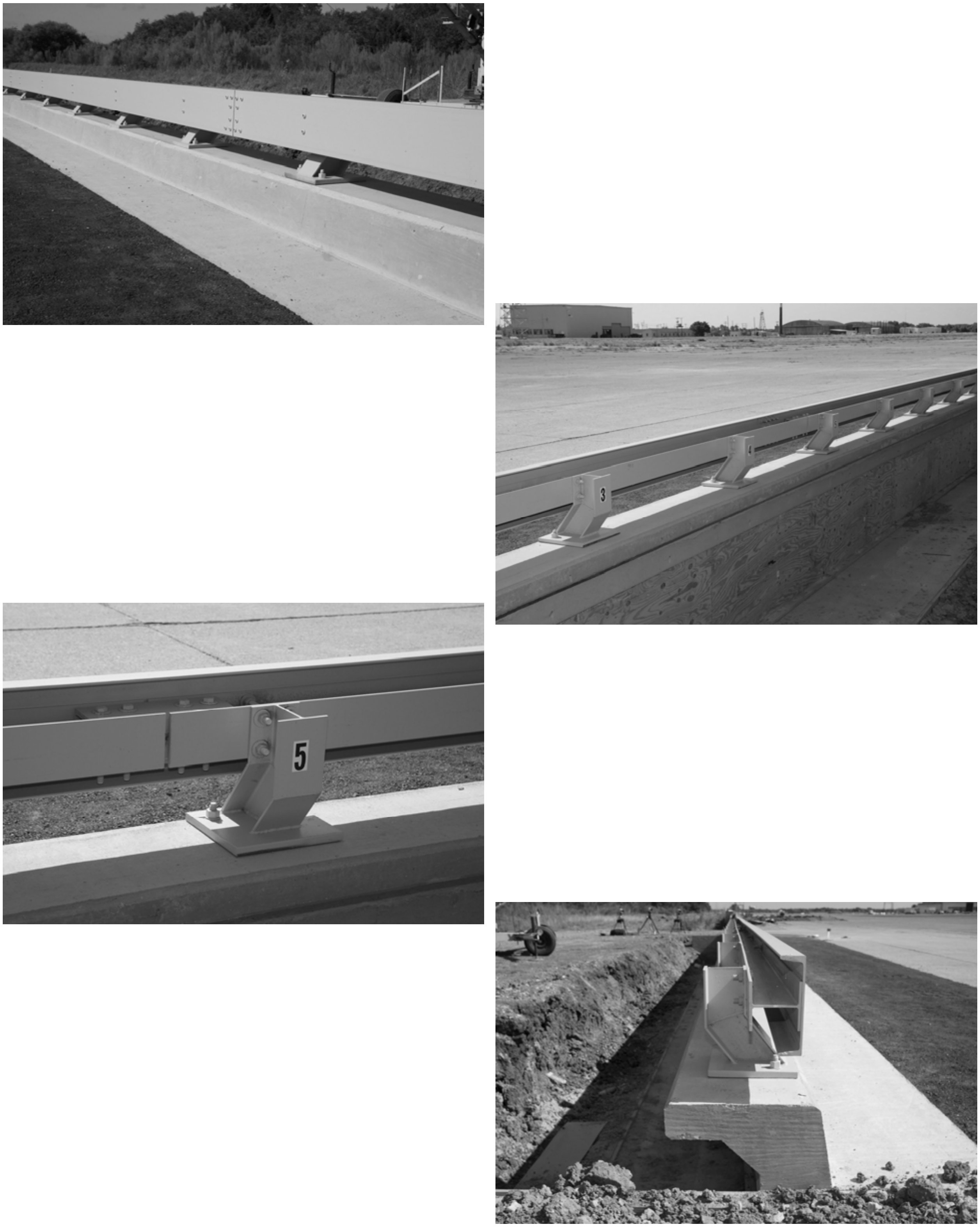
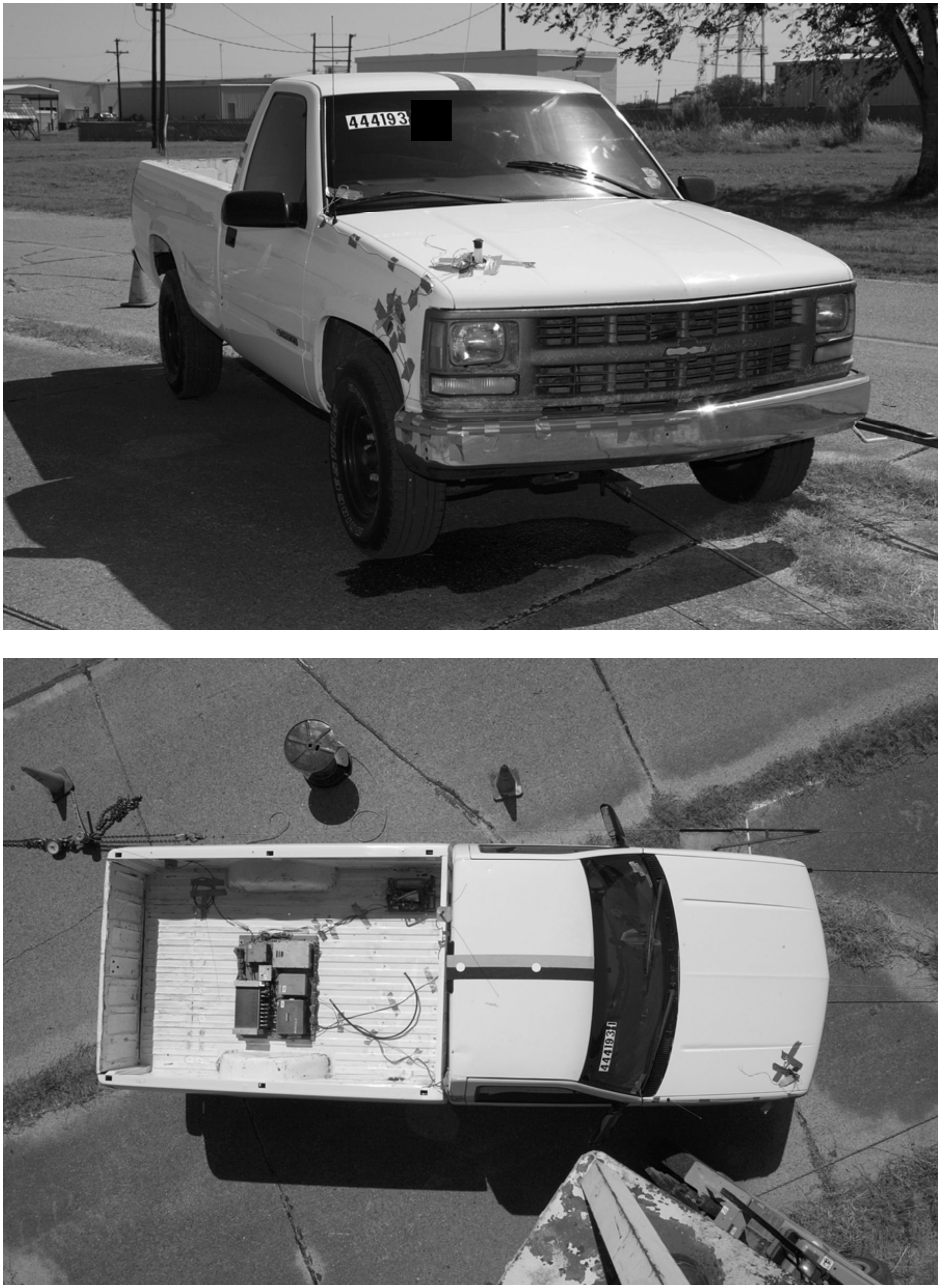


Figure 36. U.S. 281 Truss Bridge Installation before Test 444193-2.

[redacted]

Figure 37. Vehicle/Installation Geometrics for Test 444193-2.



[redacted]

Figure 38. Vehicle before Test 444193-2.

Test Description

The vehicle, traveling at 61.0 mi/h (98.2 km/h), impacted the U.S. 281 Truss Bridge Rail 0.42 m (1.37 ft) upstream of post 4 at an impact angle of 25.6 degrees. Shortly after impact, the right front tire contacted the curb; by 0.030 s, the right front tire began to ride up the curb; and at 0.035 s, the tire blew out. A fine crack on the rear of the deck at post 4 began to form at 0.050 s, and the vehicle began to redirect at 0.052 s. The crack in the bridge deck began to enlarge at 0.060 s. At 0.184 s, the vehicle was traveling parallel with the bridge rail at a speed of 51.4 mi/h (82.7 km/h). The rear of the vehicle contacted the rail element at 0.211 s. At 0.363 s, the vehicle lost contact with the bridge rail while traveling at a speed of 50.7 mi/h (81.6 km/h) and an exit angle of 9.7 degrees. At 0.411 s, the right front tire and wheel separated from the vehicle. Brakes on the vehicle were applied 2.1 s after impact; the vehicle subsequently yawed counterclockwise and came to rest adjacent to this barrier 225.2 ft (68.6 m) downstream of impact and 50.0 ft (15.2 m) forward of the traffic face of the bridge rail. Figures 59 and 60 in Appendix E show sequential photographs of the test period.

Damage to Test Installation

Damage to the Truss Bridge Rail is shown in Figures 39 and 40. There were tire marks on the face of the rail and the curb from 16.4 in (418 mm) upstream of post 4 and continuing for a distance of 14.9 ft (4.54 m), which was the length of contact of the vehicle with the bridge rail. Posts 4 and 5 were deflected toward the field side 0.4 inches (10 mm), and the concrete on the rear side of the deck was spalled. Maximum dynamic deflection during the test was 3.6 inches (91 mm).

Vehicle Damage

The pickup sustained moderate damage, as shown in Figure 41. Structural damage was imparted to the right upper and lower A-arms, right outer tie rod end, and floor pan, and the right front frame rail was deformed. Also damaged were the front bumper, hood, grill, right front quarter panel, right door, right rear exterior bed, right rear tire and wheel rim, and rear bumper. The inner rim of the right front wheel rim separated from the outer rim. Maximum exterior crush to the vehicle was 25.4 inches (645 mm) in the side plane at the right front corner at bumper height. Maximum occupant compartment deformation was 1.6 inches (42 mm) in the right firewall area. Photographs of the interior of the vehicle are shown in Figure 42. Tables 9 and 10 in Appendix D show exterior crush and occupant compartment deformations.

Figure 39. After Impact Trajectory Path for Test 444193-2.

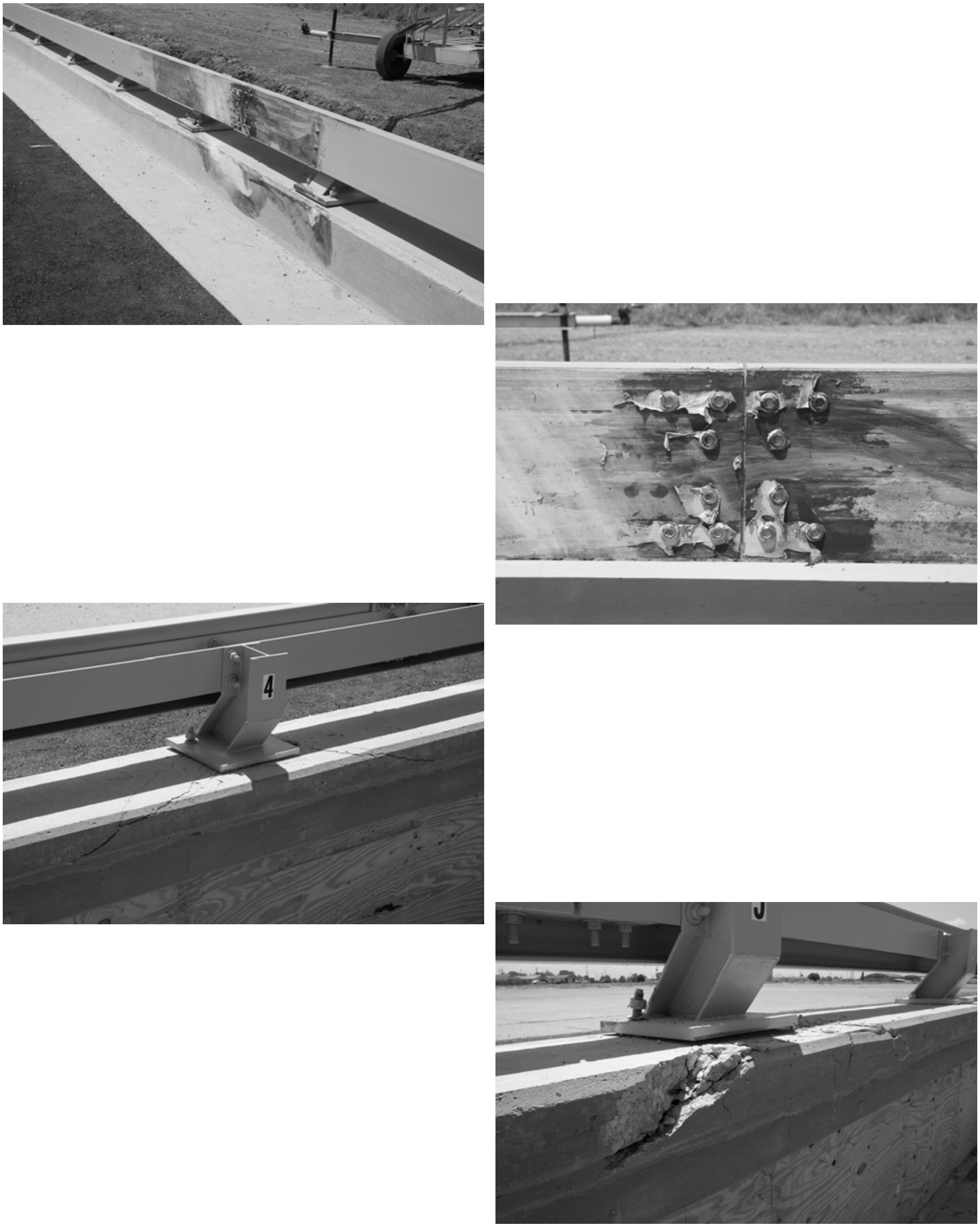


Figure 40. Installation after Test 444193-2.

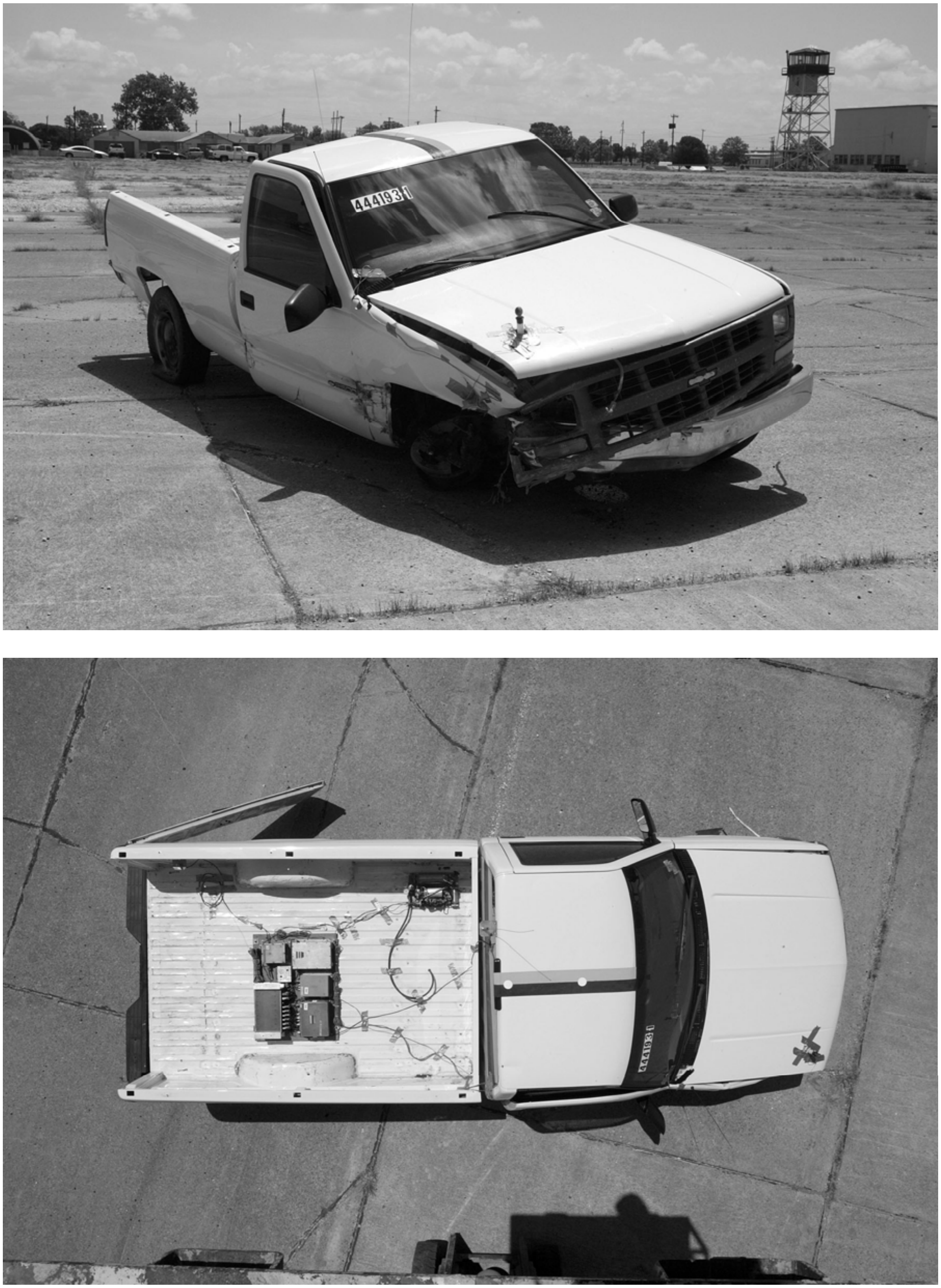


Figure 41. Vehicle after Test 444193-2.

Before Test

After Test

Figure 42. Interior of Vehicle for Test 444193-2.

Occupant Risk Factors

Data from the triaxial accelerometer, located at the vehicle's center of gravity, were digitized to compute occupant impact velocity and ridedown accelerations. Only the occupant impact velocity and ridedown accelerations in the longitudinal axis are required from these data for evaluation of criterion L of *NCHRP Report 350*. In the longitudinal direction, the occupant impact velocity was 17.4 ft/s (5.3 m/s) at 0.095 s, the highest 0.010-s occupant ridedown acceleration was -8.7 g's from 0.095 to 0.105 s, and the maximum 0.050-s average acceleration was -8.6 g's between 0.017 and 0.067 s. In the lateral direction, the occupant impact velocity was 22.6 ft/s (6.9 m/s) at 0.095 s, the highest 0.010-s occupant ridedown acceleration was -10.2 g's from 0.226 to 0.236 s, and the maximum 0.050-s average was -12.3 g's between 0.018 and 0.068 s. These data and other pertinent information from the test are summarized in Figure 43. Figures 68 through 74 in Appendix F present vehicle angular displacements and acceleration versus time traces.

ASSESSMENT OF TEST RESULTS

An assessment of the test based on the applicable *NCHRP Report 350* safety evaluation criteria is provided below.

Structural Adequacy

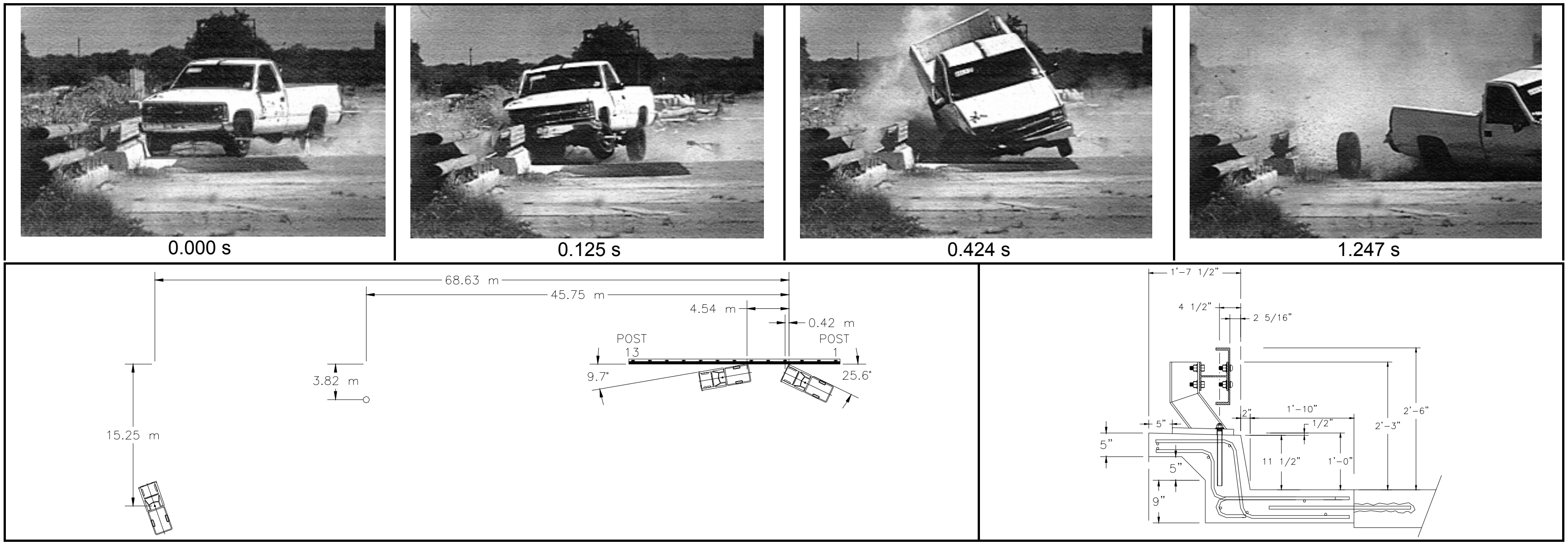
A. Test article should contain and redirect the vehicle; the vehicle should not penetrate, underride, or override the installation although controlled lateral deflection of the test article is acceptable.

Results: The Truss Bridge Rail contained and redirected the 2000P pickup truck. The 2000P pickup truck did not penetrate, underride, or override the installation. Maximum dynamic deflection during the test was 3.6 inches (91 mm). (PASS)

Occupant Risk

D. Detached elements, fragments, or other debris from the test article should not penetrate or show potential for penetrating the occupant compartment, or present an undue hazard to other traffic, pedestrians, or personnel in a work zone. Deformation of, or intrusions into, the occupant compartment that could cause serious injuries should not be permitted.

Results: No detached elements, fragments, or other debris was present to penetrate or to show potential for penetrating the occupant compartment or to present undue hazard to others in the area. Maximum occupant compartment deformation was 1.6 inches (42 mm) in the kick panel area near the passenger's feet, laterally across the cab. (PASS)


0.000 s 0.125 s 0.424 s 1.247 s

General Information	
Test Agency	Texas Transportation Institute
Test No.	444193-2
Date	07/30/03
Test Article	
Type	Bridge Rail Retrofit
Name	U.S. 281 Truss Bridge Rail Retrofit
Installation Length (ft)	75 (22.8 m)
Material or Key Elements	Steel Rail (W6x20 With C12x20.7) and Steel Post (W6x20) System
Soil Type and Condition	Concrete Footing, Dry
Test Vehicle	
Type	Production
Designation	2000P
Model	1999 Chevrolet Cheyenne 2500 Pickup
Weight (lb)	
Curb	4635 (2105 kg)
Test Inertial	4535 (2059 kg)
Dummy	N/A
Gross Static	4535 (2059 kg)

Impact Conditions	
Speed (mi/h)	61.0 (98.2 km/h)
Angle (deg)	25.6
Exit Conditions	
Speed (mi/h)	50.7 (81.6 km/h)
Angle (deg)	9.7
Occupant Risk Values	
Impact Velocity (ft/s)	
x-direction	17.4 (5.3 m/s)
y-direction	22.6 (6.9 m/s)
THIV (mi/h)	18.9 (30.5 km/h)
Ridedown Accelerations (g's)	
x-direction	-8.7
y-direction	-10.2
PHD (g's)	10.7
ASI	1.57
Max. 0.050-s Average (g's)	
x-direction	-8.6
y-direction	-12.3
z-direction	-3.5

Test Article Deflection (inches)	
Dynamic	0.4 (10 mm)
Permanent	3.6 (91 mm)
Working Width	22.6 (579 mm)
Vehicle Damage	
Exterior	
VDS	01FR2
CDC	01FREW2
Maximum Exterior Vehicle Crush (inches)	25.2 (645 mm)
Interior	
OCDI	RF0111000
Max. Occ. Compart. Deformation (inches)	1.6 (42 mm)
Post-Impact Behavior (during 1.0 s after impact)	
Max. Yaw Angle (deg)	-57.1
Max. Pitch Angle (deg)	-21.6
Max. Roll Angle (deg)	27.8

Figure 43. Summary of Results for Test 444193-2, *NCHRP Report 350* Test 3-11.

F. The vehicle should remain upright during and after collision although moderate roll, pitching, and yawing are acceptable.

Results: The vehicle remained upright during and after the collision event. (PASS)

Vehicle Trajectory

K. After collision, it is preferable that the vehicle's trajectory not intrude into adjacent traffic lanes.

Results: The vehicle came to rest upright 225.2 ft (68.6 m) downstream of impact and 50.0 ft (15.2 m) forward of the traffic face of the rail. (FAIL)

L. The occupant impact velocity in the longitudinal direction should not exceed 12 m/s and the occupant ridedown acceleration in the longitudinal direction should not exceed 20 g's.

Results: Longitudinal occupant impact velocity was 17.4 ft/s (5.3 m/s), and longitudinal ridedown acceleration was -8.7 g's. (PASS)

M. The exit angle from the test article preferably should be less than 60 percent of the test impact angle, measured at time of vehicle loss of contact with the test device.

Results: Exit angle at loss of contact was 9.7 degrees, which was 38 percent of the impact angle. (PASS)

The following supplemental evaluation factors and terminology, as presented in the FHWA memo entitled "Action: Identifying Acceptable Highway Safety Features," were used for visual assessment of test results. Factors underlined below pertain to the results of the crash test reported herein.

Passenger Compartment Intrusion

1. Windshield Intrusion
    a. No windshield contact
    b. Windshield contact, no damage
    c. Windshield contact, no intrusion
    d. Device embedded in windshield, no significant intrusion
    e. Complete intrusion into passenger compartment
    f. Partial intrusion into passenger compartment
2. Body Panel Intrusion *yes* *or* *no*

Loss of Vehicle Control

1. Physical loss of control
2. Loss of windshield visibility
3. Perceived threat to other vehicles
4. Debris on pavement

Physical Threat to Workers or Other Vehicles

1. *Harmful debris that could injure workers or others in the area*
2. *Harmful debris that could injure occupants in other vehicles*

No debris was present.

Vehicle and Device Condition

1. *Vehicle Damage*
    - *a. None*
    - *b. Minor scrapes, scratches or dents*
    - *c. Significant cosmetic dents*
    - *d. Major dents to grill and body panels*
    - [✓ *e. Major structural damage*]
2. *Windshield Damage*
    - [✓ *a. None*]
    - *b. Minor chip or crack*
    - *c. Broken, no interference with visibility*
    - *d. Broken or shattered, visibility restricted but remained intact*
    - *e. Shattered, remained intact but partially dislodged*
    - *f. Large portion removed*
    - *g. Completely removed*
3. *Device Damage*
    - *a. None*
    - *b. Superficial*
    - [✓ *c. Substantial, but can be straightened*]
    - *d. Substantial, replacement parts needed for repair*
    - *e. Cannot be repaired*

CHAPTER 5. DESIGN EXCEPTIONS

BACKGROUND

Many existing historic through-truss bridges are located on highways with posted speed limits greater than 45 mi/h (72 km/h), and a TL-3 bridge rail would be indicated. These bridges typically have narrow roadways (<28 ft [<8.5 m]), and many have narrow curbs that would not accommodate the curb-mounted TL-3 rail retrofit described in the previous chapter. TxDOT seeks methods for evaluating the response of existing truss bridge members to impact forces resulting from mounting a TL-3 retrofit rail directly to an existing historic truss. This chapter evaluates the magnitude of design impact forces appropriate for evaluating the response of typical existing historic truss bridges.

Crash testing procedures for evaluating performance of bridge rails and other highway safety structures are based on the assumption that the errant vehicle is tracking straight ahead with no side-slip and no yaw velocity. Recommended test conditions include the vehicle type/mass, speed, and approach angle. Lateral placement of the vehicle with respect to the device being tested is also included for guardrail terminals, sign supports, and other similar devices. For tests wherein a bridge rail is expected to contain and redirect the colliding vehicle, the vehicle type/mass, speed, and approach angle are the parameters addressed. TL-3 of *NCHRP Report 350* includes two redirection tests for a bridge rail. They are:

> ***NCHRP Report 350* test designation 3-10:** An 1806-lb (820 kg) passenger car impacts the CIP in the LON of the longitudinal barrier at a nominal speed and angle of 62 mi/h (100 km/h) and 20 degrees. The purpose of this test is to evaluate the overall performance of the LON section in general and occupant risks in particular.
>
> ***NCHRP Report 350* test designation 3-11:** A 4405-lb (2000 kg) pickup truck impacts the CIP in the LON of the longitudinal barrier at a nominal speed and angle of 62 mi/h (100 km/h) and 25 degrees. The test is intended to evaluate the strength of the section for containing and redirecting the pickup truck.

TL-3 is generally used for high-speed highways (posted speed limit above 45 mi/h [72 km/h]) regardless of the width or other characteristics of the highway.

NCHRP *Report 350* also includes recommended test conditions for TL-2. They are:

> ***NCHRP Report 350* test designation 2-10:** An 1806-lb (820 kg) passenger car impacts the CIP in the LON of the longitudinal barrier at a nominal speed and angle of 43.5 mi/h (70 km/h) and 20 degrees. The purpose of this test is to evaluate the overall performance of the LON section in general and occupant risks in particular.

***NCHRP Report 350* test designation 2-11:** A 4405-lb (2000 kg) pickup truck impacts the CIP in the LON of the longitudinal barrier at a nominal speed and angle of 43.5 mi/h (70 km/h) and 25 degrees. The test is intended to evaluate the strength of the section for containing and redirecting the pickup truck.

AASHTO LRFD Bridge Design Specifications indicate that the design force for a TL-3 bridge rail is 54 kips (240 kN) applied at 24 inches (610 mm) above the deck as a line load 4.0 ft (1.2 m) in length. For a TL-2 rail, the magnitude of force is lower, 27 kips (120 kN).

The magnitude of force imposed on a bridge rail is a function of the collision conditions (i.e., vehicle type/mass, speed, and approach angle). Reducing the magnitude of any of these parameters would reduce the severity of the collision and the magnitude of the force applied to the bridge rail.

ANALYSIS

Vehicle Type/Mass

Limiting the vehicle type/mass does not seem to be an effective and feasible approach to limiting forces imposed on a bridge rail. Limits would need to exclude pickup trucks and large automobiles, which would render the bridge virtually non-functional.

Vehicle Speed

The posted speed limit at a truss bridge could simply be reduced to 45 mi/h (72 km/h) or some value significantly below 62 mi/h (100 km/h). However, such a reduced speed limit would not be effective in reducing the actual travel speed without implementation of extensive enforcement, which is deemed to be cost prohibitive.

Approach Angle

Roadway widths for through truss bridges are generally less than 28 ft (8.5 m), and many are less than 24 ft (7.3 m). This narrow width physically limits the approach angle that a specified vehicle can achieve when traveling at a speed of 62 mi/h (100 km/h).

An analysis of maximum possible approach angles than can be achieved for various speeds and roadway widths with a friction value of 0.7 was performed using a point-mass model (see Figure 44). For the analysis, the vehicle is represented as a point traveling in a lane at the specified speed. A lateral friction coefficient of 0.7 was used. The vehicle is assumed to follow a circular path having a radius of curvature that will result in maximum side force. The vehicle is assumed to travel across the unoccupied adjacent lane and impact the bridge rail at the maximum possible angle. The value of that angle (θ) was calculated for various combinations of speed and roadway width.

V = velocity
M = vehicle mass
θ = impact angle

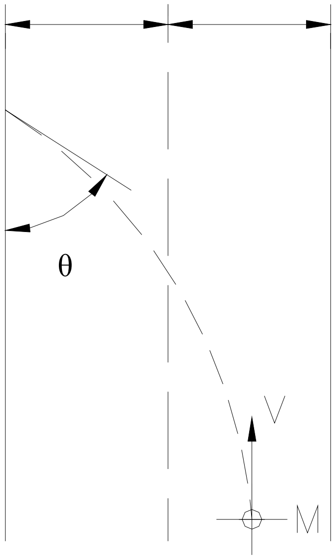


Figure 44. Geometrics for Point-Mass Model.

Results of the analysis are presented in Figure 45. The analysis indicates that for a roadway width of 28 ft (8.5 m) and a speed of 62 mi/h (100 km/h), the maximum approach angle is about 19 degrees. For a roadway width of 24 ft (7.3 m), the maximum approach angle is about 18 degrees.

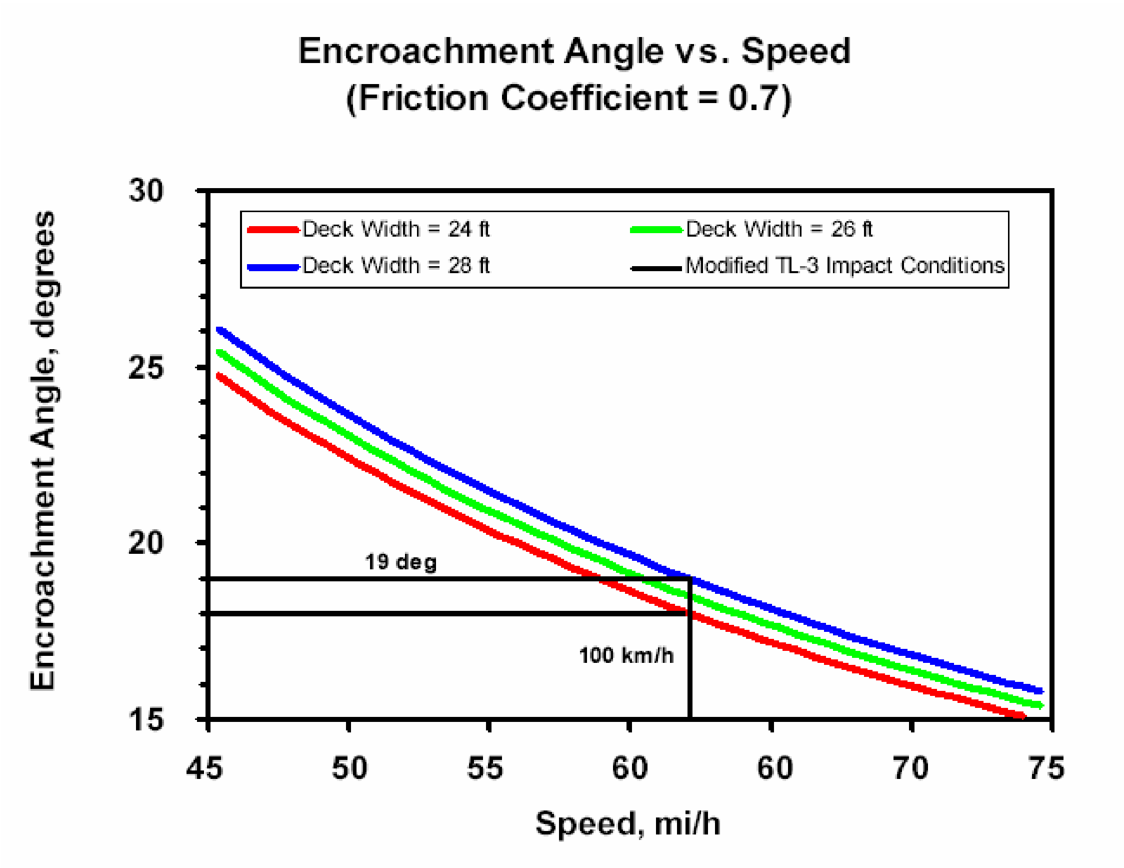


Figure 45. Encroachment Angle versus Speed.

This analysis leads to the conclusion that test conditions (and design loads) of reduced severity would be appropriate for truss bridge rails on structures 28 ft (8.5 m) or less in width. The data indicate that the test conditions of 62 mi/h (100 km/h) and 20 degrees for a 4405-lb (2000 kg) pickup is appropriate. For those test conditions, the transverse force applied to the bridge rail is less than 54 kips (240 kN) and can be quantified as follows.

A procedure for computing average lateral (transverse) force imposed on a longitudinal barrier as presented in *NCHRP Report 86* is as follows *(3)*.

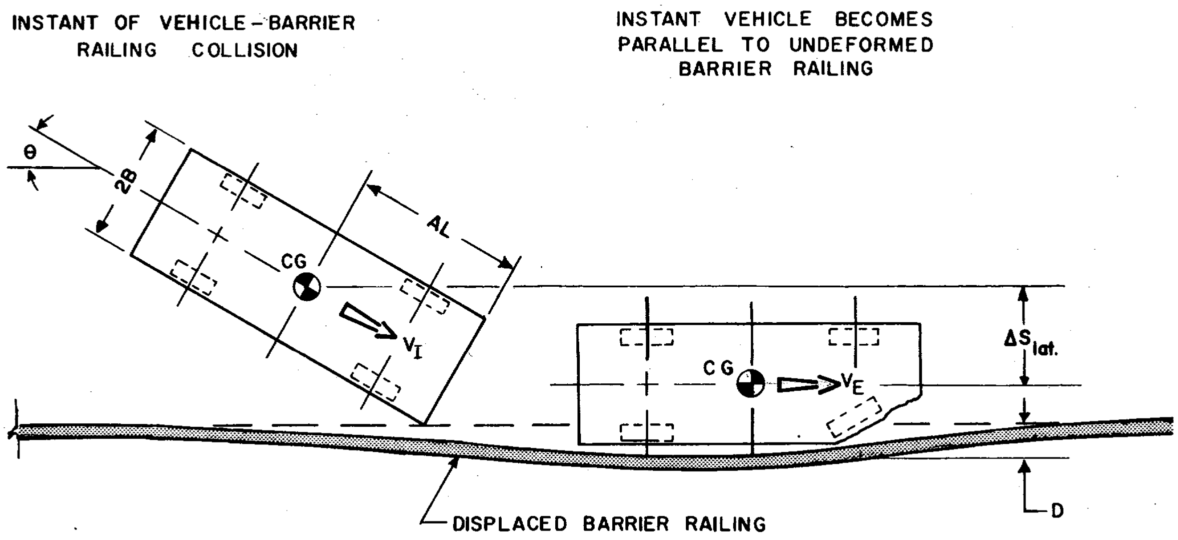


Figure 46. Mathematical Model of Vehicle-Barrier Railing Collision.

$$G_{lat} = \frac{V_I^2 \sin^2(\theta)}{2g\{AL\sin(\theta) - B[1-\cos(\theta)] + D\}}$$

$$F_{lat} = W(G_{lat})$$

in which:

L = Vehicle length (ft)
$2B$ = Vehicle width (ft)
CG = Center of gravity
D = Lateral displacement of barrier railing (ft)
ΔS = Movement of the vehicle (ft)
AL = Distance from vehicle's front end to center of mass (ft)
V_E = Vehicle exit velocity (ft/s)
V_I = Vehicle impact velocity (ft/s)
θ = Vehicle impact angle (deg)
g = Acceleration due to gravity (ft/s^2)
W = Vehicle weight (lb)

If it is assumed that the magnitude of force versus time has the shape of a sine wave, the maximum force applied to the bridge rail is:

$$F_{max} = \frac{\pi}{2} F_{avg}$$

The equations above can be combined to yield:

$$F_{max} = \frac{\pi W V_I^2 \sin^2(\theta)}{4g\{AL\sin(\theta) - B[1-\cos(\theta)] + D\}}$$

W = 4405 lb (2000 kg)
V_I = 62 mi/h (90.9 ft/s) [100 km/h (27.8 m/s)]
θ = 25 degrees
A = 0.409
L = 215 in = 17.9 ft (5461 mm = 5.46 m)
B = 74 in = 7.17 ft (1880 mm = 1.88 m)
D = 0
g = 32.2 ft/s^2 (9.8 m/s^2)

The maximum force, F_{max}, is computed to be:

$$F_{max} = 63 \text{ kips (280 kN)}$$

The design value of 54 kips (240 kN) in *AASHTO LRFD Bridge Design Specifications* is an average over 0.050 s and is based on measured forces from full-scale crash tests. If the approach angle is reduced to 20 degrees, the computed maximum force is:

$$F_{max} = 48.7 \text{ kips (217 kN)}$$

Thus, one could reduce the design load to 42 kips (187 kN).

$$\frac{48.7}{63} \text{ (54 kips) = 42 kips (187 kN)}$$

CONCLUSION

The relatively narrow roadway widths of some truss bridges located on high-speed highways justify modification of the design crash test conditions and design loads for bridge rails for those structures. The analysis presented herein shows that the approach angle for TL-3 conditions can be reduced from 25 degrees to 20 degrees for a roadway width of 28 ft (8.5 m) or less. The resulting transverse force imposed on the rail can be reduced from 54 kips (240 kN) to 42 kips (187 kN).

CHAPTER 6. TRAFFIC RAIL FOR NEW TRUSS BRIDGES

BACKGROUND

TxDOT plans several new truss bridges throughout the state. Currently, the bridge railing proposed for these structures consists of a standard TxDOT railing, the T101, which is supported by a cast-in-place concrete deck. TxDOT would prefer to have the option to support a bridge rail system from the truss members in lieu of supporting the railing from the concrete deck. The primary advantage of using a truss-supported bridge rail is to allow alternate types of deck. One disadvantage to using a truss-supported bridge rail is that the bridge structure must be adequately designed to resist the crash loads imparted from the bridge rail directly to the truss members. A truss-mounted bridge railing system will provide the bridge designer with more options and greater flexibility in designing steel truss bridges.

The purpose of this task was to develop a truss-mounted bridge railing design that meets the strength requirements of *NCHRP Report 350* TL-3. The railing system should minimize the force imparted to supporting truss members and be acceptable for varying span lengths up to 20 ft (6.1 m) between supporting truss members. Forces imposed on the truss members from the railing system for TL-3 conditions were quantified.

DEVELOPMENT OF THE DESIGN

On February 23, 2004, TTI and TxDOT personnel met to discuss and establish requirements and guidelines for the design of a truss-mounted bridge rail for new truss bridges. The typical new truss is assumed to be a Warren-type or Pratt-type pony truss with vertical truss web members at each panel point. The new bridge rail design should meet the requirements of *NCHRP Report 350* TL-3 and be supported by vertical truss web members and end posts only. The loading conditions for TL-3 consist of a 54-kip (240 kN) force distributed over 4 ft (1.2 m) along the railing system. For a two-rail bridge rail system, this 54-kip (240 kN) force is divided evenly for each rail element, or 27-kip (120 kN) force distributed over 4 ft (1.2 m) per rail element. The new design should also incorporate the use of crushable blockouts that limit concentrated forces applied to supporting truss members. Magnitude of the reactions applied to the truss members from the crushable blockouts were to be defined and will be used by the bridge designer to design the bridge truss members. The new design should be suitable for attachment to vertical truss members spaced up to 20 ft (6.1 m).

For this project, finite element modeling was performed on several sizes of crushable pipe blockouts using the computer modeling program LS-DYNA. The blocks were loaded with diametrically opposing plate loads. The crushable pipe blockouts analyzed for this project ranged in size from 6-inch (152 mm) diameter Schedule 40 pipe to 10-inch (254 mm) diameter Schedule 80 pipe. Seven different crushable pipe blockouts were analyzed. Five of the seven blockouts were 6 inches (152 mm) in length and the remaining two were 8 inches (203 mm) in length. A summary of the force versus crush distance for each pipe blockout type is shown in the calculations in Appendix H.

Structural analyses of several different rails using the results obtained from the crushable pipe blockouts were performed using STAAD Pro. TL-3 conditions require that the bridge rail system resist 54 kips (240 kN) of transverse load distributed over a 4-ft (1.2 m) longitudinal distance. For the two-rail system considered, the load was divided equally between the two rail elements, i.e., 27 kips (120 kN) applied to each rail element. Analyses were performed on several different combinations of rail sizes and crushable pipe blockout types using five continuous spans with span lengths ranging from 10 ft to 20 ft (3.0 m to 6.1 m). The crushable pipe blockouts were modeled as multi-linear springs with spring constants, "k" (force/crush), used to approximate the graphs shown on page seven of the calculations in Appendix H. Analyses were performed on each rail/crushable pipe combination with the 27 kips (120 kN) distributed over 4 ft (1.2 m) located at:

- mid-span,
- centered over a crushable pipe support (vertical truss member support), and
- at the end of the rail element.

A summary of the data obtained from the analyses on the different rail/crushable pipe blockout combinations is presented in the calculations in Appendix H.

SUMMARY OF THE DESIGN

A new bridge rail design was selected based on the results from the analyses. This new bridge rail design consists of two railing members fabricated from HSS8×8×6 (HSS203×203×152) tubular members. The recommended height of the top and bottom rail members is 30 inches (762 mm) and 16 inches (406 mm), respectively. Researchers recommend 10-inch (254 mm) diameter Schedule 80 (extra strong) A53, grade B pipe blockouts, 6 inches (152 mm) in length be used to support the rail at all vertical truss member locations. Considering the height and geometry of the rail elements, there is a low potential of vehicular interaction with the truss members based on Figures A13.1.1-2 and A13.1.1-3 in Section 13 of the *AASHTO LRFD Bridge Design Specifications*. Details of the recommended design are shown as Figures 47 and 48. A graph of the force versus crush displacement of the selected 10-inch (254 mm) Schedule 80 pipe blockout is shown as Figure 49.

The new bridge rail design developed from this project meets the strength requirements of *NCHRP Report 350*, TL-3. This railing is designed for mounting directly to Pratt-type or Warren-type trusses that have vertical truss members spaced 20 ft (6.1 m) or less and rigidly connected to the transverse floorbeams. A minimum clear space of 3 inches (76 mm) is recommended between the railing and any diagonal truss members that do not support the rail. The railing is designed for installation by bolted connection to vertical members. The railing will meet *NCHRP Report 350* TL-3 requirements provided that:

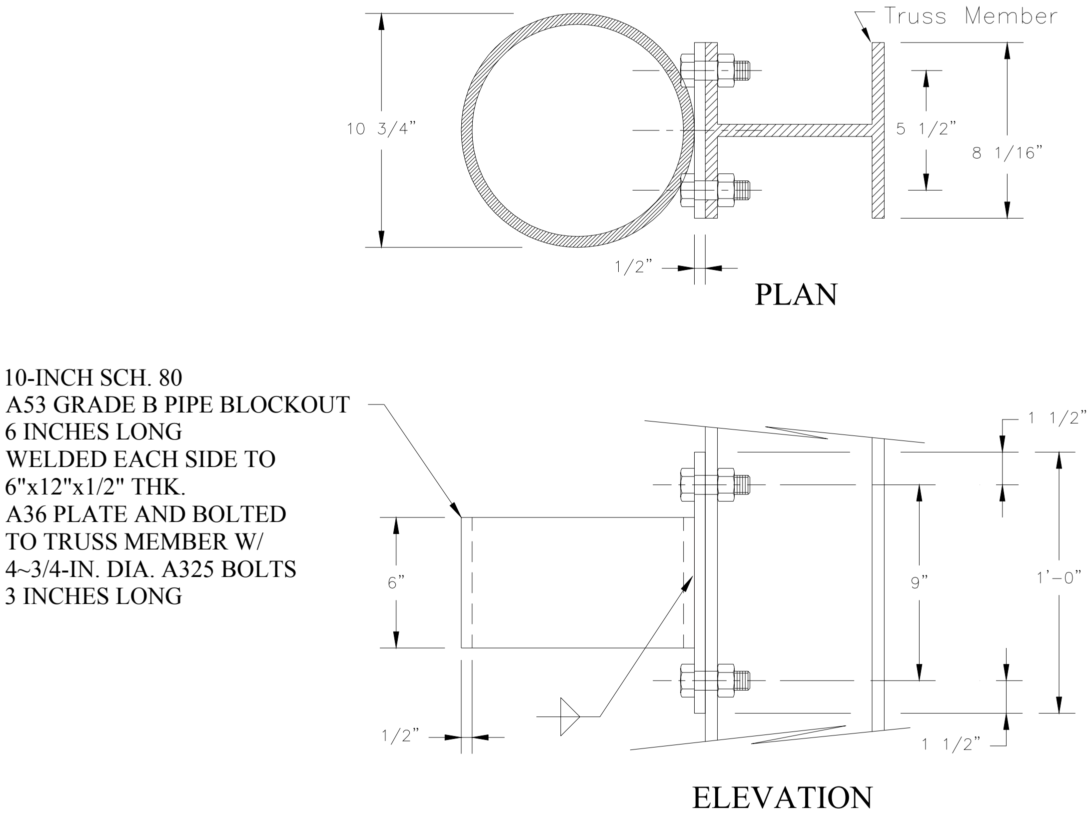
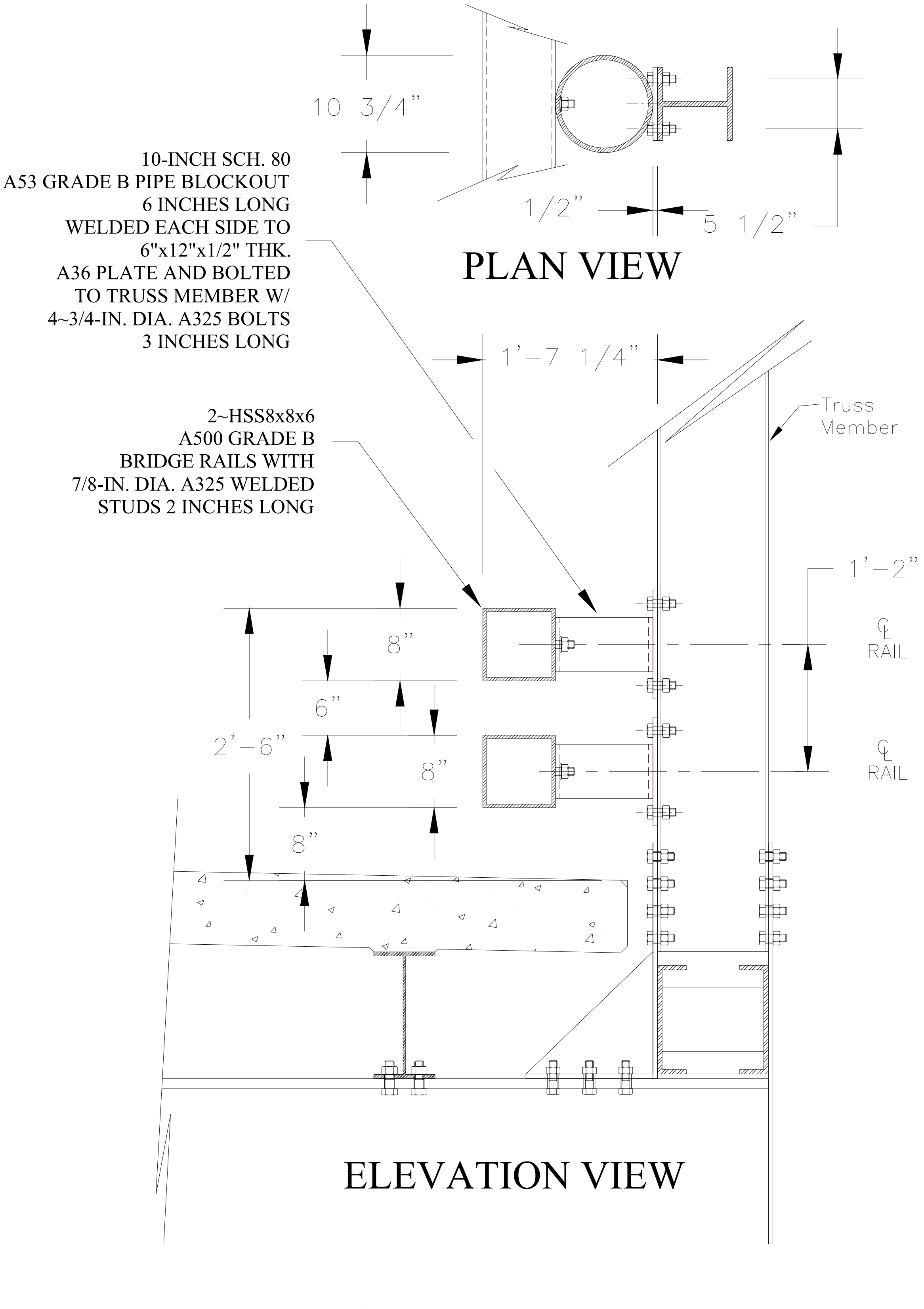


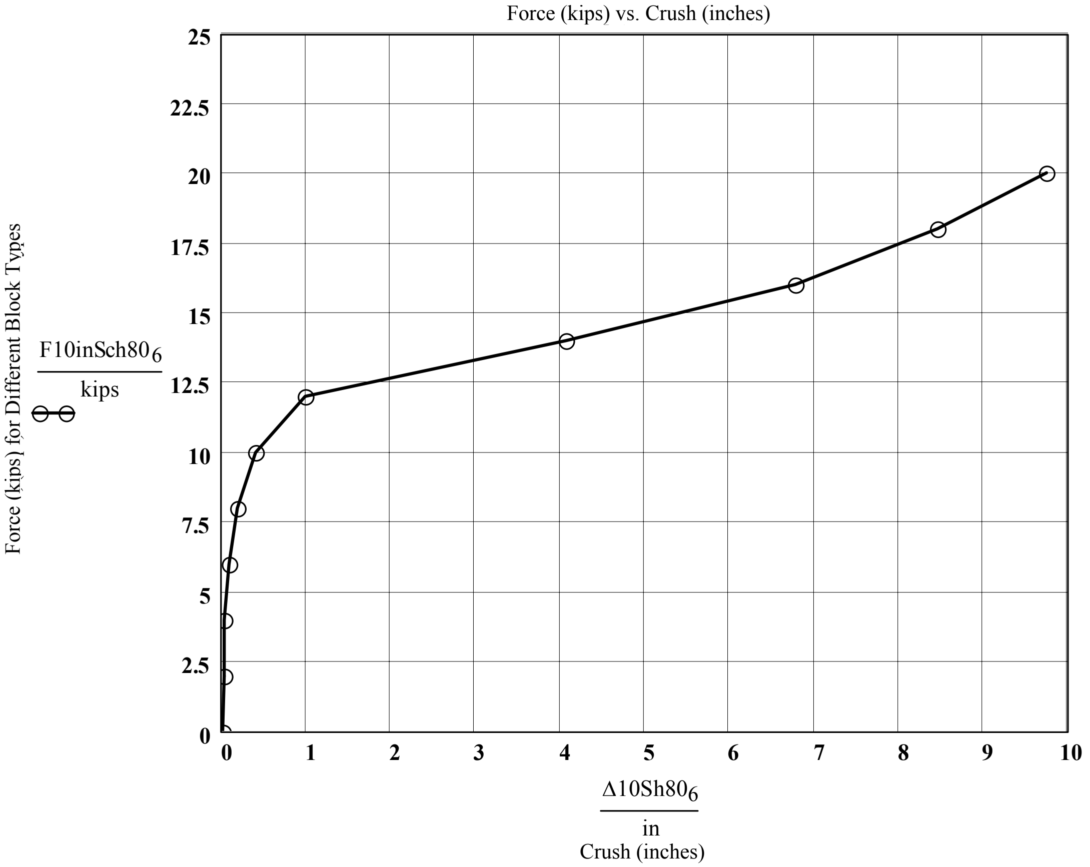
Figure 47. Details of the Recommended Crushable Pipe Blockout.



10-INCH SCH. 80
A53 GRADE B PIPE BLOCKOUT
6 INCHES LONG
WELDED EACH SIDE TO
6"x12"x1/2" THK.
A36 PLATE AND BOLTED
TO TRUSS MEMBER W/
4~3/4-IN. DIA. A325 BOLTS
3 INCHES LONG

2~HSS8x8x6
A500 GRADE B
BRIDGE RAILS WITH
7/8-IN. DIA. A325 WELDED
STUDS 2 INCHES LONG

Figure 48. Details of Recommended New Bridge Rail Design.



$F10inSch80_6$ = 10-inch Diameter Schedule 80, A53 Grade B Pipe, 6 inches in Length

Figure 49. Plot of Force (kips) versus Crush Distance (inches) for 10-inch (254 mm) Schedule 80, A53 Grade B Pipe Blockout, 6 inches (152 mm) in Length.

1. the spacing between vertical members does not exceed 20 ft (6.1 m), and

2. the truss members and all associated components are designed for the theoretical crash loads transmitted to the truss through the rail plus all dead load including the rail weight.

The following tables provide recommended crash loads to be used in the design of the bridge structure. Table 1 refers to crash loads applied to intermediate truss members (see Figure 50). Table 2 refers to the situation where crash loads are applied to the end of the bridge railing system connected to the end truss members. These loads are applicable where the bridge railing system does not extend beyond the end of the truss (see Figure 51). The loads presented in these tables should be used to analyze a 3-D model of the truss bridge and connections in conjunction with the dead load of the structure. The bridge designer should consider the application of these loads at the various locations along the truss to produce the highest stress in the truss members. The designer should confirm that the capacities of the members exceed the maximum member force due to the loading. For additional information, please refer to the calculations included in the Appendix H.

TxDOT anticipates that most new truss construction will be of the pre-fabricated, fabricator-designed type. Implementation of the new rail system with this type of truss would require that the fabricator/designer could demonstrate that the truss has been designed for the crash rail impact load case.

Table 1. Recommended Lateral Design Loads for Intermediate Steel Truss Members.

Bridge Rail Type: 2~HSS8×8×6 Rails with 10-inch Schedule 80 A53 Pipe Blockouts, 6 inches Long		
Support Spacing (ft)	Lateral Design Force Per Rail Element Load at Support* (Intermediate Truss Members) (Force F1, kips)	Lateral Design Force Per Rail Element Load at Adjacent Supports (X2) (Intermediate Truss Members)* (Force F2, kips)
10	12.5	9.0
12	13.0	9.0
14	13.5	9.0
16	14.0	9.0
18	14.5	8.5
20	15.5	8.5

* Load applied to Upper and Lower Rail

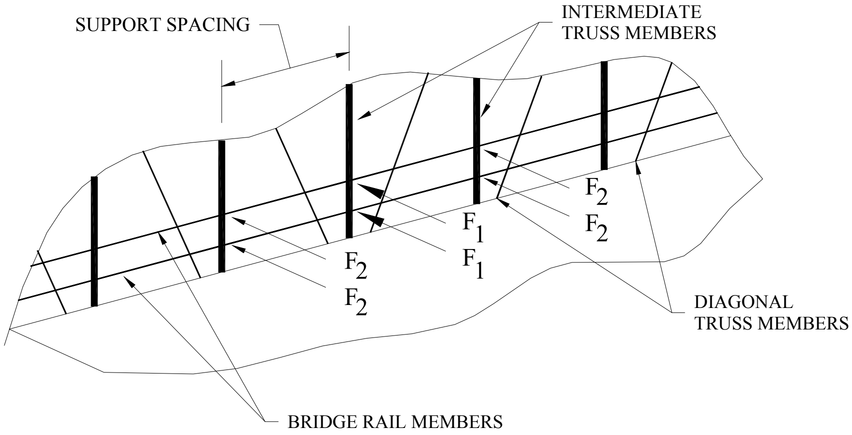


Figure 50. Crash Loads at Intermediate Truss Members.

Table 2. Recommended Lateral Design Loads at End Steel Truss Member and Adjacent Member.

(Loads Based on Railing Terminating at End Truss Member) Bridge Rail Type: 2~HSS8×8×6 Rails with 10-inch Schedule 80 A53 Pipe Blockouts, 6 inches Long		
Support Spacing (ft)	Lateral Design Force Per Rail Element Load at End Support* (Force F3, kips)	Lateral Design Force Per Rail Element Load at Adjacent Support* (Force F4, kips)
10	16.5	13.0
12	17.5	13.0
14	18.5	13.0
16	19.0	12.5
18	20.0	12.0
20	21.0	10.0

* Load applied to Upper and Lower Rail

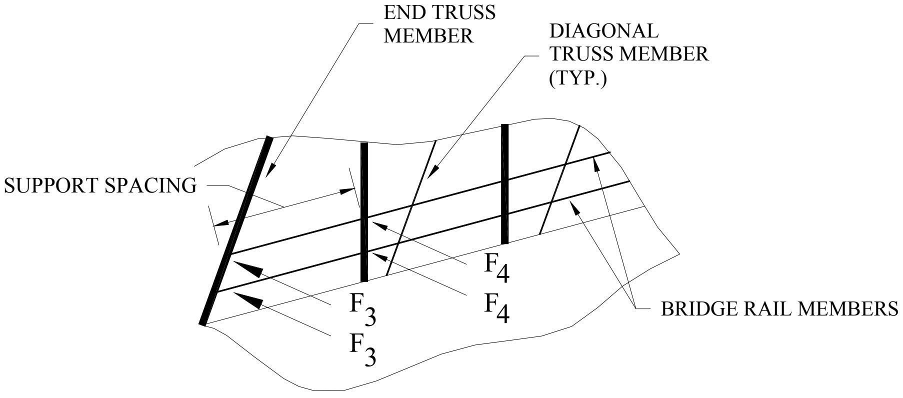


Figure 51. Crash Loads at End Truss Members.

EVALUATION OF NEW RAILING DESIGN FOR DEER CREEK TRUSS BRIDGE

On July 1, 2003, TTI personnel received from TxDOT a set of fabrication drawings entitled "98' Truss Bridge, 28' Roadway Width, Deer Creek Bridge, Dewitt County, Texas" and dated March 7, 2002. The Deer Creek Bridge is typical of new truss bridges used by TxDOT that are prefabricated and designed by the fabricator. These drawings present details for a 98-ft (29.9 m) long Warren Type Steel Pony Truss Bridge with verticals at panel points. The total height of the steel trusses is 10 ft (3.0 m) from the center of the bottom chords to the center of the top chords. These drawings have been approved for construction. This bridge will be constructed using a TxDOT Type T101 bridge rail supported by an 8-inch (203 mm) thick concrete deck. TxDOT proposes to use several bridge structures of this type in the future for new bridge construction. As part of this project, TTI has performed preliminary analyses to determine if the Deer Creek structure as designed is adequate to support crash loads from the railing design proposed for new truss bridges in the study reported herein.

Details of 98-ft (29.9 m) Deer Creek Truss Bridge

The current 98-ft (29.9 m) long Deer Creek Steel Truss Bridge in Dewitt County, Texas, consists of two Warren-Type Steel Pony Trusses with vertical and suspended floor beams. The bridge trusses consist of seven panels, with each panel 14 ft (4.3 m) in length. The center-to-center height between the top and bottom chords is 10 ft (3.0 m). The width of the bridge between the pony trusses is 31 ft 8 inches (9.65 m). W27×129 (W690×192) floor beams suspended below the bottom chord are supported at the panel points and are used to support five equally spaced W14×34 (W360×51) stringers. These stringers are used to support an 8-inch (203 mm) thick concrete deck with a 2 percent cross-slope. The concrete deck is 30 ft 3 inches (9.2 m) wide and is used to support a TxDOT Type T101 bridge rail on each side of the concrete deck. The clear roadway width between the railings is 28 ft 0 inch (8.5 m). The steel trusses consist of W12×26 (W310×39) diagonals and verticals. The bottom chords of the trusses consist of two C12×30 (C310×45) structural shapes in the exterior panels and two MC12×40 (MC31×60) structural shapes in the center panel. The top chords in the trusses range in size from a W12×50 (W310×74) on the ends to a W12×87 (W310×129) in the center of the trusses. Steel rods, 1 inch (25 mm) in diameter, are used as lateral cross bracing between the suspended floor beams. All superstructure steel is designated as American Society for Testing and Materials (ASTM) A709, grade 50W (A588 weathering type) steel.

Analyses of 98-ft (29.9 m) Deer Creek Truss Bridge

Analyses of the current bridge design were performed using the three-dimensional structural engineering program RISA-3D. The loads used in the analysis consisted of the dead load weight of the structure plus the impact rail loads developed for this project for a truss-mounted rail system. The design dead loads used in the analysis consist of the self-weight of the steel members and the dead load of the 8-inch (203 mm) thick slab with the stay-in-place forms. The distributed force of the slab and the pan forms total 135 lb-force/ft^2 (psf) (931 kPa). The impact loads used in the analysis consist of the loads developed for the design of the new truss-mounted bridge rail supported by vertical truss members spaced 14 ft (4.3 m) apart which were developed for this project. These loads consist of 13.5 kips (60 kN) located at a vertical support with 9.0 kips (40 kN) on the adjacent vertical truss members per rail element. A brief sketch of the imposed crash loads from the new truss-mounted rail is shown in Figure 52.

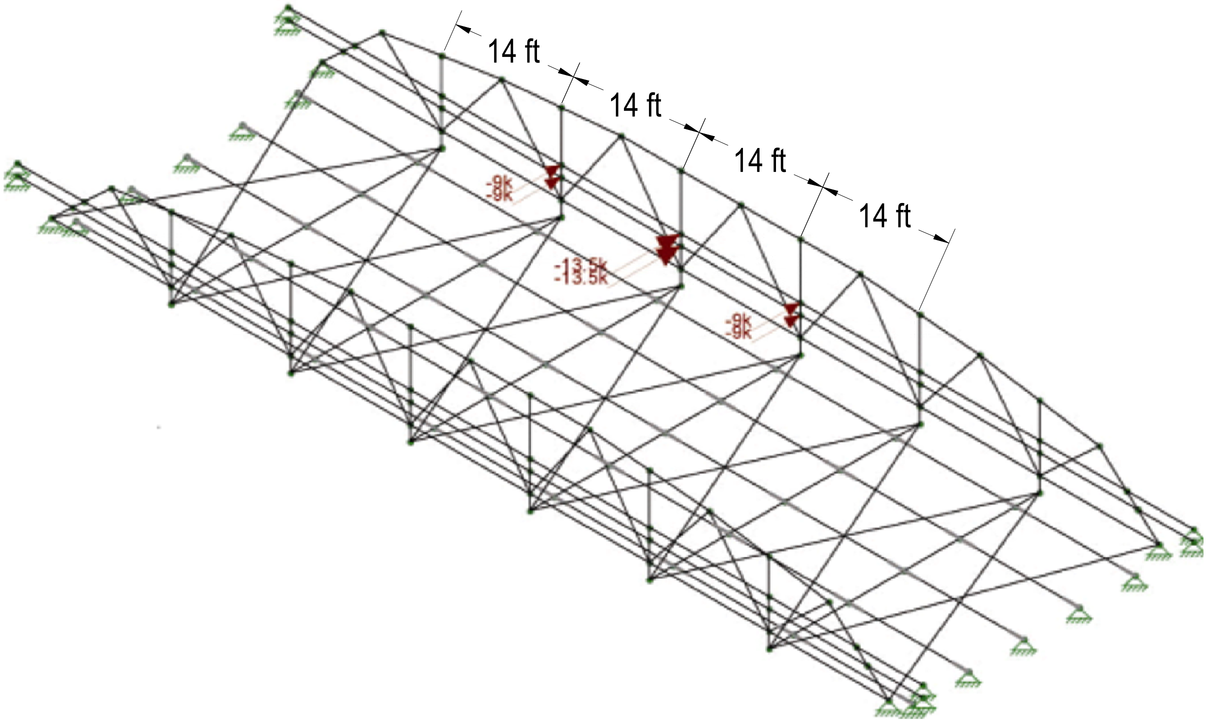


Figure 52. Superimposed Crash Loads from New Truss-Mounted Bridge Rail for Deer Creek Bridge Analysis.

The bridge railing members used in the analysis consist of two HSS8×8×6 (HSS203×203×152) tubes similar to the design shown in Figure 48. The bridge rails were connected to the vertical truss members and extended beyond the exterior members and connected to a simple pin-type connection beyond the exterior members to simulate the connection to a concrete parapet. The height of the bridge rail above the pavement surface was approximately 30 inches (762 mm).

Based on the results from the analysis of the existing Deer Creek Bridge with the proposed rail loads shown in Figure 50, several design modifications are required. The primary modifications required for the structure are increased moment resisting connections between the floor beams and the vertical truss members to resist the lateral crash loads. Moment resisting connections are also required at the exterior truss members (chords). If adequate moment resisting connections are provided at exterior chord members and at all connections between vertical truss members and bottom floor beams, some resizing of the truss members will be required to meet the strength requirements of AASHTO's *LRFD Bridge Design Specifications*. In addition, other changes will likely be required, such as resizing of gusset plates in the top chord member connections to adequately resist the crash loads. The modifications presented in this report pertain to the 98-ft (29.9 m) Deer Creek Bridge structure and may or may not apply to other bridge structures similar in type, length, size, and geometry.

CHAPTER 7. SUMMARY OF FINDINGS

LLANO TRUSS BRIDGE RAIL

A retrofit bridge railing was designed for the Llano Truss Bridge. The design reuses the existing C12×20.7 (C310×31) rail element and provides a TS8×4×1/2 (TS203×103×13) backup rail element to distribute longitudinal load. Short lengths of 5-inch (127 mm) and 6-inch (152 mm) diameter schedule 40 pipe form crushable blockouts to limit forces imposed on the truss members. A prototype of this railing was constructed and subjected to a full-scale crash test for TL-2 conditions.

The Llano Truss Bridge Rail contained and redirected the 2000P pickup truck. The 2000P pickup truck did not penetrate, underride, or override the installation. Maximum dynamic deflection during the test was 1.8 inches (47 mm). No detached elements, fragments, or other debris was present to penetrate or to show potential for penetrating the occupant compartment or to present undue hazard to others in the area. Maximum occupant compartment deformation was 0.4 inch (10 mm) in the kick panel area near the passenger's feet, laterally across the cab. The vehicle remained upright during and after the collision event. The vehicle came to rest upright 172.6 ft (52.6 m) downstream of impact and 6.2 ft (1.9 m) forward of the traffic face of the rail. Longitudinal occupant impact velocity was 13.8 ft/s (4.2 m/s) and longitudinal ridedown acceleration was -7.0 g's. Exit angle at loss of contact was 5.0 degrees, which was 20 percent of the impact angle.

The prototype for the retrofit of the Llano Truss Bridge performed acceptably according to the criteria specified for *NCHRP Report 350* Test 2-11, as shown in Table 3.

U.S. 281 TRUSS BRIDGE RAIL

A retrofit bridge railing was designed for the U.S. 281/Brazos River Bridge. The existing railing consists of a C12×20.7 (C310×31) mounted on truss members and intermediate posts. The channel rail element was reused and a W6×20 (W150×30) rail element backed up and stiffened the channel. New posts were designed to mount on the concrete safety walk. This design distributes loads through the concrete deck rather than applying them directly to the truss members. A prototype of the railing was constructed and subjected to a full-scale crash test for TL-3 conditions.

The U.S. 281Truss Bridge Rail contained and redirected the 2000P pickup truck. The 2000P pickup truck did not penetrate, underride, or override the installation. Maximum dynamic deflection during the test was 3.6 inches (91 mm). No detached elements, fragments, or other debris was present to penetrate or to show potential for penetrating the occupant compartment or to present undue hazard to others in the area. Maximum occupant compartment deformation was 1.6 inches (42 mm) in the kick panel area near the passenger's feet, laterally across the cab. The vehicle remained upright during and after the collision event. The vehicle came to rest upright 225.2 ft (68.6 m) downstream of impact and 50.0 ft (15.2 m) forward of the traffic face of the rail. Longitudinal occupant impact velocity was 17.4 ft/s (5.3 m/s) and longitudinal ridedown acceleration was -8.7 g's. Exit angle at loss of contact was 9.7 degrees, which was 38 percent of the impact angle.

The prototype for the retrofit of the U.S. 281 Truss Bridge performed acceptably according to the criteria specified for *NCHRP Report 350* Test 3-11, as shown in Table 4.

DESIGN EXCEPTIONS

The relatively narrow roadway widths of some existing historic truss bridges located on high-speed highways justify modification of the design crash test conditions and design loads for bridge rails for those structures. The analysis presented herein shows that the approach angle for TL-3 conditions can be reduced from 25 degrees to 20 degrees for a roadway width of 28 ft (8.5 m) or less. The resulting transverse force imposed on the rail can be reduced from 54 kips (240 kN) to 42 kips (187 kN).

TRAFFIC RAIL FOR NEW TRUSS BRIDGES

A new railing design for TL-3 was developed and is proposed for use on new truss bridges. It provides two tubular steel rail elements mounted on crushable blockouts made from 10-inch (254 mm) diameter schedule 80 pipe. Total height of the railing above the top of the deck is 2 ft 6 inches (0.8 m), and the traffic face presents suitable geometry. The railing is adequate for spans up to 20 ft (6.1 m) between supporting truss members. The crushable blockouts limit the lateral force applied to truss members to 15 kips (67 kN) or less, if the blockout is not crushed more than 5-1/2 inches (140 mm).

Since TxDOT anticipates that most new truss construction will be of the pre-fabricated, fabricator-designed type, implementation of the new rail system would require that the fabricator/designer demonstrate that the truss has been designed for the rail crash loading.

Table 3. Performance Evaluation Summary for *NCHRP Report 350* Test 2-11 on the Llano Truss Bridge Rail.

Test Agency: Texas Transportation Institute | Test No.: 444193-1 | Test Date: 05/30/2003

NCHRP Report 350 **Test 2-11 Evaluation Criteria**	**Test Results**	**Assessment**
Structural Adequacy A. *Test article should contain and redirect the vehicle; the vehicle should not penetrate, underride, or override the installation, although controlled lateral deflection of the test article is acceptable.*	The Truss Bridge Rail contained and redirected the 2000P pickup truck. The 2000P pickup truck did not penetrate, underride, or override the installation. Maximum dynamic deflection during the test was 1.8 inches (47 mm).	Pass
Occupant Risk D. *Detached elements, fragments, or other debris from the test article should not penetrate or show potential for penetrating the occupant compartment, or present an undue hazard to other traffic, pedestrians, or personnel in a work zone. Deformations of, or intrusions into, the occupant compartment that could cause serious injuries should not be permitted.*	No detached elements, fragments, or other debris was present to penetrate or to show potential for penetrating the occupant compartment or to present undue hazard to others in the area. Maximum occupant compartment deformation was 0.4 inch (10 mm) in the kick panel area near the passenger's feet, laterally across the cab.	Pass
F. *The vehicle should remain upright during and after collision, although moderate roll, pitching, and yawing are acceptable.*	The vehicle remained upright during and after the collision event.	Pass
Vehicle Trajectory K. *After collision it is preferable that the vehicle's trajectory not intrude into adjacent traffic lanes.*	The vehicle came to rest upright 172.6 ft (52.6 m) downstream of impact and 6.2 ft (1.9 m) forward of the traffic face of the rail.	Pass*
L. *The occupant impact velocity in the longitudinal direction should not exceed 12 m/s and the occupant ridedown acceleration in the longitudinal direction should not exceed 20 g's.*	Longitudinal occupant impact velocity was 13.8 ft/s (4.2 m/s) and longitudinal ridedown acceleration was -7.0 g's.	Pass
M. *The exit angle from the test article preferably should be less than 60 percent of test impact angle, measured at time of vehicle loss of contact with test device.*	Exit angle at loss of contact was 5.0 degrees, which was 20 percent of the impact angle.	Pass *

*Criteria K and M are preferable, not required.

Table 4. Performance Evaluation Summary for *NCHRP Report 350* Test 3-11 on the U.S. 281 Truss Bridge Rail.

Test Agency: Texas Transportation Institute — Test No.: 444193-2 — Test Date: 07/30/2003

***NCHRP Report 350* Test 3-11 Evaluation Criteria**	**Test Results**	**Assessment**
Structural Adequacy A. *Test article should contain and redirect the vehicle; the vehicle should not penetrate, underride, or override the installation, although controlled lateral deflection of the test article is acceptable.*	The Truss Bridge Rail contained and redirected the 2000P pickup truck. The 2000P pickup truck did not penetrate, underride, or override the installation. Maximum dynamic deflection during the test was 3.6 inches (91 mm).	Pass
Occupant Risk D. *Detached elements, fragments, or other debris from the test article should not penetrate or show potential for penetrating the occupant compartment, or present an undue hazard to other traffic, pedestrians, or personnel in a work zone. Deformations of, or intrusions into, the occupant compartment that could cause serious injuries should not be permitted.*	No detached elements, fragments, or other debris was present to penetrate or to show potential for penetrating the occupant compartment or to present undue hazard to others in the area. Maximum occupant compartment deformation was 1.6 inches (42 mm) in the kick panel area near the passenger's feet, laterally across the cab.	Pass
F. *The vehicle should remain upright during and after collision, although moderate roll, pitching, and yawing are acceptable.*	The vehicle remained upright during and after the collision event.	Pass
Vehicle Trajectory K. *After collision it is preferable that the vehicle's trajectory not intrude into adjacent traffic lanes.*	The vehicle came to rest upright 225.2 ft (68.6) downstream of impact and 50.0 ft (15.2 m) forward of the traffic face of the rail.	Fail*
L. *The occupant impact velocity in the longitudinal direction should not exceed 12 m/s and the occupant ridedown acceleration in the longitudinal direction should not exceed 20 g's.*	Longitudinal occupant impact velocity was 17.4 ft/s (5.3 m/s) and longitudinal ridedown acceleration was -8.7 g's.	Pass
M. *The exit angle from the test article preferably should be less than 60 percent of test impact angle, measured at time of vehicle loss of contact with test device.*	Exit angle at loss of contact was 9.7 degrees, which was 38 percent of the impact angle.	Pass *

*Criteria K and M are preferable, not required.

CHAPTER 8. IMPLEMENTATION

EXISTING TRUSS RETROFIT RAILS

A truss-mounted retrofit railing was designed for the Roy B. Inks Bridge on State Highway 16 over the Llano River in Llano, Texas. A prototype of this railing was subjected to a full-scale crash test for TL-2 conditions, and its performance was found acceptable. A structural analysis of the trusses indicates they are adequate to resist collision loads from the railing. With additional detailing, the retrofit will be suitable for installation on the Roy B. Inks Bridge.

A curb-mounted retrofit railing was designed for the U.S. 281 Bridge over the Brazos River in Palo Pinto County. A prototype of this railing was subjected to a full-scale crash test for TL-3 conditions, and its performance was found acceptable. With additional detailing, the retrofit will be suitable for installation on the U.S. 281 Bridge.

The retrofit railings developed for the Inks Bridge and the U.S. 281 Bridge can be adapted for use on other historic metal truss bridges on the State Highway System provided that they have similar curbs, similar design speeds, and similar truss geometry, and provided that analysis can demonstrate adequate strength to resist the theoretical impact loads. A reduction of AASHTO design forces for traffic railings is recommended for theoretical TL-3 impact load analysis of existing trusses when roadway widths are 28 ft (8.5 m) or less.

NEW TRUSS RAILS

The new bridge rail design developed from this research meets the strength requirements of *NCHRP Report 350*, Test Level 3. This railing is designed for mounting directly to Pratt-type or Warren-type trusses that have vertical truss members rigidly connected to transverse floorbeams. A minimum clear space of 3 inches (76 mm) is recommended between the railing and any diagnonal truss members that do not support the rail. The railing is designed for installation by bolted connection to the vertical members. The railing will meet *NCHRP Report 350* TL-3 requirements provided that:

1. the spacing between vertical members does not exceed 20 ft (6.1 m), and
2. the truss members and all associated components are designed for the theoretical crash loads transmitted to the truss through the rail, plus all dead load including the rail weight.

The following tables provide recommended crash loads to be used in the design of the bridge structure. Table 5 refers to crash loads applied to intermediate truss members (see Figure 53). Table 6 refers to the situation where crash loads are applied to the end of the bridge railing system connected to the end truss members. These loads are applicable where the bridge railing system does not extend beyond the end of the truss (see Figure 54). The loads presented in these tables should be used to analyze a 3-D model of the truss bridge and connections in conjunction with the dead load of the structure. The designer should confirm that the capacities of the members exceed the maximum member force due to the loading.

TxDOT anticipates that most new truss construction will be of the pre-fabricated, fabricator-designed type. Implementation of the new rail system with this type of truss would require that the fabricator/designer could demonstrate that the truss has been designed for the crash rail impact load case.

Table 5. Design Transverse Crash Loads for Intermediate Steel Truss Members.

Bridge Rail Type 2~HSS8×8×6 Rails with 10-inch Schedule 80 A53 Pipe Blockouts, 6 inches Long.		
Support Spacing (ft)	Lateral Design Force Per Rail Element Load at Support* (Intermediate Truss Members) (Force F1)	Lateral Design Force Per Rail Element Load at Adjacent Supports (X2) (Intermediate Truss Members)* (Force F2)
10	12.5	9.0
12	13.0	9.0
14	13.5	9.0
16	14.0	9.0
18	14.5	8.5
20	15.5	8.5

* Load applied to Upper and Lower Rail

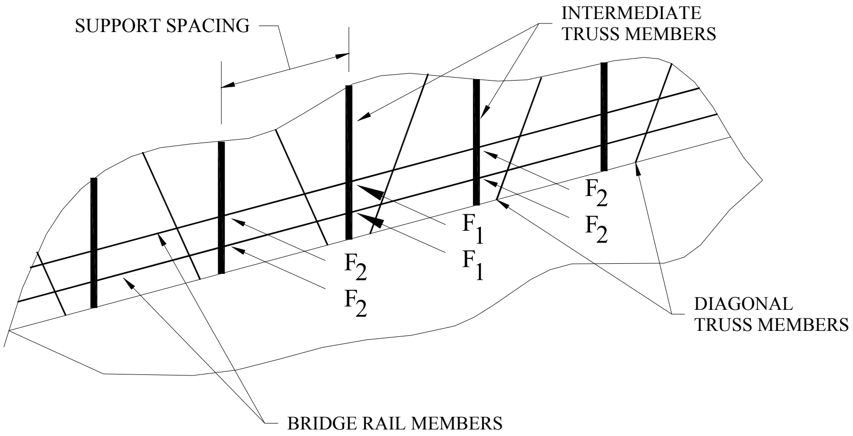


Figure 53. Configuration of Design Crash Loads at Intermediate Truss Members.

Table 6. Design Transverse Crash Loads at End Steel Truss Member and Adjacent Member.

<table>
<tr><td colspan="3">(Loads based on railing terminating at end truss member)
Bridge Rail Type: 2~HSS8×8×6 Rails with 10-inch Schedule 80 A53 Pipe Blockouts, 6 inches Long</td></tr>
<tr><th>Support Spacing (ft)</th><th>Lateral Design Force Per Rail Element Load at End Support* (Force F3)</th><th>Lateral Design Force Per Rail Element Load at Adjacent Support* (Force F4)</th></tr>
<tr><td>10</td><td>16.5</td><td>13.0</td></tr>
<tr><td>12</td><td>17.5</td><td>13.0</td></tr>
<tr><td>14</td><td>18.5</td><td>13.0</td></tr>
<tr><td>16</td><td>19.0</td><td>12.5</td></tr>
<tr><td>18</td><td>20.0</td><td>12.0</td></tr>
<tr><td>20</td><td>21.0</td><td>10.0</td></tr>
</table>

* Load applied to Upper and Lower Rail

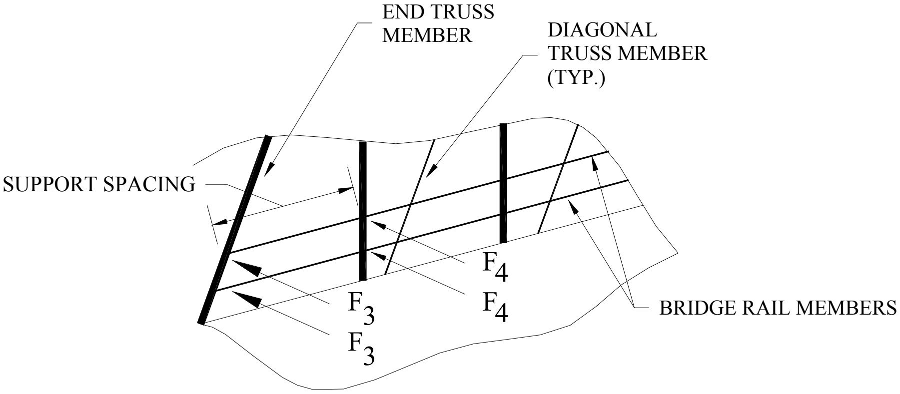


Figure 54. Configuration of Design Crash Loads at End Truss Members.

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2. *2000 Interim AASHTO LRFD Bridge Design Specifications, Customary U.S. Units, Second Edition*, American Association of State Highway and Transportation Officials, Washington, D.C., 1998.

3. R.M. Olson, E.R. Post, and W.F. McFarland, *Tentative Service Requirements for Bridge Rail Systems*, National Cooperative Highway Research Program Report 86, Transportation Research Board, National Research Council, Washington, D.C., 1970.

APPENDIX A. CRASH TEST PROCEDURES AND DATA ANALYSIS

The crash test and data analysis procedures were performed in accordance with guidelines presented in *NCHRP Report 350*. Brief descriptions of these procedures are presented below.

ELECTRONIC INSTRUMENTATION AND DATA PROCESSING

The test vehicle was instrumented with three solid-state angular rate transducers to measure roll, pitch, and yaw rates; a triaxial accelerometer near the vehicle center of gravity to measure longitudinal, lateral, and vertical acceleration levels; and a backup biaxial accelerometer in the rear of the vehicle to measure longitudinal and lateral acceleration levels. These accelerometers were Endevco Model 2262CA piezoresistive accelerometers with a ±100 g range.

The accelerometers are strain gage type with a linear millivolt output proportional to acceleration. Angular rate transducers are solid-state gas flow units designed for high-“g” service. Signal conditioners and amplifiers in the test vehicle increase the low-level signals to a ±2.5 volt maximum level. The signal conditioners also provide the capability of an R-cal or shunt calibration for the accelerometers and a precision voltage calibration for the rate transducers. The electronic signals from the accelerometers and rate transducers are transmitted to a base station by a 15-channel, constant bandwidth, Inter-Range Instrumentation Group (IRIG) FM/FM telemetry link for recording on magnetic tape and display on a real-time strip chart. Calibration signals from the test vehicle are recorded before the test and immediately afterward. A crystal-controlled time reference signal is recorded simultaneously with the data. Wooden dowels actuate pressure-sensitive switches on the bumper of the impacting vehicle prior to impact to indicate the elapsed time over a known distance and provide a measurement of impact velocity. The initial contact also produces an “event” mark on the data record to establish the instant of contact with the installation.

The multiplex of data channels, transmitted on one radio frequency, is received and demultiplexed onto separate tracks of a 28-track IRIG tape recorder. After the test, the data are played back from the tape machine and digitized. A proprietary software program (WinDigit) converts the analog data from each transducer into engineering units using the R-cal and pre-zero values at 10,000 samples per second per channel. WinDigit also provides Society of Automotive Engineers (SAE) J211 class 180 phaseless digital filtering and vehicle impact velocity.

All accelerometers are calibrated annually according to SAE J211 4.6.1 with an Endevco 2901 precision primary vibration standard. This device and its support instruments are returned to the factory annually for a National Institute of Standards Technology (NIST) traceable calibration. The subsystems of each data channel are also evaluated annually, using instruments with current NIST traceability, and the results are factored into the accuracy of the total data channel, per SAE J211. Calibrations and evaluations are repeated any time data are suspect.

The Test Risk Assessment Program (TRAP) uses the data from WinDigit to compute occupant/compartment impact velocities, time of occupant/compartment impact after vehicle impact, and the highest 10-ms average ridedown acceleration. WinDigit calculates change in vehicle velocity at the end of a given impulse period. In addition, it computes maximum average accelerations over 50-ms intervals in each of the three directions. For reporting purposes, the data from the vehicle-mounted accelerometers are filtered with a 60-Hz digital filter, and acceleration versus time curves for the longitudinal, lateral, and vertical directions are plotted using TRAP.

TRAP uses the data from the yaw, pitch, and roll rate transducers to compute angular displacement in degrees at 0.0001-s intervals and then plots yaw, pitch, and roll versus time. These displacements are in reference to the vehicle-fixed coordinate system with the initial position and orientation of the vehicle-fixed coordinate systems being initial impact.

ANTHROPOMORPHIC DUMMY INSTRUMENTATION

Use of a dummy in the 2000P vehicle is optional according to *NCHRP Report 350,* and there was no dummy used in the test.

PHOTOGRAPHIC INSTRUMENTATION AND DATA PROCESSING

Photographic coverage of the test included three high-speed cameras: one overhead with a field of view perpendicular to the ground and directly over the impact point, one placed behind the installation at an angle, and a third placed to have a field of view parallel to and aligned with the installation at the downstream end. A flashbulb activated by pressure-sensitive tape switches was positioned on the impacting vehicle to indicate the instant of contact with the installation and was visible from each camera. The films from these high-speed cameras were analyzed to observe phenomena occurring during the collision and to obtain time-event, displacement, and angular data. A BetaCam, a VHS-format video camera and recorder, and still cameras recorded and documented conditions of the test vehicle and installation before and after the test.

TEST VEHICLE PROPULSION AND GUIDANCE

The test vehicle was towed into the test installation using a steel cable guidance and reverse tow system. A steel cable for guiding the test vehicle was tensioned along the path, anchored at each end, and threaded through an attachment to the front wheel of the test vehicle. An additional steel cable was connected to the test vehicle, passed around a pulley near the impact point, through a pulley on the tow vehicle, and then anchored to the ground such that the tow vehicle moved away from the test site. A 2:1 speed ratio between the test and tow vehicle existed with this system. Just prior to impact with the installation, the test vehicle was released to be freewheeling and unrestrained. The vehicle remained freewheeling (i.e., no steering or braking inputs) until the vehicle cleared the immediate area of the test site, at which time brakes on the vehicle were activated to bring it to a safe and controlled stop.

APPENDIX B. DESIGN OF RETROFIT RAIL FOR ROY B. INKS BRIDGE OVER LLANO RIVER, LLANO, TEXAS

The calculations contained in this appendix pertain to the design of a retrofit bridge rail for the Roy B. Inks Bridge over the Llano River in Llano, Texas. Analyses performed on the existing design revealed that it did not meet the current Test Level 2 strength requirements as specified in *AASHTO LRFD Bridge Design Specifications*. The existing design utilized a single steel rail fabricated from C12×20.7 (C310×31) channel attached to all bridge truss members with riveted clip angles. The clip angles attach to the flanges of the channel and connect to the truss members. Intermediate posts between the truss members support the rail. For this project, the researchers considered several different options to strengthen the existing design to meet the current strength requirements. Adding a structural tube behind the channel rail and reducing the height of the rail was selected as the best option by the researchers and TxDOT personnel.

The retrofit rail design consists of blocking out the existing C12×20.7 (C310×31) rail approximately 6 inches (152 mm) from its current location and lowering the rail height from 37 inches (940 mm) to 32 inches (813 mm) above the pavement surface. To maintain the clear roadway width of 24 ft (7.3 m), the researchers selected a TS8×4×1/2 (TS203×102×12.7) steel tube located behind the channel. Several different blockout types were considered at the post locations. Steel blockouts fabricated from W8×18 (W200×27) shape were selected for the intermediate post locations. These blockouts serve to move the rail near the face of the curb. Blockouts are also required where the rail attaches to the truss members. The researchers considered several different blockout types for use at the truss members. Considering the geometry of all the different truss members, 5-inch (127 mm) diameter and 6-inch (152 mm) diameter steel pipe blockouts were selected and analyzed for this project. These blockouts serve to support the rail at the truss member locations as well as limit the crash loads imparted to the truss members due to the crush characteristics of the pipe. Strength analyses were performed on the retrofit rail, and the results of these analyses indicate that it satisfies the strength requirements specified in *AASHTO LRFD Design Specifications* for TL-2. Analyses of the crush characteristics of the pipe blockouts were performed separately. The calculations considering the performance of these pipe blockouts and the strength of the truss members are presented in Appendix C of this report.

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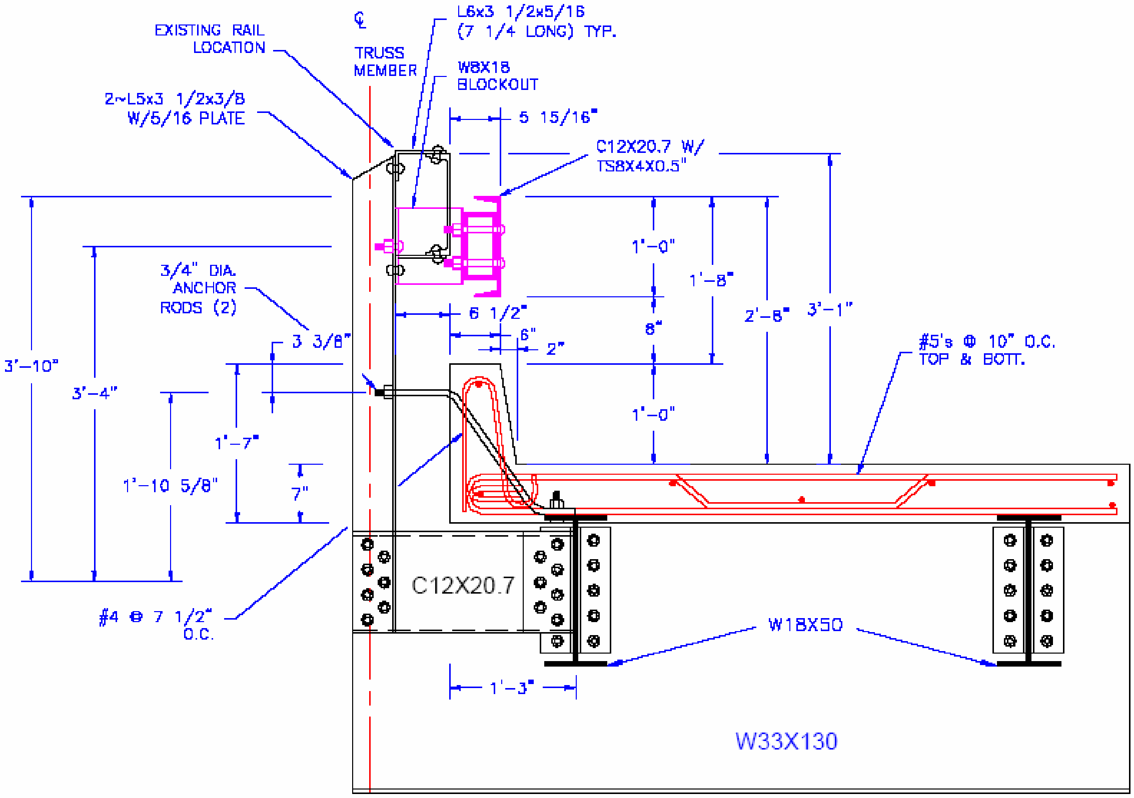
SUBJECT Retrofit Railing for SH 16
Truss Bridge over Llano River

BY: W. Williams
CKD: ____________

CLIENT TxDOT

Given Information: 1.) SH 16 at the Llano River, Llano County.
2.) Built 1936, 4 Spans @ 198'-6 3/4" Each



PROPOSED 32-IN RAIL SUPERIMPOSED ON A TYPICAL EXISTING BRIDGERAIL POST

SUBJECT Retrofit Railing for SH 16
Truss Bridge over Llano River
CLIENT TxDOT

BY: W. Williams
CKD:

$f_{ysteel} := 30ksi$ $Z_{yrail} := 3.47in^3$

$F_{usteel} := 50ksi$ $TL2_{load} := 27kips$

$Dia_{rod} := 0.75in$ $Ht := 32in$ New rail height

$Area_{rod} := \pi \times Dia_{rod}^{\ 2} \times 0.25$

$L := 7.00ft$ ..... L = Post spacing (ft) used for "As-Is" analysis... 84 inches worst case

.... L = Post spacing 64.5 inches average

$Z_{yrail} = 3.47\,in^3$ ..... Plastic modulus of rail about weak axis

1.) Calculate the Plastic Strength of the Existing Rail (M_p):

$M_{p1} := Z_{yrail} \times f_{ysteel}$ $Z_{yrail} = 3.47\,in^3$ $f_{ysteel} = 30\,ksi$ $M_{p1} = 8.68\,kip \times ft$

Assume C12x20.7 will be discontinuous in the field (as it presently is existing), therefore, assume plastic strength "Z" of the new tube shape only:

.... If a TS9x6x3/8 tube is used: $Z_{yT9} := 23.1in^3$

$M_{p2} := Z_{yT9} \times f_{ysteel}$ $M_{p2} = 57.75\,kip \times ft$

.... If a TS8x4x1/2 tube is used: $Z_{yT8} := 15in^3$

$M_{p3} := Z_{yT8} \times f_{ysteel}$ $M_{p3} = 37.5\,kip \times ft$

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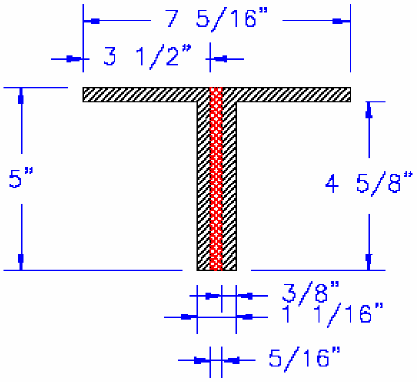
2.) Calculate the Plastic Strength of the Posts (P_{post1}):



1.) Calculate pastic neutral axis:

$x := 1in$

Given

$7.3125in \times 0.375in + (x \times 1.0625in) = (4.625in - x) \times 1.0625in$

$x := \mathrm{Find}(x)$

$x = 1.02\,in$

$Area_1 := 7.3125in \times .375in + (x \times 1.0625in)$ $\quad Area_1 = 3.83\,in^2$

$Area_2 := (4.625in - x) \times 1.0625in$ $\quad Area_2 = 3.83\,in^2$

$$y_1 := \frac{\left[7.3125 \times in \times 0.375 \times in \times \left(x + \frac{0.375 \times in}{2}\right)\right] + \left[(x \times 1.0625 \times in) \times \frac{x}{2}\right]}{Area_1}$$

$y_1 = 1.01\,in$

$y_2 := \frac{4.625in - x}{2}$ $\quad y_2 = 1.8\,in$

$$Z := \left(\text{Area}_2\right) \times y_2 + \left(\text{Area}_1\right) \times y_1$$

$$Z = 10.77\,\text{in}^3 \qquad F_y := 30\text{ksi}$$

$$M_{post} := Z \times F_y$$

$$M_{post} = 26.92\,\text{kip} \times \text{ft}$$

$$P_{Post1} := \frac{M_{post}}{32\text{in} - 12\text{in} + 3.375\text{in}}$$

$$P_{Post1} = 13.82\,\text{kip}$$

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3.) Calculate the Strength of the Post Based on Tension Failure of 3/4" ϕ Rods:

$F_{usteel} = 50\,\text{ksi}$ .... Rupture strength of steel (approx.) (for 2-3/4" Dia. rods)

$\phi := 1.0 \qquad \phi R_n := \phi \times F_{usteel} \times (0.75 \times Area_{rod}) \times 2$

$\phi R_n = 33.13\,\text{kips}$

$$P_{post2} := \frac{22.625 \times in \times (\phi R_n)}{46in} \qquad P_{post2} = 16.3\,\text{kips}$$

Therefore: $P_{Post} := P_{Post1}$

$P_{Post} = 13.82\,\text{kips}$

4.) Calculate the Force Applied to the 3/4" Dia. Rods:

$$F_{rods} := \frac{46in \times P_{Post1}}{22.625in} \qquad F_{rods} = 28.1\,\text{kip}$$

5.) Calculate the Force into W18x50 from the Max. Strength of the Post:

.... Now sum hor. forces:

$F_{W18x50} := F_{rods} - P_{Post1} \qquad F_{W18x50} = 14.28\,\text{kips}$

...... Calculate applied torsion to W18x50 girder using applied shear from double angles:

$$Torsion_{W18x50} := \frac{F_{W18x50} \times 9in}{\frac{12in}{ft}} \qquad Torsion_{W18x50} = 10.71\,\text{kips} \times \text{ft}$$

...... Use this force as applied torsion on W18x50 in RISA 3D model

therefore: $P_{Post} := P_{Post1}$ Change these variables to match conditions

$M_p := M_{p3}$ ... Strength for TS8x4x1/2 tube

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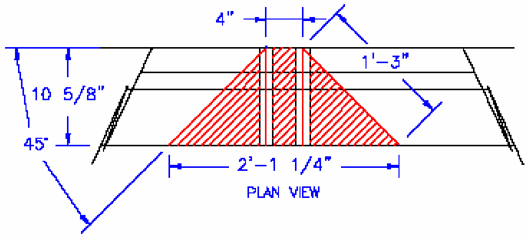
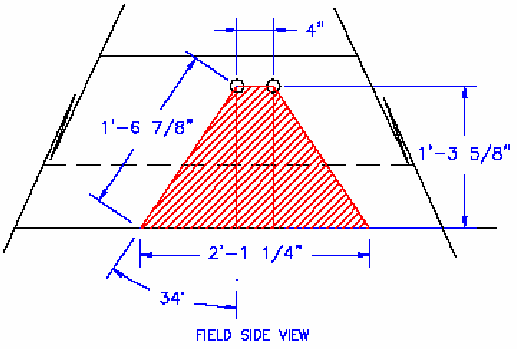
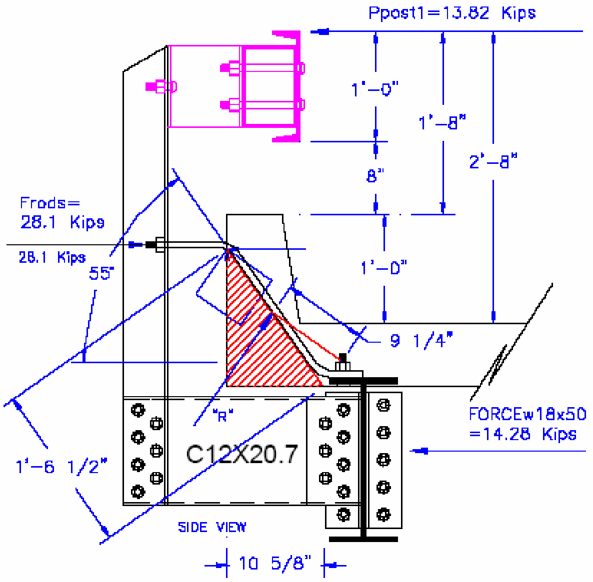
What about this case where rods completely "punch through"?:







$Area_1 := 4in \times 18.5in$ $\quad$ $f'_c := 3000psi$ $\quad$ Assumed compressive strength of concrete

$Area_2 := 15in \times 18.5in$

$Area_{tension} := \left(Area_1 + Area_2\right) \times 0.85$ $\quad$ $Area_{tension} = 298.78\,in^2$

$$V_{cr} := 4 \times \sqrt{f'_c \times psi} \times 0.85 \times Area_{tension}$$

$$V_{cr} = 55.64\,kips$$

7.) Determine Total Rail Resistance of TS8x4x1/2 Rail for One Span:

$P_{Post} = 13.82\,\text{kips}$ Post Spacing $L = 7\ \text{ft}$

$M_p = 37.5\,\text{kip} \times \text{ft}$ $L_t := 4.0\text{ft}$

$N := 1$ ..... Single Span Check

$$R_{1span} := \frac{16 \times M_p + (N-1) \times (N+1) \times P_{Post} \times L}{2 \times N \times L - L_t}$$

$R_{1span} = 60\,\text{kips}$

8.) Determine Total Rail Resistance of TS8x4x1/2 Rail for Two Span:

$N := 2$ ..... Two Span w/ Load applied @ Post

$M_p = 37.5\,\text{kips} \times \text{ft}$ $P_{Post} = 13.82\,\text{kips}$ $L = 7\ \text{ft}$

$$R_{2span} := \frac{16 \times M_p + N^2 \times P_{Post} \times L}{2 \times N \times L - L_t}$$

$R_{2span} = 41.12\,\text{kips}$

9.) Determine Total Rail Resistance of TS8x4x1/2 Rail for Three Span:

$P_{Post} = 13.82\,\text{kips}$ $M_p = 37.5\,\text{kip} \times \text{ft}$ $N := 3$ ..... Three Span Check

$$R_{3span} := \frac{16 \times M_p + (N-1) \times (N+1) \times P_{Post} \times L}{2 \times N \times L - L_t}$$

$R_{3span} = 36.16\,\text{kips}$

10.) Determine Total Rail Resistance of TS8x4x1/2 Rail for Four Span:

$N := 4$ ..... 4 Spans: $M_p = 37.5\,\text{kip} \times \text{ft}$ $P_{Post} = 13.82\,\text{kips}$ $L = 7\ \text{ft}$

$$R_{4span} := \frac{16 \times M_p + N^2 \times P_{Post} \times L}{2 \times N \times L - L_t}$$

$$R_{4span} = 41.3\,\text{kips}$$

11.) Determine Total Rail Resistance of TS8x4x1/2 Rail for Five Span:

$P_{Post} = 13.82\,\text{kips}$ Limiting Post Strength @ Baseplate

$M_p = 37.5\,\text{kips} \times \text{ft}$ $N := 5$ ..... 5 Span Check

$$R_{5span} := \frac{16 \times M_p + (N - 1) \times (N + 1) \times P_{Post} \times L}{2 \times N \times L - L_t}$$

$$R_{5span} = 44.27\,\text{kips}$$

12.) Determine Total Rail Resistance of TS8x4x1/2 Rail for Six Span:

$N := 6$ ..... 6 Spans:

$M_p = 37.5\,\text{kips} \times \text{ft}$ $P_{Post} = 13.82\,\text{kips}$ $L = 7\ \text{ft}$

$$R_{6span} := \frac{16 \times M_p + N^2 \times P_{Post} \times L}{2 \times N \times L - L_t}$$

$$R_{6span} = 51.03\,\text{kips}$$

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

$$f_y := 40\text{ksi}$$

$$f'_c = 3000\,\text{psi}$$

$$A_{sc} := 0.20\text{in}^2 \times 1.6$$ Vertical #4 Bars @ 7.5" O.C. or 1.6 bars per foot

$$f_y = 40\,\text{ksi}$$

$$b_c := 12\text{in}$$

$$a_c := \frac{A_{sc} \times f_y}{0.85 \times f'_c \times b_c}$$

$$a_c = 0.42\,\text{in}$$

$$d_c := 8\text{in} - 2\text{in}$$ "approximate distance to tension steel"

$$d_c = 6\,\text{in}$$

Calculate M_c:

with: $d_c = 6\,\text{in}$ $\phi = 1$

$A_{sc} = 0.32\,\text{in}^2$

$f_y = 40\,\text{ksi}$

$a_c = 0.42\,\text{in}$

Therefore:

$$M_c := \frac{(\phi \times A_{sc} \times f_y) \times \left(d_c - \frac{a_c}{2}\right)}{\text{ft}}$$

$$M_c = 6.18\frac{\text{kip} \times \text{ft}}{\text{ft}}$$ Flexural Resistance of 12-inch curb

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14.) Calculate the Bending Capacity of the Wall about the Vertical Axis: M_w, (kip x ft):

$A_{sw} := 0.31\text{in}^2 \times 1$ ... 1 Longitudinal Bars total in curb

$f_y = 40\,\text{ksi}$

$H_w := 12\text{in}$ ... Height of Wall

$b_c = 1\ \text{ft}$

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

$a_w = 0.41\,\text{in}$

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

$d_w = 2\,\text{in}$

Calculate M_w: with:

$d_w = 2\,\text{in}$ $H_w = 1\ \text{ft}$

$A_{sw} = 0.31\,\text{in}^2$ $\phi = 1$

$f_y = 40\,\text{ksi}$

$a_w = 0.41\,\text{in}$

Therefore:

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

$M_w = 1.86\,\frac{\text{kip} \times \text{ft}}{\text{ft}}$ Flexural Resistance of Wall in (K-FT/FT)

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15.) Determine Ultimate Resistance of Wall at Mid-span:

Calculate L_c ~ Critical length of yield line failure pattern:

with:

$M_b := 0\text{kip} \times \text{ft}$ ... No additional beam action resistance

$M_c = 6.18 \frac{\text{kip} \times \text{ft}}{\text{ft}}$

$M_w = 1.86 \frac{\text{kip} \times \text{ft}}{\text{ft}}$

$H := 12\text{in}$ ... Height of curb, (ft)

$L_t = 4\ \text{ft}$ Longitudinal length of distribution of impact force, (ft), TL-2 Conditions

$$L_c := \frac{L_t}{2} + \sqrt{\left(\frac{L_t}{2}\right)^2 + \frac{8 \times H \times (M_b + M_w \times H)}{M_c}}$$

(LRFD Equation A13.3.1-2, pg. A13-6)

$L_c = 4.53\ \text{ft}$

$$R_w := \left(\frac{2}{2 \times L_c - L_t}\right) \times \left(8 \times M_b + 8 \times M_w \times H + \frac{M_c \times L_c^{\,2}}{H}\right)$$

(Equation A13.3.1-1, pg. A13-6)

$R_w = 55.97\,\text{kips}$ Transverse Resistance of Curb, (Kips)

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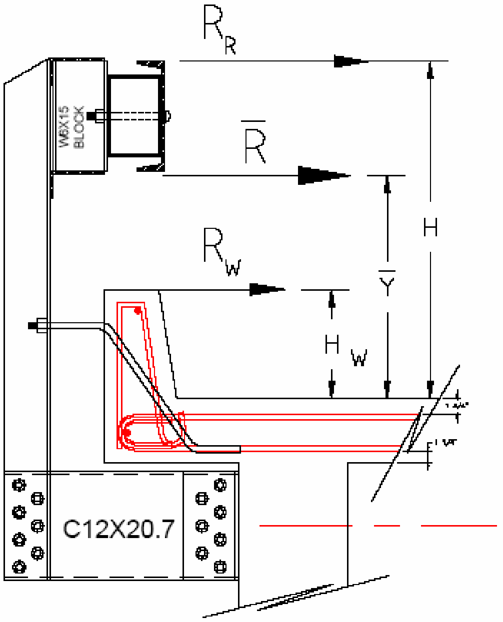
16.) Determine Combined Resultant Strength of Rail & Curb @ Mid-span:



$R_R := R_{3span}$

$R_R = 36.16\,kips$

$H_R := 32in$

$H_R = 2.67\;ft$

$H_W = 12\,in$

$R_W = 55.97\,kips$

Therefore:

$R_{Bar} := R_R + R_W$

$R_{Bar} = 92.13\,kips$ Total Combined Resistance of Rail, (kips)

$Y_{Bar} := \dfrac{R_R \times H_R + R_W \times H_W}{R_{Bar}}$ $Y_{Bar} = 19.85\,in$ Height of Resultant, ft

******* Resultant @ 24-inch Height *********

$Y_{27} := 27in$

$R_{bar27} := \dfrac{R_{Bar} \times Y_{Bar}}{Y_{27}}$

$R_{bar27} = 67.73\,kips$ @ 27 inches

***** Resultant @ 32-inch Height ***********

$Y_{32} := 32in$

$R_{bar32} := \dfrac{R_{Bar} \times Y_{Bar}}{Y_{32}}$

$R_{bar32} = 57.15\,kips$ @ 32 inches

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In Summary: The new 32-inch high retrofit rail meets the requirements for TL-2 loading conditions (F_t = 27 kips @ 27 inches).

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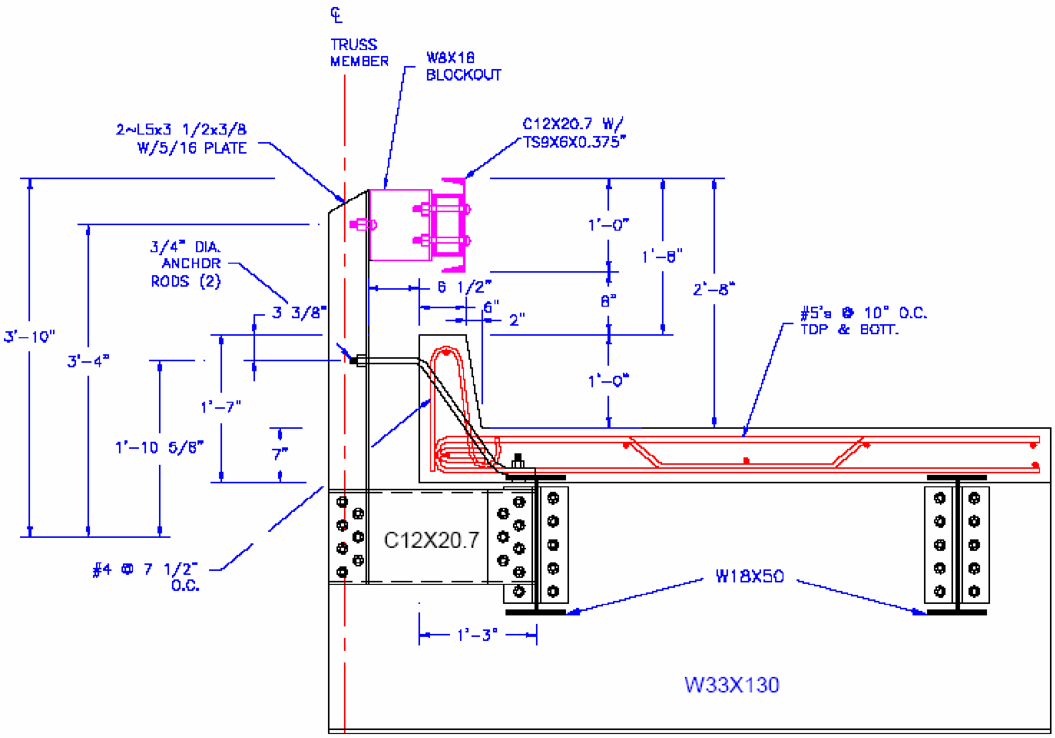

SECTION E-E: TYPICAL SECTION
AT BRIDGERAIL POST ELEVATION

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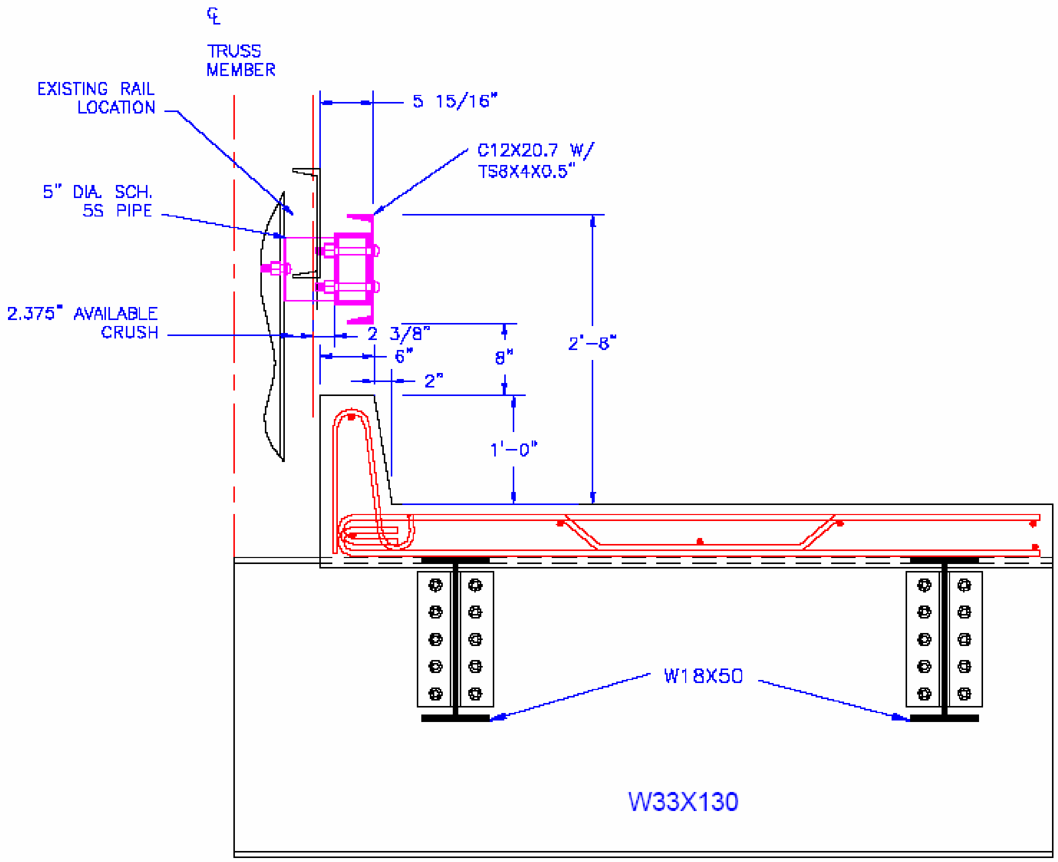

SECTION B–B: MODIFIED RAIL SECTION
2~C15X55 W/18.25X5/8" PLATE TOP
(BRIDGE TRUSS ENDS)

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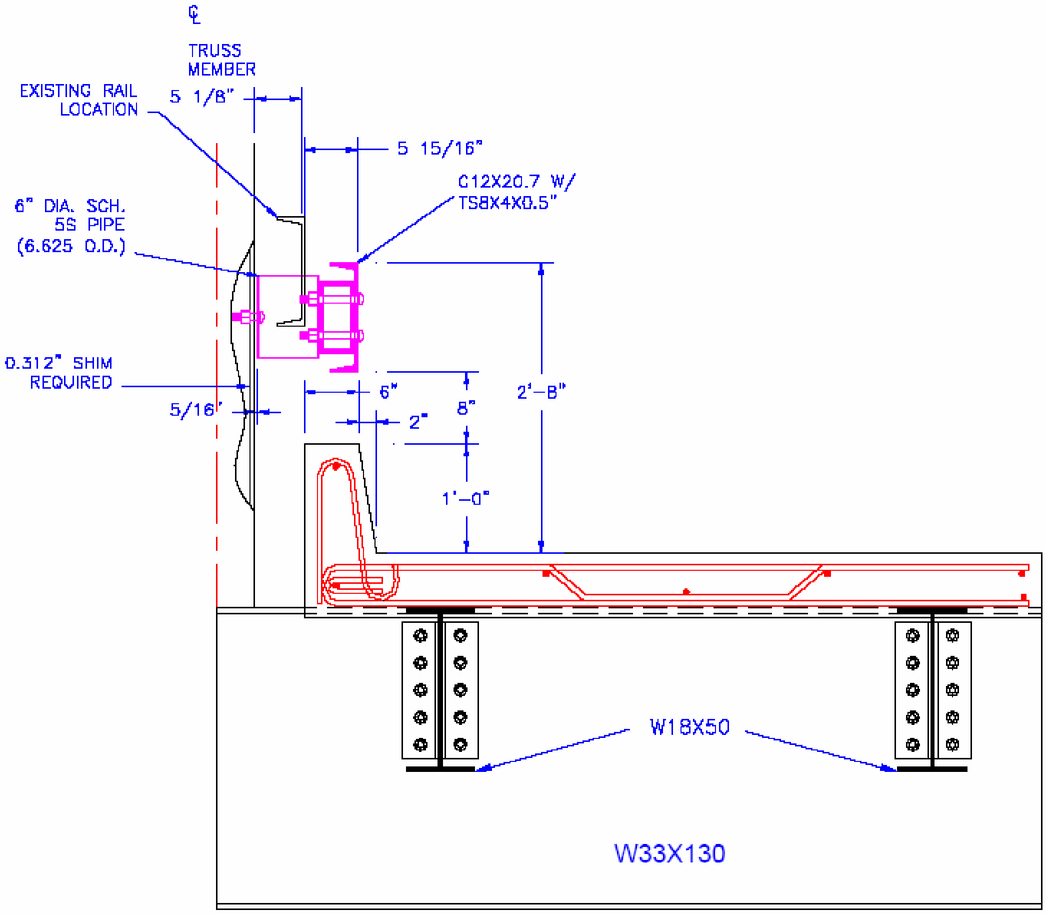

SECTION C-C: MODIFIED RAIL
SECTION AT W8X40's
VERTICALS & DIAGONALS

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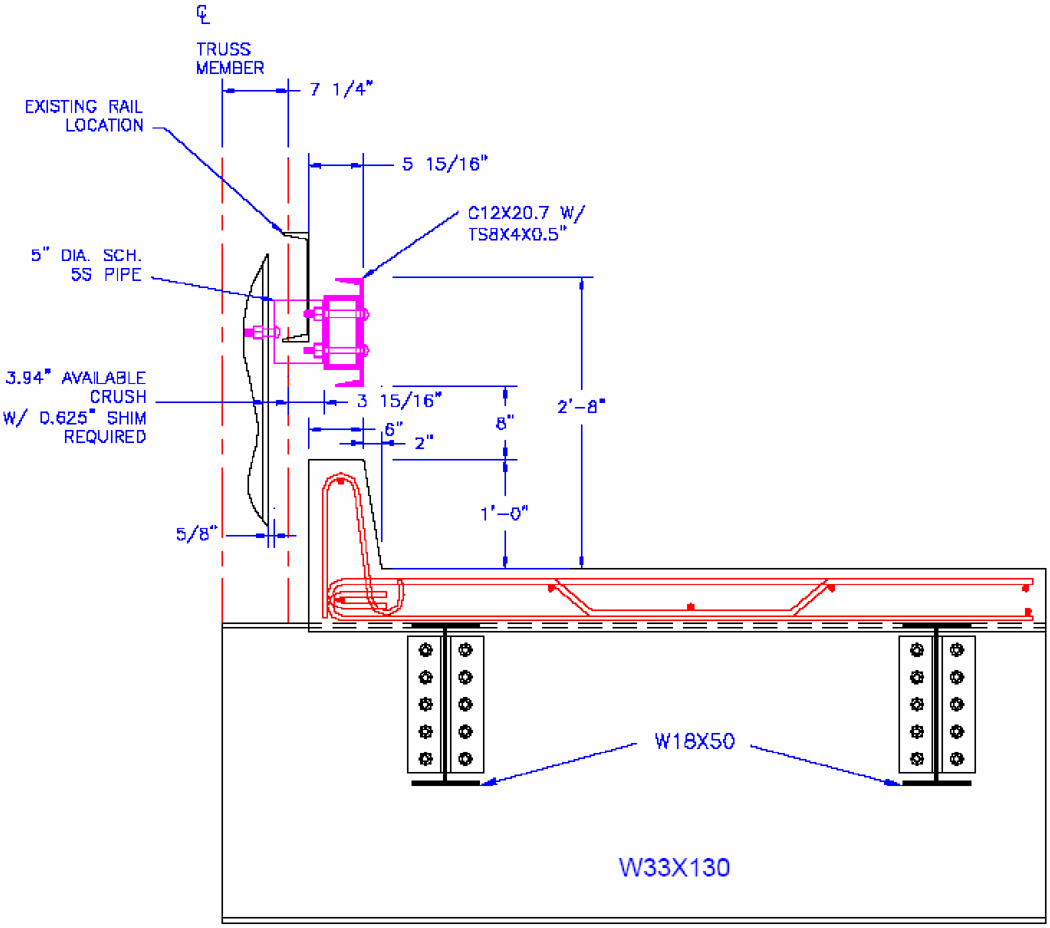

SECTION D–D: MODIFIED RAIL
SECTION AT 2~C9X20's
(BRIDGE TRUSS VERTICALS)

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17.) Truss W8x40 Member Check for 8.75-kip Rail Load:

$l := 40ft \qquad x := 7.25ft \qquad P := 8.75kips$

$b := l - x \qquad a := l - b$

$$M_{max} := \frac{P \times a \times l}{l} \qquad M_{max} = 63.44\,kip \times ft$$

$F_b := 30ksi$

$Z_{xw8} := 39.9in^3$

$$SR_{w8impact} := \frac{\frac{M_{max}}{Z_{xw8}}}{F_b}$$

$SR_{w8impact} = 0.64$ ..... o.k. ... < 1.0!

18.) Truss 2~C9x15's Members Check for 8.75-kip Rail Load:

$l := 35ft \qquad x := 6.0ft \qquad P := 8.75kips$

$A_{C9x15} := 4.41in^2$

$b := l - x \qquad a := l - b$

$x_p := 0.243in$

$$M_{max} := \frac{P \times a \times l}{l} \qquad M_{max} = 52.5\,kip \times ft$$

$b_f := 2.485in$

$F_b := 30ksi$

$$d := \frac{14.5 \times in - 2 \times \left(b_f - x_p\right)}{2} \qquad d = 5.01\,in \qquad Z_{C9} := 2 \times \left(A_{C9x15} \times d\right) \qquad Z_{C9} = 44.17\,in^3$$

$$SR_{C9} := \frac{\frac{M_{max}}{Z_{C9}}}{F_b} \qquad SR_{C9} = 0.48$$

... o.k. < 1.0!

SUBJECT Retrofit Railing for SH 16
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CKD: ____________

19.) Calculate Slab & Curb Weight over Outside Stringer for use in RISA-3D Model for W18x50 Stringer:

$$\mathrm{Width_c} := 2.25\mathrm{ft} + 15\mathrm{in} \qquad \mathrm{Width_c} = 3.5\ \mathrm{ft}$$

$$\mathrm{Slab_{wt}} := \left(\mathrm{Width_c} \times 7 \times \mathrm{in} \times 0.150 \times \frac{\mathrm{kip}}{\mathrm{ft}^3}\right) + \left[(12)\mathrm{in} \times 6\mathrm{in} \times 0.150 \frac{\mathrm{kip}}{\mathrm{ft}^3}\right]$$

$$\mathrm{Slab_{wt}} = 1.23 \times 10^4\ \mathrm{ft} \cdot \mathrm{s}^{-2} \frac{\mathrm{lb}}{\mathrm{ft}}$$

$$\mathrm{L} := 22\mathrm{ft} \qquad \mathrm{b_f} := 7.5$$

$$\mathrm{Pressure_{W18x50}} := \frac{\mathrm{Slab_{wt}} \times \mathrm{L}}{\mathrm{b_f} \times \mathrm{L}}$$

$$\mathrm{Pressure_{W18x50}} = 50.83\ \mathrm{ft\,psf}$$ .... say 51psf

APPENDIX C. ANALYSES OF BRIDGE TRUSS MEMBERS SUPPORTING RETROFIT RAIL, ROY B. INKS BRIDGE OVER LLANO RIVER, LLANO, TEXAS

The calculations contained in this appendix pertain to the analyses of the collision loads imposed on the truss members that support the retrofit rail for the State Highway 16 Truss Bridge over the Llano River in Llano, Texas. This retrofit rail element is a section constructed from TS8×4×1/2 (TS203×102×12.7) attached to a C12×20.7 (C310×31) channel. As a means to minimize the collision forces transmitted to the truss members from vehicular collision forces, the researchers proposed to block out the rail from the existing truss members using either 5-inch (127 mm) or 6-inch (152 mm) A53 grade B schedule 40 pipe blockouts, depending on the geometry of the truss members. At the intermediate post locations, the researchers recommend that the new rail be blocked out using short pieces of W8×18 (W200×27) structural shape (steel blockouts). These posts are located between truss members. This new rail design using the pipe blockouts and the intermediate posts with W8×18 (W200×27) blockouts was used in the installation that was tested on May 30, 2003. The crushable pipe blockouts were included in the design to limit the collision loads applied to the truss members. Based on the results from the calculations contained in this appendix, the existing truss members have sufficient capacity to support the dead loads from the structure and resist the TL-2 collision loading conditions from the new retrofit rail design when this rail is blocked out from the truss members using the crushable steel pipe blockouts. Additional details are provided below.

Researchers performed structural analyses using the new rail design rigidly attached to the truss members. They calculated section properties of all the composite sections used in the bridge structure and developed a structural model of a single bridge span 198 ft 6-3/4 inches (60.52 m) in length. The calculated section properties of these members are presented in this appendix. The structural modeling program RISA-3D was used to develop and analyze the model. The dead load of the structure and the TL-2 design forces for traffic railings from AASHTO *LRFD Bridge Design Specifications* Table A13.2-1 were considered in the analyses. TL-2 loading conditions in the transverse and vertical directions as per AASHTO *LRFD Bridge Design Specifications* were applied at different locations along the bridge rail to produce the maximum stress in the truss member from the dead and collision loads. The transverse force used in the analyses consisted of 27 kips (120 kN) distributed over 4 ft (1.2 m). The vertical force used in the analyses consisted of 4.5 kips (20 kN) distributed over 4 ft (1.2 m). A longitudinal force of 4.0 kips (18 kN) distributed over 4 ft (1.2 m) was used in the analyses. The recommended longitudinal design force of 9.0 kips (40 kN) was reduced due to some force transmitted to the concrete curb. The truss members were analyzed in accordance with *LRFD Bridge Design Specifications* with respect to the combined stress in the truss members from the dead and collision loads. The stress ratios (ratio of required axial and biaxial bending strengths to the nominal strengths of the section) were less than 1.0 in all members except the second vertical members from the ends of the truss, which are built-up sections constructed from two C9×20 (C230×30) channel shapes. Further analyses were required to determine if the crushable pipe blockouts would reduce the stress in these members from the TL-2 crash loads. The purpose of this study was to determine if these members could adequately resist the TL-2 loading from the rail with the use of the crushable steel pipe blockouts between the rail and the truss members.

Finite element analyses using LS-DYNA were performed on both the 5-inch (127 mm) and 6-inch (152 mm) schedule 40 pipe blockouts to determine the maximum crush strength for each size. The length of the pipe blockouts was 12 inches (305 mm). The maximum crush strength for the 5-inch (127 mm) diameter blockout was approximately 6.7 kips per inch (1.17 kN/mm) of deflection. The maximum crush strength for the 6-inch (152 mm) diameter blockout was approximately 7.8 kips per inch (1.37 kN/mm) of deflection. A summary of the crush strength analyses is shown in Figure C-1. The results from these analyses were used in a separate structural analysis of the rail to determine the actual applied force to truss members from collision loads on the rail.

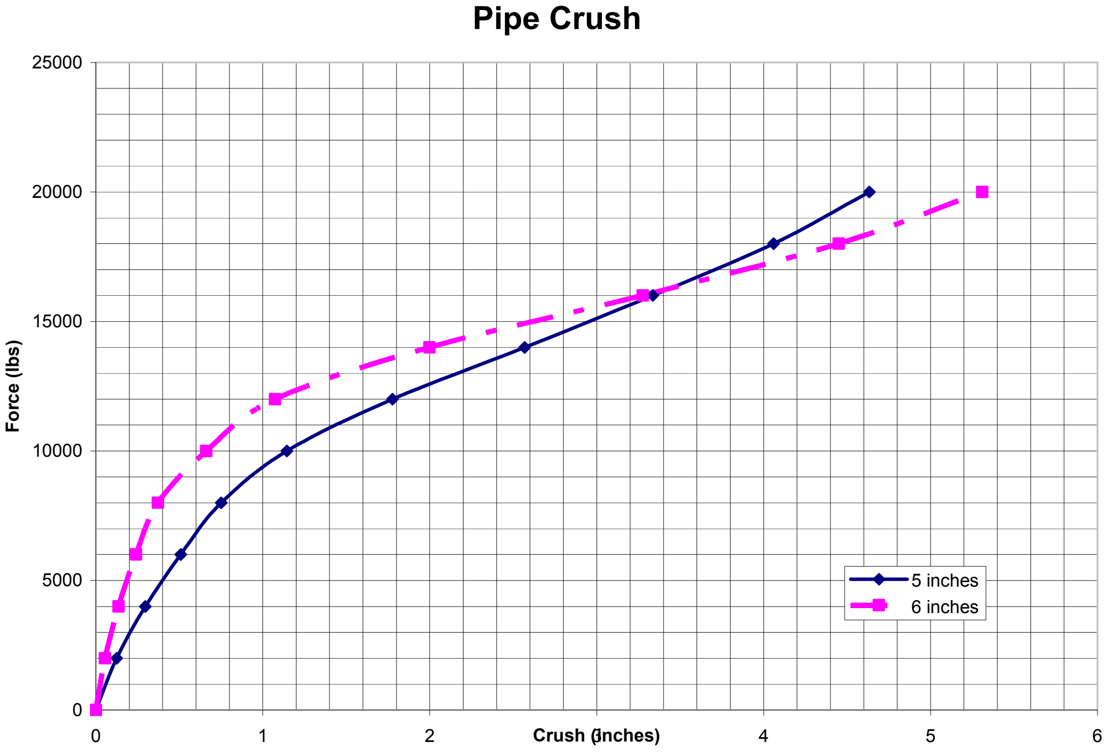


Figure C-1. Crush Strength Analyses.

Structural analyses were performed on the bridge rail system supported by intermediate posts and the bridge rail attached directly to the steel truss members (without crushable pipe blockouts between the rail and truss members). TL-2 loading conditions were applied to the rail at the vertical and diagonal truss members. Axial and biaxial bending strength analyses were performed on the truss members considering the crash loads applied to the rail at the truss members and the axial loads applied to the truss members from the dead weight of the bridge structure. The built-up engineering properties of each section were determined and used in these analyses. The results from these analyses and the properties of the built-up sections are included herein. Based on the results from the strength analyses, one truss member (member L2-U2, Node 4) was considerably overstressed when crash loads were applied to the rail with the rail directly attached to the truss member.

A separate analysis using RISA-3D was performed using the geometry and properties of the rail with the TL-2 collision loads applied to the rail at truss member L2-U2 (Node 4) with the crush strength characteristics of the crushable pipe blockout used between the rail and the truss members. Lateral spring supports with the strength characteristics of the blockouts obtained from the LS-DYNA analyses simulated the support conditions from the pipe blockouts located at the truss members. Test Level 2 transverse loading conditions were again applied to the rail at the critical truss member L2-U2. This analysis was performed to determine the magnitude of the load transmitted to the truss member considering the crush from the crushable pipe blockouts. For the 5-inch (127 mm) crushable pipe, lateral spring supports with a stiffness of 6.7 kips per inch (1.17 kN/mm) of deflection were used. For the 6-inch (152 mm) crushable pipe, lateral spring supports with a stiffness of 7.75 kips per inch (1.36 kN/mm) of deflection were used. The reactions from the lateral spring supports at the critical vertical truss member L2-U2 and at adjacent diagonal truss member were obtained and used as point loads in a separate full three-dimensional structural analysis of the bridge structure.

From the previous analysis, considering the crushable pipe blockout strength characteristic as previously described, a reaction with a magnitude of approximately 8.7 kips was applied to member L2-U2 from the crushable pipe blockout. A separate biaxial bending and axial strength analysis was performed on member L2-U2 using the magnitude of this reaction applied to the truss member. The calculated stress ratio (ratio of biaxial bending and axial forces applied to the member to the nominal strengths of the member) was less than 1.0. The results from these analyses are included herein. Based on the results from these analyses, the crushable pipe blockouts did serve to minimize the collision loading applied to truss member L2-U2. Therefore, it is recommended that crushable pipe blockouts support the rail at all truss member locations. When the crushable pipe blockouts support the rail, the truss members have sufficient capacity to resist the dead loads from the structure and the crash loads from TL-2 loading conditions.

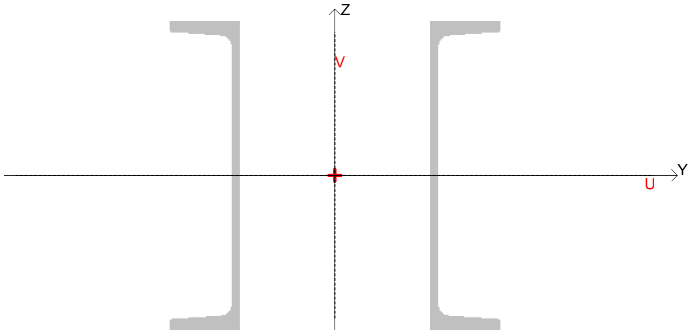
State Highway 16, Roy B. Inks Bridge over Llano River

Truss Member Section Properties

for

Three-Dimensional RISA-3D Structural Model

Members M155 & M156 (L0-L2) Bott. Chord



Section element	Rotation angle	Mirror	Material	E (kip/inch^2)
American Standard Channels C15X33.9			Steel	29732.747
American Standard Channels C15X33.9		+	Steel	29732.747

The overall dimensions of the section are 16.047 x 15.0 inch

Basic geometry of the section

	Parameter	Value	
A	Sectional area	19.915	$inch^2$
α	Angle of principal inertia axes	0.0	deg
I_y	Inertia moment about centroidal Y1-axis parallel with Y-axis	630.0	$inch^4$
I_z	Inertia moment about centroidal Z1-axis parallel with Z-axis	599.567	$inch^4$
I_t	Torsional moment of inertia (St. Venant)	2.04	$inch^4$
i_y	Radius of inertia about Y1-axis	5.624	inch
i_z	Radius of inertia about Z1-axis	5.487	inch
W_{u+}	Maximum resisting moment about U-axis	84.0	$inch^3$
W_{u-}	Minimum resisting moment about U-axis	84.0	$inch^3$
W_{v+}	Maximum resisting moment about V-axis	74.712	$inch^3$
W_{v-}	Minimum resisting moment about V-axis	74.712	$inch^3$
$W_{pl,u}$	Plastic resisting moment about U-axis	101.346	$inch^3$
$W_{pl,v}$	Plastic resisting moment about V-axis	108.317	$inch^3$
I_u	Maximum inertia moment	630.0	$inch^4$
I_v	Minimum inertia moment	599.567	$inch^4$
i_u	Maximum radius of inertia	5.624	inch
i_v	Minimum radius of inertia	5.487	inch
a_{u+}	Middle point along positive direction of Y(U)-axis	3.752	inch
a_{u-}	Middle point along negative direction of Y(U)-axis	3.752	inch
a_{v+}	Middle point along positive direction of Z(V)-axis	4.218	inch
a_{v-}	Middle point along negative direction of Z(V)-axis	4.218	inch
y_M	Coordinate of the center of gravity along Y-axis	0.0	inch
z_M	Coordinate of the center of gravity along Z-axis	0.0	inch

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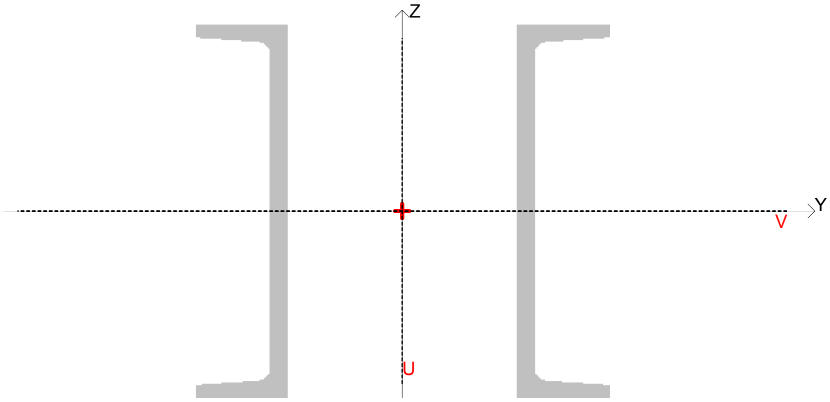
Member M187 (L2-L3) Bott. Chord



Section element	Rotation angle	Mirror	Material	E (kip/inch^2)
American Standard Channels C15X50		+	Steel	29732.747
American Standard Channels C15X50			Steel	29732.747

The overall dimensions of the section are 16.646 x 15.0 inch

Basic geometry of the section

	Parameter	Value	
A	Sectional area	29.393	$inch^2$
α	Angle of principal inertia axes	-90.0	deg
I_y	Inertia moment about centroidal Y1-axis parallel with Y-axis	808.0	$inch^4$
I_z	Inertia moment about centroidal Z1-axis parallel with Z-axis	881.313	$inch^4$
I_t	Torsional moment of inertia (St. Venant)	5.34	$inch^4$
i_y	Radius of inertia about Y1-axis	5.243	inch
i_z	Radius of inertia about Z1-axis	5.476	inch
W_{u+}	Maximum resisting moment about U-axis	105.863	$inch^3$
W_{u-}	Minimum resisting moment about U-axis	105.863	$inch^3$
W_{v+}	Maximum resisting moment about V-axis	107.733	$inch^3$
W_{v-}	Minimum resisting moment about V-axis	107.733	$inch^3$
$W_{pl,u}$	Plastic resisting moment about U-axis	158.459	$inch^3$
$W_{pl,v}$	Plastic resisting moment about V-axis	135.936	$inch^3$
I_u	Maximum inertia moment	881.313	$inch^4$
I_v	Minimum inertia moment	808.0	$inch^4$
i_u	Maximum radius of inertia	5.476	inch
i_v	Minimum radius of inertia	5.243	inch
a_{u+}	Middle point along positive direction of Y(U)-axis	3.665	inch
a_{u-}	Middle point along negative direction of Y(U)-axis	3.665	inch
a_{v+}	Middle point along positive direction of Z(V)-axis	3.602	inch
a_{v-}	Middle point along negative direction of Z(V)-axis	3.602	inch
y_M	Coordinate of the center of gravity along Y-axis	1.35058e-015	
z_M	Coordinate of the center of gravity along Z-axis	0.0	inch

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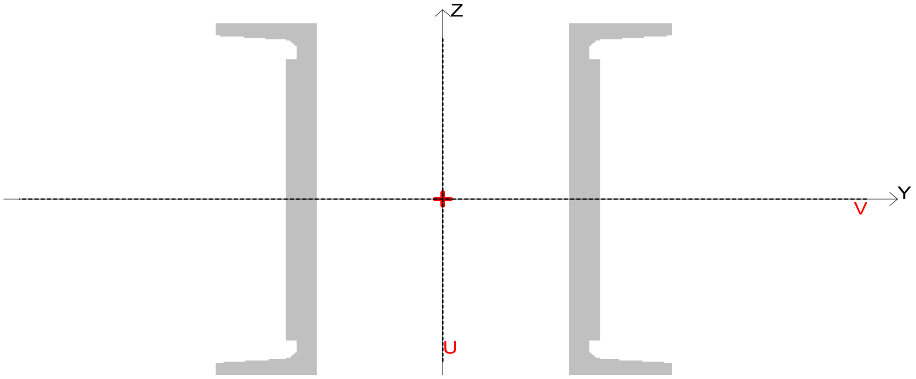
Member M181 (L3-L4) & M182 (L4-L5) Bott. Chord Members



Section element	Rotation angle	Mirror	Material	E (kip/inch^2)
American Standard Channels C15X50		+	Steel	29732.747
American Standard Channels C15X50			Steel	29732.747
Sheet 12 x 0.4375	90.0		Steel	29732.747
Sheet 12 x 0.4375	90.0		Steel	29732.747

The overall dimensions of the section are 16.646 x 15.0 inch

Basic geometry of the section

	Parameter	Value	
A	Sectional area	39.893	inch^2
α	Angle of principal inertia axes	-90.0	deg
I_y	Inertia moment about centroidal Y1-axis parallel with Y-axis	934.0	inch^4
I_z	Inertia moment about centroidal Z1-axis parallel with Z-axis	1204.178	inch^4
I_t	Torsional moment of inertia (St. Venant)	5.97	inch^4
i_y	Radius of inertia about Y1-axis	4.839	inch
i_z	Radius of inertia about Z1-axis	5.494	inch
W_{u+}	Maximum resisting moment about U-axis	144.646	inch^3
W_{u-}	Minimum resisting moment about U-axis	144.646	inch^3
W_{v+}	Maximum resisting moment about V-axis	124.533	inch^3
W_{v-}	Minimum resisting moment about V-axis	124.533	inch^3
$W_{pl,u}$	Plastic resisting moment about U-axis	216.584	inch^3
$W_{pl,v}$	Plastic resisting moment about V-axis	167.294	inch^3
I_u	Maximum inertia moment	1204.178	inch^4
I_v	Minimum inertia moment	934.0	inch^4
i_u	Maximum radius of inertia	5.494	inch
i_v	Minimum radius of inertia	4.839	inch
a_{u+}	Middle point along positive direction of Y(U)-axis	3.122	inch
a_{u-}	Middle point along negative direction of Y(U)-axis	3.122	inch
a_{v+}	Middle point along positive direction of Z(V)-axis	3.626	inch
a_{v-}	Middle point along negative direction of Z(V)-axis	3.626	inch
y_M	Coordinate of the center of gravity along Y-axis	1.16095e-015	
z_M	Coordinate of the center of gravity along Z-axis	0.0	inch

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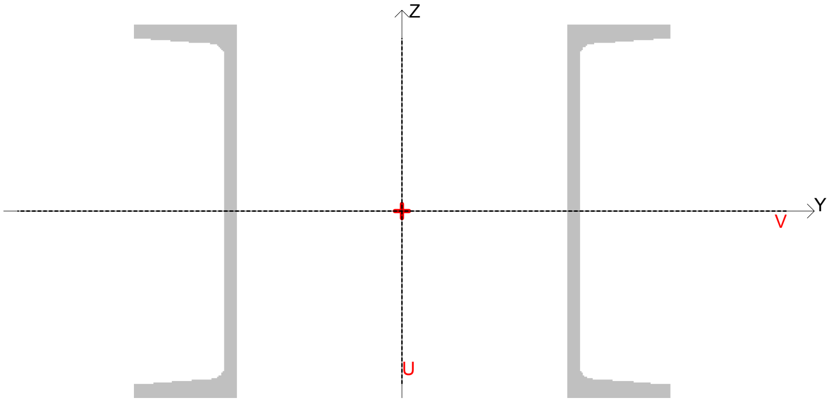
Members M143 (L3-U3) & M144 (L4-U4) Vertical Truss Members



Section element	**Rotation angle**	**Mirror**	**Material**	**E (kip/inch^2)**
American Standard Channels C9X15		+	Steel	29732.747
American Standard Channels C9X15			Steel	29732.747

The overall dimensions of the section are 12.969 x 9.0 inch

Basic geometry of the section

	Parameter	**Value**	
A	Sectional area	8.818	$inch^2$
α	Angle of principal inertia axes	-90.0	deg
I_y	Inertia moment about centroidal Y1-axis parallel with Y-axis	102.0	$inch^4$
I_z	Inertia moment about centroidal Z1-axis parallel with Z-axis	189.311	$inch^4$
I_t	Torsional moment of inertia (St. Venant)	0.42	$inch^4$
i_y	Radius of inertia about Y1-axis	3.401	inch
i_z	Radius of inertia about Z1-axis	4.633	inch
W_{u+}	Maximum resisting moment about U-axis	29.192	$inch^3$
W_{u-}	Minimum resisting moment about U-axis	29.192	$inch^3$
W_{v+}	Maximum resisting moment about V-axis	22.667	$inch^3$
W_{v-}	Minimum resisting moment about V-axis	22.667	$inch^3$
$W_{pl,u}$	Plastic resisting moment about U-axis	40.35	$inch^3$
$W_{pl,v}$	Plastic resisting moment about V-axis	27.132	$inch^3$
I_u	Maximum inertia moment	189.311	$inch^4$
I_v	Minimum inertia moment	102.0	$inch^4$
i_u	Maximum radius of inertia	4.633	inch
i_v	Minimum radius of inertia	3.401	inch
a_{u+}	Middle point along positive direction of Y(U)-axis	2.571	inch
a_{u-}	Middle point along negative direction of Y(U)-axis	2.571	inch
a_{v+}	Middle point along positive direction of Z(V)-axis	3.311	inch
a_{v-}	Middle point along negative direction of Z(V)-axis	3.311	inch
y_M	Coordinate of the center of gravity along Y-axis	4.0	inch
z_M	Coordinate of the center of gravity along Z-axis	4.5	inch

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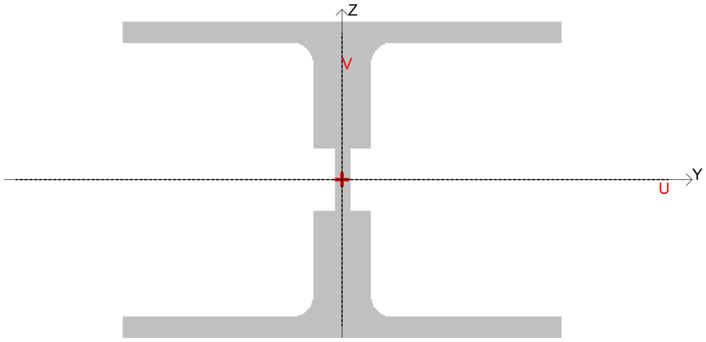
Member M266 (L2-U1) Diag. Truss Memb.



Section element	Rotation angle	Mirror	Material	E (kip/inch^2)
Unequal Angles L5X3X1/2	90.0		Steel	29732.747
Unequal Angles L5X3X1/2	270.0	+	Steel	29732.747
Unequal Angles L5X3X1/2	270.0		Steel	29732.747
Unequal Angles L5X3X1/2	90.0	+	Steel	29732.747
Sheet 7.5 x 0.375	90.0		Steel	29732.747

The overall dimensions of the section are 10.37 x 7.496 inch

Basic geometry of the section

	Parameter	Value	
A	Sectional area	17.809	inch^2
α	Angle of principal inertia axes	0.0	deg
I_y	Inertia moment about centroidal Y1-axis parallel with Y-axis	157.65	inch^4
I_z	Inertia moment about centroidal Z1-axis parallel with Z-axis	94.947	inch^4
I_t	Torsional moment of inertia (St. Venant)	1.243	inch^4
i_y	Radius of inertia about Y1-axis	2.975	inch
i_z	Radius of inertia about Z1-axis	2.309	inch
W_{u+}	Maximum resisting moment about U-axis	42.04	inch^3
W_{u-}	Minimum resisting moment about U-axis	42.04	inch^3
W_{v+}	Maximum resisting moment about V-axis	18.303	inch^3
W_{v-}	Minimum resisting moment about V-axis	18.303	inch^3
$W_{pl,u}$	Plastic resisting moment about U-axis	50.851	inch^3
$W_{pl,v}$	Plastic resisting moment about V-axis	29.454	inch^3
I_u	Maximum inertia moment	157.65	inch^4
I_v	Minimum inertia moment	94.947	inch^4
i_u	Maximum radius of inertia	2.975	inch
i_v	Minimum radius of inertia	2.309	inch
a_{u+}	Middle point along positive direction of Y(U)-axis	1.028	inch
a_{u-}	Middle point along negative direction of Y(U)-axis	1.028	inch
a_{v+}	Middle point along positive direction of Z(V)-axis	2.361	inch
a_{v-}	Middle point along negative direction of Z(V)-axis	2.361	inch
y_M	Coordinate of the center of gravity along Y-axis	9.6386e-017	inch
z_M	Coordinate of the center of gravity along Z-axis	-8.50588e-019	

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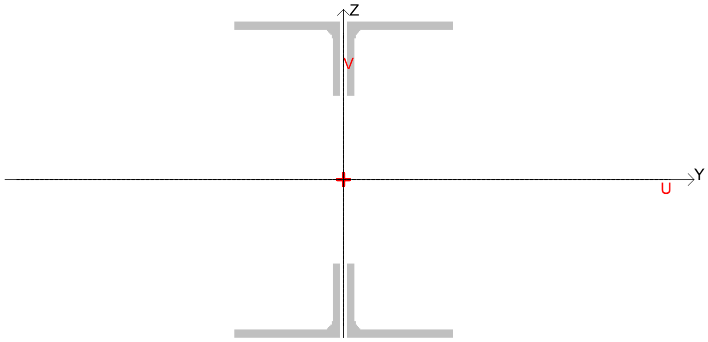
Typical Top Sway Bracing Member (Multiple)



Section element	Rotation angle	Mirror	Material	E (kip/inch^2)
Unequal Angles L5X3-1/2X3/8	90.0	+	Steel	29732.747
Unequal Angles L5X3-1/2X3/8	90.0		Steel	29732.747
Unequal Angles L5X3-1/2X3/8	270.0	+	Steel	29732.747
Unequal Angles L5X3-1/2X3/8	270.0		Steel	29732.747

The overall dimensions of the section are 10.307 x 15.0 inch

Basic geometry of the section

	Parameter	Value	
A	Sectional area	12.197	inch^2
α	Angle of principal inertia axes	0.0	deg
I_y	Inertia moment about centroidal Y1-axis parallel with Y-axis	549.141	inch^4
I_z	Inertia moment about centroidal Z1-axis parallel with Z-axis	69.759	inch^4
I_t	Torsional moment of inertia (St. Venant)	0.533	inch^4
i_y	Radius of inertia about Y1-axis	6.71	inch
i_z	Radius of inertia about Z1-axis	2.392	inch
W_{u+}	Maximum resisting moment about U-axis	73.219	inch^3
W_{u-}	Minimum resisting moment about U-axis	73.219	inch^3
W_{v+}	Maximum resisting moment about V-axis	13.529	inch^3
W_{v-}	Minimum resisting moment about V-axis	13.529	inch^3
$W_{pl,u}$	Plastic resisting moment about U-axis	82.788	inch^3
$W_{pl,v}$	Plastic resisting moment about V-axis	21.789	inch^3
I_u	Maximum inertia moment	549.141	inch^4
I_v	Minimum inertia moment	69.759	inch^4
i_u	Maximum radius of inertia	6.71	inch
i_v	Minimum radius of inertia	2.392	inch
a_{u+}	Middle point along positive direction of Y(U)-axis	1.109	inch
a_{u-}	Middle point along negative direction of Y(U)-axis	1.109	inch
a_{v+}	Middle point along positive direction of Z(V)-axis	6.003	inch
a_{v-}	Middle point along negative direction of Z(V)-axis	6.003	inch
y_M	Coordinate of the center of gravity along Y-axis	0.0	inch
z_M	Coordinate of the center of gravity along Z-axis	0.0	inch

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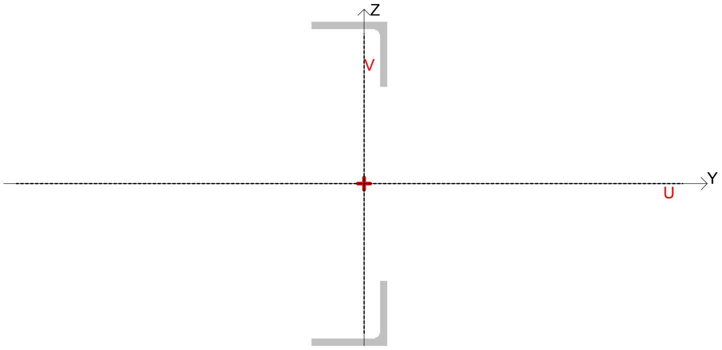
Typical Top Sway Brace Member (Multiple)



Section element	Rotation angle	Mirror	Material	E (kip/inch^2)
Unequal Angles L3-1/2X3X5/16	90.0	+	Steel	29732.747
Unequal Angles L3-1/2X3X5/16	90.0		Steel	29732.747

The overall dimensions the section are 3.496 x 15.0 inch

Basic geometry of the section

	Parameter	Value	
A	Sectional area	3.859	inch^2
α	Angle of principal inertia axes	0.0	deg
I_y	Inertia moment about centroidal Y1-axis parallel with Y-axis	175.468	inch^4
I_z	Inertia moment about centroidal Z1-axis parallel with Z-axis	4.692	inch^4
I_t	Torsional moment of inertia (St. Venant)	0.118	inch^4
i_y	Radius of inertia about Y1-axis	6.743	inch
i_z	Radius of inertia about Z1-axis	1.103	inch
W_{u+}	Maximum resisting moment about U-axis	23.396	inch^3
W_{u-}	Minimum resisting moment about U-axis	23.396	inch^3
W_{v+}	Maximum resisting moment about V-axis	1.93	inch^3
W_{v-}	Minimum resisting moment about V-axis	4.39	inch^3
$W_{pl,u}$	Plastic resisting moment about U-axis	26.541	inch^3
$W_{pl,v}$	Plastic resisting moment about V-axis	3.879	inch^3
I_u	Maximum inertia moment	175.468	inch^4
I_v	Minimum inertia moment	4.692	inch^4
i_u	Maximum radius of inertia	6.743	inch
i_v	Minimum radius of inertia	1.103	inch
a_{u+}	Middle point along positive direction of Y(U)-axis	0.5	inch
a_{u-}	Middle point along negative direction of Y(U)-axis	1.138	inch
a_{v+}	Middle point along positive direction of Z(V)-axis	6.063	inch
a_{v-}	Middle point along negative direction of Z(V)-axis	6.063	inch
y_M	Coordinate of the center of gravity along Y-axis	-1.36592e-016	
z_M	Coordinate of the center of gravity along Z-axis	0.0	inch

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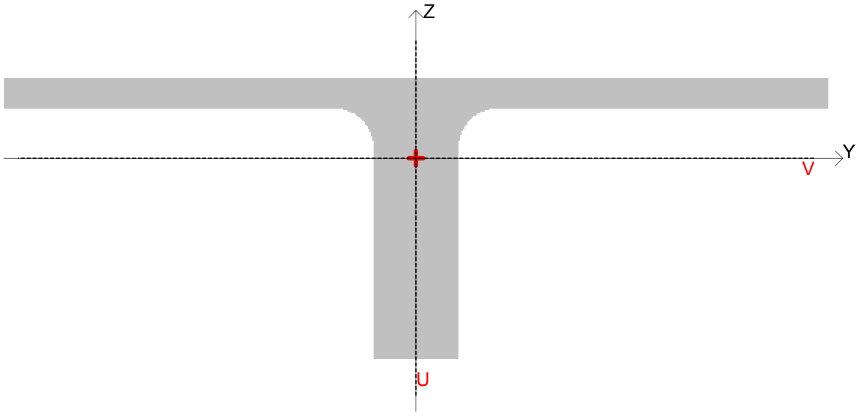
Top Sway Brace Member (Multiple)



Section element	Rotation angle	Mirror	Material	E (kip/inch^2)
Unequal Angles L5X3-1/2X3/8	90.0	+	Steel	29732.747
Unequal Angles L5X3-1/2X3/8	270.0		Steel	29732.747
Sheet 3.5 x 0.3125	90.0		Steel	29732.747

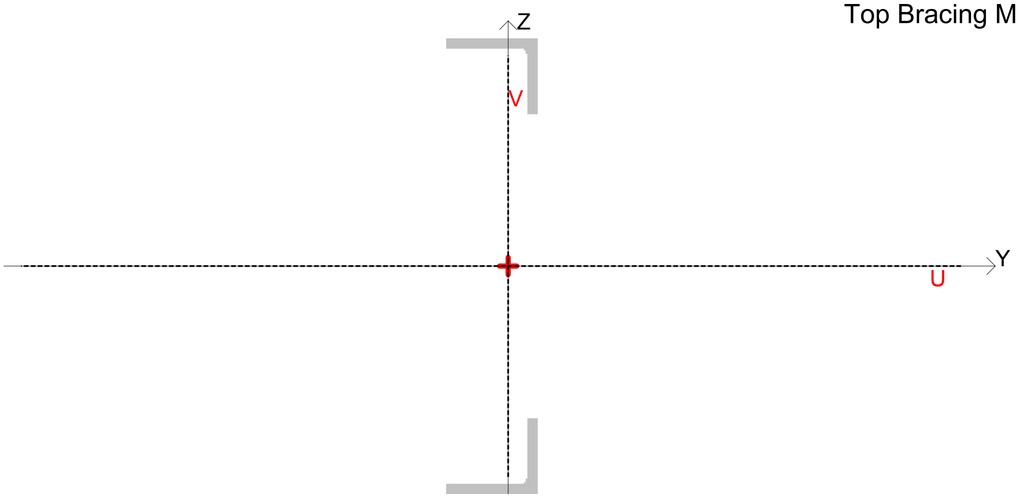
The overall dimensions of the section are 10.307 x 3.496 inch

Basic geometry of the section

	Parameter	Value	
A	Sectional area	7.192	inch^2
α	Angle of principal inertia axes	-90.0	deg
I_y	Inertia moment about centroidal Y1-axis parallel with Y-axis	8.215	inch^4
I_z	Inertia moment about centroidal Z1-axis parallel with Z-axis	34.889	inch^4
I_t	Torsional moment of inertia (St. Venant)	0.3	inch^4
i_y	Radius of inertia about Y1-axis	1.069	inch
i_z	Radius of inertia about Z1-axis	2.202	inch
W_{u+}	Maximum resisting moment about U-axis	6.766	inch^3
W_{u-}	Minimum resisting moment about U-axis	6.766	inch^3
W_{v+}	Maximum resisting moment about V-axis	3.289	inch^3
W_{v-}	Minimum resisting moment about V-axis	8.194	inch^3
$W_{pl,u}$	Plastic resisting moment about U-axis	10.871	inch^3
$W_{pl,v}$	Plastic resisting moment about V-axis	6.574	inch^3
I_u	Maximum inertia moment	34.889	inch^4
I_v	Minimum inertia moment	8.215	inch^4
i_u	Maximum radius of inertia	2.202	inch
i_v	Minimum radius of inertia	1.069	inch
a_{u+}	Middle point along positive direction of Y(U)-axis	0.457	inch
a_{u-}	Middle point along negative direction of Y(U)-axis	1.139	inch
a_{v+}	Middle point along positive direction of Z(V)-axis	0.941	inch
a_{v-}	Middle point along negative direction of Z(V)-axis	0.941	inch
y_M	Coordinate of the center of gravity along Y-axis	7.21437e-018	
z_M	Coordinate of the center of gravity along Z-axis	1.43736e-017	

File: T:\2002-2003\444192\LlanoBrige_Sections\2L5x3.5x0.375_swayBracing.sec

Top Bracing Member (Multiple)



Section element	Rotation angle	Mirror	Material	E (kip/inch^2)
Unequal Angles L3X2-1/2X5/16	90.0	+	Steel	29732.747
Unequal Angles L3X2-1/2X5/16	90.0		Steel	29732.747

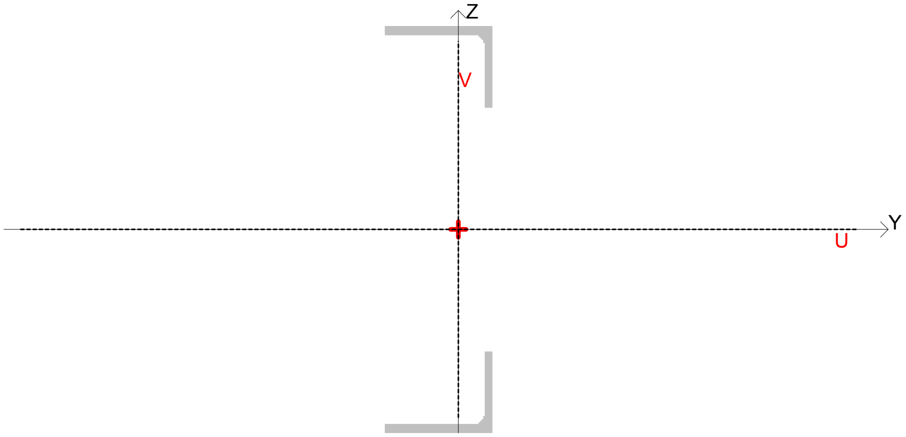
The overall dimensions of the section are 2.996 x 15.0 inch

Basic geometry of the section

	Parameter	Value	
A	Sectional area	3.239	$inch^2$
α	Angle of principal inertia axes	0.0	deg
I_y	Inertia moment about centroidal Y1-axis parallel with Y-axis	152.067	$inch^4$
I_z	Inertia moment about centroidal Z1-axis parallel with Z-axis	2.857	$inch^4$
I_t	Torsional moment of inertia (St. Venant)	0.097	$inch^4$
i_y	Radius of inertia about Y1-axis	6.852	inch
i_z	Radius of inertia about Z1-axis	0.939	inch
W_{u+}	Maximum resisting moment about U-axis	20.276	$inch^3$
W_{u-}	Minimum resisting moment about U-axis	20.276	$inch^3$
W_{v+}	Maximum resisting moment about V-axis	1.386	$inch^3$
W_{v-}	Minimum resisting moment about V-axis	3.042	$inch^3$
$W_{pl,u}$	Plastic resisting moment about U-axis	22.468	$inch^3$
$W_{pl,v}$	Plastic resisting moment about V-axis	2.731	$inch^3$
I_u	Maximum inertia moment	152.067	$inch^4$
I_v	Minimum inertia moment	2.857	$inch^4$
i_u	Maximum radius of inertia	6.852	inch
i_v	Minimum radius of inertia	0.939	inch
a_{u+}	Middle point along positive direction of Y(U)-axis	0.428	inch
a_{u-}	Middle point along negative direction of Y(U)-axis	0.939	inch
a_{v+}	Middle point along positive direction of Z(V)-axis	6.259	inch
a_{v-}	Middle point along negative direction of Z(V)-axis	6.259	inch
y_M	Coordinate of the center of gravity along Y-axis	0.0	inch
z_M	Coordinate of the center of gravity along Z-axis	0.0	inch

File: T:\2002-2003\444192\LlanoBrige_Sections\2L3X2.5X0.3125_Tension.sec

Portal Bracing Member (Multiple)



Section element	Rotation angle	Mirror	Material	E (kip/inch^2)
Unequal Angles L4X3X5/16	90.0		Steel	29732.747
Unequal Angles L4X3X5/16	90.0	+	Steel	29732.747

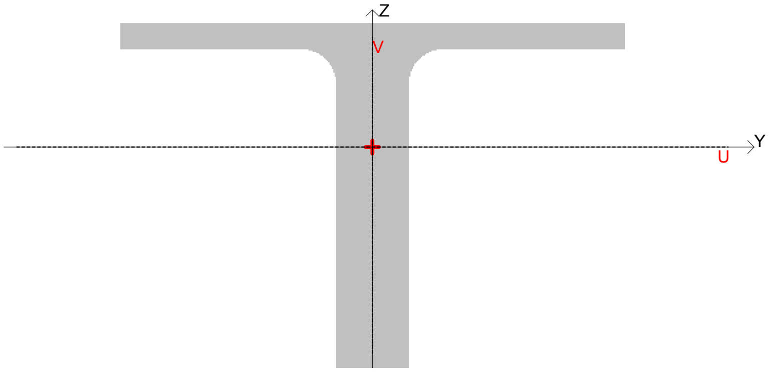
The overall dimensions of the section are 3.996 x 15.0 inch

Basic geometry of the section

	Parameter	Value	
A	Sectional area	4.179	inch^2
α	Angle of principal inertia axes	0.0	deg
I_y	Inertia moment about centroidal Y1-axis parallel with Y-axis	192.73	inch^4
I_z	Inertia moment about centroidal Z1-axis parallel with Z-axis	6.819	inch^4
I_t	Torsional moment of inertia (St. Venant)	0.127	inch^4
i_y	Radius of inertia about Y1-axis	6.791	inch
i_z	Radius of inertia about Z1-axis	1.277	inch
W_{u+}	Maximum resisting moment about U-axis	25.697	inch^3
W_{u-}	Minimum resisting moment about U-axis	25.697	inch^3
W_{v+}	Maximum resisting moment about V-axis	2.495	inch^3
W_{v-}	Minimum resisting moment about V-axis	5.38	inch^3
$W_{pl,u}$	Plastic resisting moment about U-axis	28.854	inch^3
$W_{pl,v}$	Plastic resisting moment about V-axis	4.86	inch^3
I_u	Maximum inertia moment	192.73	inch^4
I_v	Minimum inertia moment	6.819	inch^4
i_u	Maximum radius of inertia	6.791	inch
i_v	Minimum radius of inertia	1.277	inch
a_{u+}	Middle point along positive direction of Y(U)-axis	0.597	inch
a_{u-}	Middle point along negative direction of Y(U)-axis	1.288	inch
a_{v+}	Middle point along positive direction of Z(V)-axis	6.149	inch
a_{v-}	Middle point along negative direction of Z(V)-axis	6.149	inch
y_M	Coordinate of the center of gravity along Y-axis	0.0	inch
z_M	Coordinate of the center of gravity along Z-axis	0.0	inch

File: T:\2002-2003\444192\LlanoBrige_Sections\2L4X3X5_portal.sec

Typical Intermediate Rail Post



Section element	**Rotation angle**	**Mirror**	**Material**	**E (kip/inch^2)**
Unequal Angles L5X3-1/2X3/8			Steel	29732.747
Unequal Angles L5X3-1/2X3/8		+	Steel	29732.747
Sheet 5 x 0.3125	90.0		Steel	29732.747

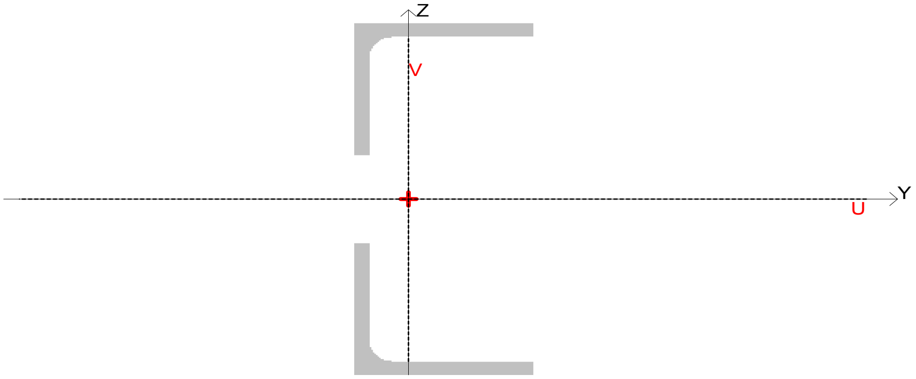
The overall dimensions of the section are 7.307 x 4.996 inch

Basic geometry of the section

	Parameter	**Value**	
A	Sectional area	7.661	$inch^2$
α	Angle of principal inertia axes	0.0	deg
I_y	Inertia moment about centroidal Y1-axis parallel with Y-axis	19.907	$inch^4$
I_z	Inertia moment about centroidal Z1-axis parallel with Z-axis	12.794	$inch^4$
I_t	Torsional moment of inertia (St. Venant)	0.314	$inch^4$
i_y	Radius of inertia about Y1-axis	1.612	inch
i_z	Radius of inertia about Z1-axis	1.292	inch
W_{u+}	Maximum resisting moment about U-axis	6.217	$inch^3$
W_{u-}	Minimum resisting moment about U-axis	11.073	$inch^3$
W_{v+}	Maximum resisting moment about V-axis	3.499	$inch^3$
W_{v-}	Minimum resisting moment about V-axis	3.499	$inch^3$
$W_{pl,u}$	Plastic resisting moment about U-axis	10.81	$inch^3$
$W_{pl,v}$	Plastic resisting moment about V-axis	6.349	$inch^3$
I_u	Maximum inertia moment	19.907	$inch^4$
I_v	Minimum inertia moment	12.794	$inch^4$
i_u	Maximum radius of inertia	1.612	inch
i_v	Minimum radius of inertia	1.292	inch
a_{u+}	Middle point along positive direction of Y(U)-axis	0.457	inch
a_{u-}	Middle point along negative direction of Y(U)-axis	0.457	inch
a_{v+}	Middle point along positive direction of Z(V)-axis	0.811	inch
a_{v-}	Middle point along negative direction of Z(V)-axis	1.445	inch
y_M	Coordinate of the center of gravity along Y-axis	-4.54181e-018	
z_M	Coordinate of the center of gravity along Z-axis	2.15906e-016	

File: T:\2002-2003\444192\LlanoBrige_Sections\2L5X3.5X0.375_PL0.3125_Railpost.sec

Members M37 (L4-U5) & M38 (L5-U4) Diagonal Truss Members
& Floor Bracing Member (Multiple)



Section element	Rotation angle	Mirror	Material	E (kip/inch^2)
Unequal Angles L3-1/2X3X5/16	-90.0		Steel	29732.747
Unequal Angles L3-1/2X3X5/16	-90.0	+	Steel	29732.747

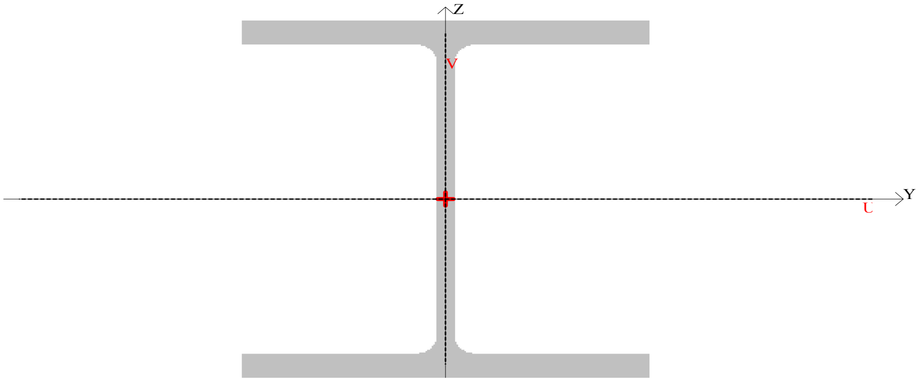
The overall dimensions of the section are 3.496 x 8.0 inch

Basic geometry of the section

	Parameter	Value	
A	Sectional area	3.859	inch^2
α	Angle of principal inertia axes	0.0	deg
I_y	Inertia moment about centroidal Y1-axis parallel with Y-axis	42.245	inch^4
I_z	Inertia moment about centroidal Z1-axis parallel with Z-axis	4.692	inch^4
I_t	Torsional moment of inertia (St. Venant)	0.118	inch^4
i_y	Radius of inertia about Y1-axis	3.309	inch
i_z	Radius of inertia about Z1-axis	1.103	inch
W_{u+}	Maximum resisting moment about U-axis	10.561	inch^3
W_{u-}	Minimum resisting moment about U-axis	10.561	inch^3
W_{v+}	Maximum resisting moment about V-axis	1.93	inch^3
W_{v-}	Minimum resisting moment about V-axis	4.39	inch^3
$W_{pl,u}$	Plastic resisting moment about U-axis	12.677	inch^3
$W_{pl,v}$	Plastic resisting moment about V-axis	3.717	inch^3
I_u	Maximum inertia moment	42.245	inch^4
I_v	Minimum inertia moment	4.692	inch^4
i_u	Maximum radius of inertia	3.309	inch
i_v	Minimum radius of inertia	1.103	inch
a_{u+}	Middle point along positive direction of Y(U)-axis	0.5	inch
a_{u-}	Middle point along negative direction of Y(U)-axis	1.138	inch
a_{v+}	Middle point along positive direction of Z(V)-axis	2.737	inch
a_{v-}	Middle point along negative direction of Z(V)-axis	2.737	inch
y_M	Coordinate of the center of gravity along Y-axis	0.0	inch
z_M	Coordinate of the center of gravity along Z-axis	0.0	inch

File: T:\2002-2003\444193\LlanoBridge_Sections\2L3.5x3x5TrussBrace.sec

Members M259 (L1-U1) Vertical, M260 (L3-U2) Diagonal
& M261 (L4-U3) Diagonal



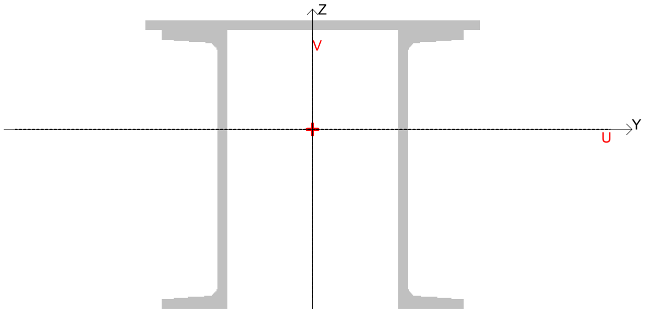
Section element	**Rotation angle**	**Mirror**	**Material**	**E (kip/inch^2)**
Wide Flange Shapes W8X40			Steel	29732.747

The overall dimensions of the section are 8.063 x 8.244 inch

Basic geometry of the section

	Parameter	**Value**	
A	Sectional area	11.697	$inch^2$
α	Angle of principal inertia axes	0.0	deg
I_y	Inertia moment about centroidal Y1-axis parallel with Y-axis	146.0	$inch^4$
I_z	Inertia moment about centroidal Z1-axis parallel with Z-axis	49.1	$inch^4$
I_t	Torsional moment of inertia (St. Venant)	1.12	$inch^4$
i_y	Radius of inertia about Y1-axis	3.533	inch
i_z	Radius of inertia about Z1-axis	2.049	inch
W_{u+}	Maximum resisting moment about U-axis	35.394	$inch^3$
W_{u-}	Minimum resisting moment about U-axis	35.394	$inch^3$
W_{v+}	Maximum resisting moment about V-axis	12.169	$inch^3$
W_{v-}	Minimum resisting moment about V-axis	12.169	$inch^3$
$W_{pl,u}$	Plastic resisting moment about U-axis	39.638	$inch^3$
$W_{pl,v}$	Plastic resisting moment about V-axis	18.435	$inch^3$
I_u	Maximum inertia moment	146.0	$inch^4$
I_v	Minimum inertia moment	49.1	$inch^4$
i_u	Maximum radius of inertia	3.533	inch
i_v	Minimum radius of inertia	2.049	inch
a_{u+}	Middle point along positive direction of Y(U)-axis	1.04	inch
a_{u-}	Middle point along negative direction of Y(U)-axis	1.04	inch
a_{v+}	Middle point along positive direction of Z(V)-axis	3.026	inch
a_{v-}	Middle point along negative direction of Z(V)-axis	3.026	inch
y_M	Coordinate of the center of gravity along Y-axis	0.0	inch
z_M	Coordinate of the center of gravity along Z-axis	0.0	inch

Member M163 (U1-U2) Top Chord



Section element	Rotation angle	Mirror	Material	E (kip/inch^2)
American Standard Channels C15X40		+	Steel	29732.747
American Standard Channels C15X40			Steel	29732.747
Sheet 18 x 0.5			Steel	29732.747

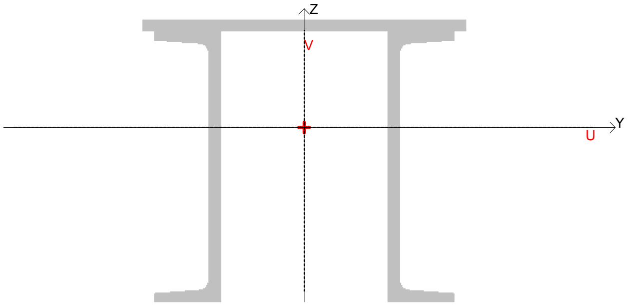
The overall dimensions of the section are 18.0 x 15.496 inch

Basic geometry of the section

	Parameter	Value	
A	Sectional area	32.594	inch^2
α	Angle of principal inertia axes	0.0	deg
I_y	Inertia moment about centroidal Y1-axis parallel with Y-axis	1089.49	inch^4
I_z	Inertia moment about centroidal Z1-axis parallel with Z-axis	944.885	inch^4
I_t	Torsional moment of inertia (St. Venant)	3.625	inch^4
i_y	Radius of inertia about Y1-axis	5.782	inch
i_z	Radius of inertia about Z1-axis	5.384	inch
W_{u+}	Maximum resisting moment about U-axis	113.018	inch^3
W_{u-}	Minimum resisting moment about U-axis	185.918	inch^3
W_{v+}	Maximum resisting moment about V-axis	104.987	inch^3
W_{v-}	Minimum resisting moment about V-axis	104.987	inch^3
$W_{pl,u}$	Plastic resisting moment about U-axis	169.82	inch^3
$W_{pl,v}$	Plastic resisting moment about V-axis	166.789	inch^3
I_u	Maximum inertia moment	1089.49	inch^4
I_v	Minimum inertia moment	944.885	inch^4
i_u	Maximum radius of inertia	5.782	inch
i_v	Minimum radius of inertia	5.384	inch
a_{u+}	Middle point along positive direction of Y(U)-axis	3.221	inch
a_{u-}	Middle point along negative direction of Y(U)-axis	3.221	inch
a_{v+}	Middle point along positive direction of Z(V)-axis	3.467	inch
a_{v-}	Middle point along negative direction of Z(V)-axis	5.704	inch
y_M	Coordinate of the center of gravity along Y-axis	-6.81223e-016	
z_M	Coordinate of the center of gravity along Z-axis	-4.05975e-016	

File: T:\2002-2003\444192\Bridge Sections Llano\C15x45.sec

Members M167 (L0-U1) End Post & M169, M170 & M172 (U2-U5) Top Chord



Section element	Rotation angle	Mirror	Material	E (kip/inch^2)
American Standard Channels C15X50		+	Steel	29732.747
American Standard Channels C15X50			Steel	29732.747
Sheet 18 x 0.625			Steel	29732.747

The overall dimensions of the section are 18.0 x 15.622 inch

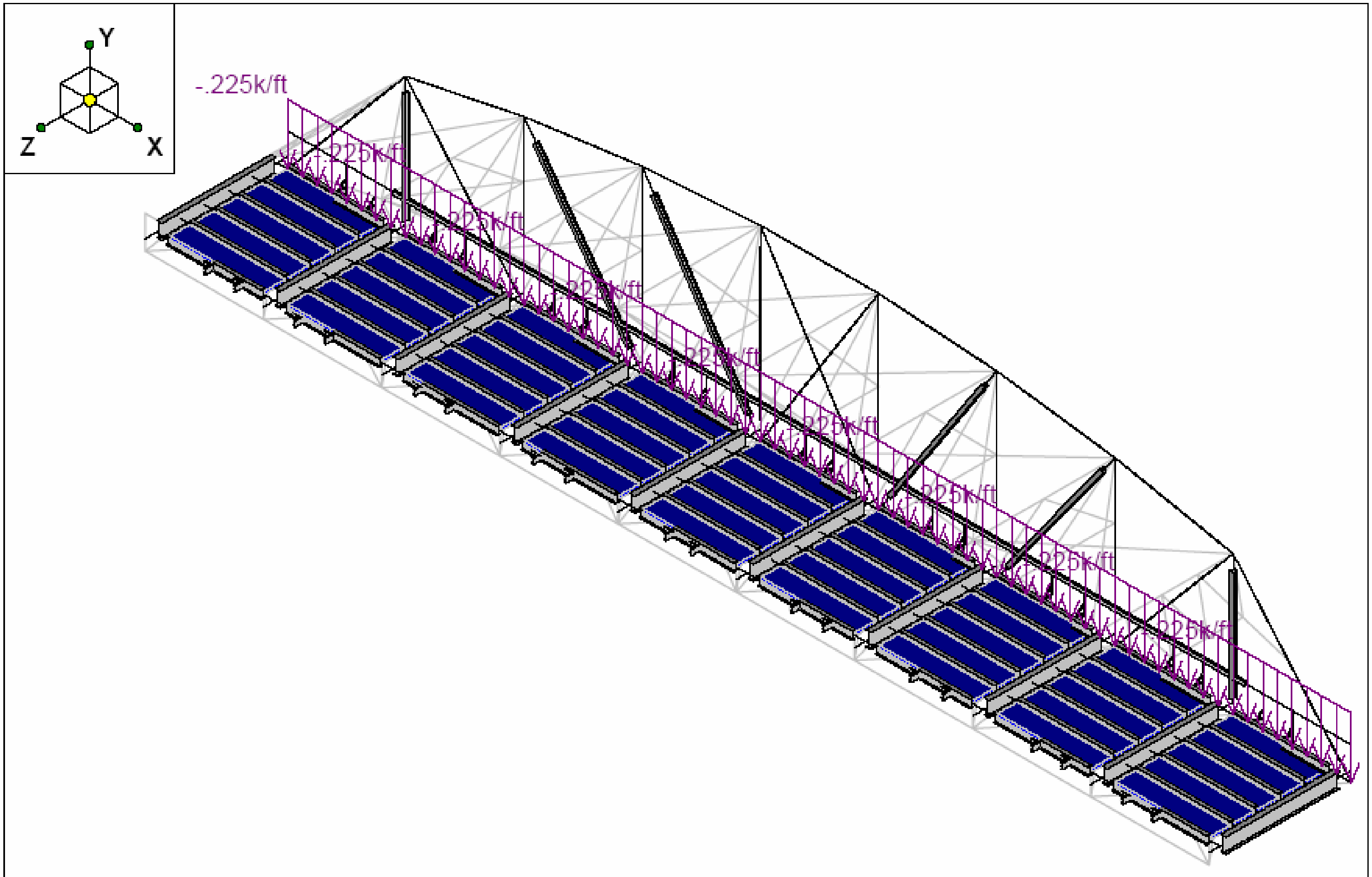
Basic geometry of the section

	Parameter	Value	
A	Sectional area	40.643	$inch^2$
α	Angle of principal inertia axes	0.0	deg
I_y	Inertia moment about centroidal Y1-axis parallel with Y-axis	1304.949	$inch^4$
I_z	Inertia moment about centroidal Z1-axis parallel with Z-axis	1190.159	$inch^4$
I_t	Torsional moment of inertia (St. Venant)	6.717	$inch^4$
i_y	Radius of inertia about Y1-axis	5.666	inch
i_z	Radius of inertia about Z1-axis	5.411	inch
W_{u+}	Maximum resisting moment about U-axis	135.052	$inch^3$
W_{u-}	Minimum resisting moment about U-axis	218.86	$inch^3$
W_{v+}	Maximum resisting moment about V-axis	132.24	$inch^3$
W_{v-}	Minimum resisting moment about V-axis	132.24	$inch^3$
$W_{pl,u}$	Plastic resisting moment about U-axis	206.458	$inch^3$
$W_{pl,v}$	Plastic resisting moment about V-axis	209.36	$inch^3$
I_u	Maximum inertia moment	1304.949	$inch^4$
I_v	Minimum inertia moment	1190.159	$inch^4$
i_u	Maximum radius of inertia	5.666	inch
i_v	Minimum radius of inertia	5.411	inch
a_{u+}	Middle point along positive direction of Y(U)-axis	3.254	inch
a_{u-}	Middle point along negative direction of Y(U)-axis	3.254	inch
a_{v+}	Middle point along positive direction of Z(V)-axis	3.323	inch
a_{v-}	Middle point along negative direction of Z(V)-axis	5.385	inch
y_M	Coordinate of the center of gravity along Y-axis	-1.03975e-016	
z_M	Coordinate of the center of gravity along Z-axis	-3.25579e-016	

File: T:\2002-2003\444192\Bridge Sections Llano\C15x50.sec

State Highway 16, Roy B. Inks Bridge over Llano River

Three-Dimensional RISA-3D Structural Model and Analysis Results



Loads: BLC 1, Dead Load

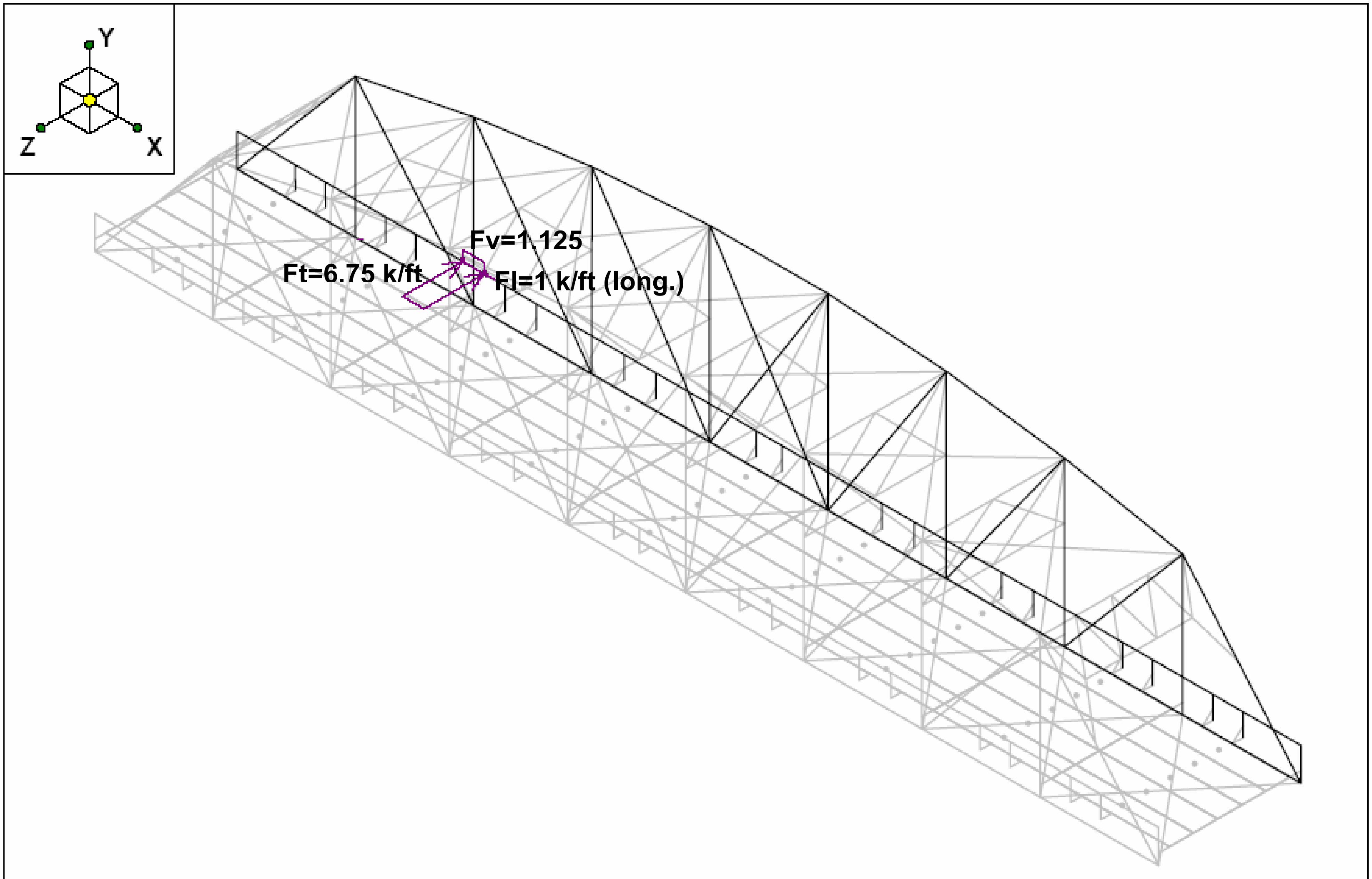
Llano Truss Bridge Structural Analysis

William Williams, P.E.

June 10, 2005 at 10:16 AM

Dead Loads = 7" Thk. Slab (Blue Plates) + Wt. Members + Curb Wt.

Llano_RisaModel4.r3d



Loads: BLC 2, Impact Load

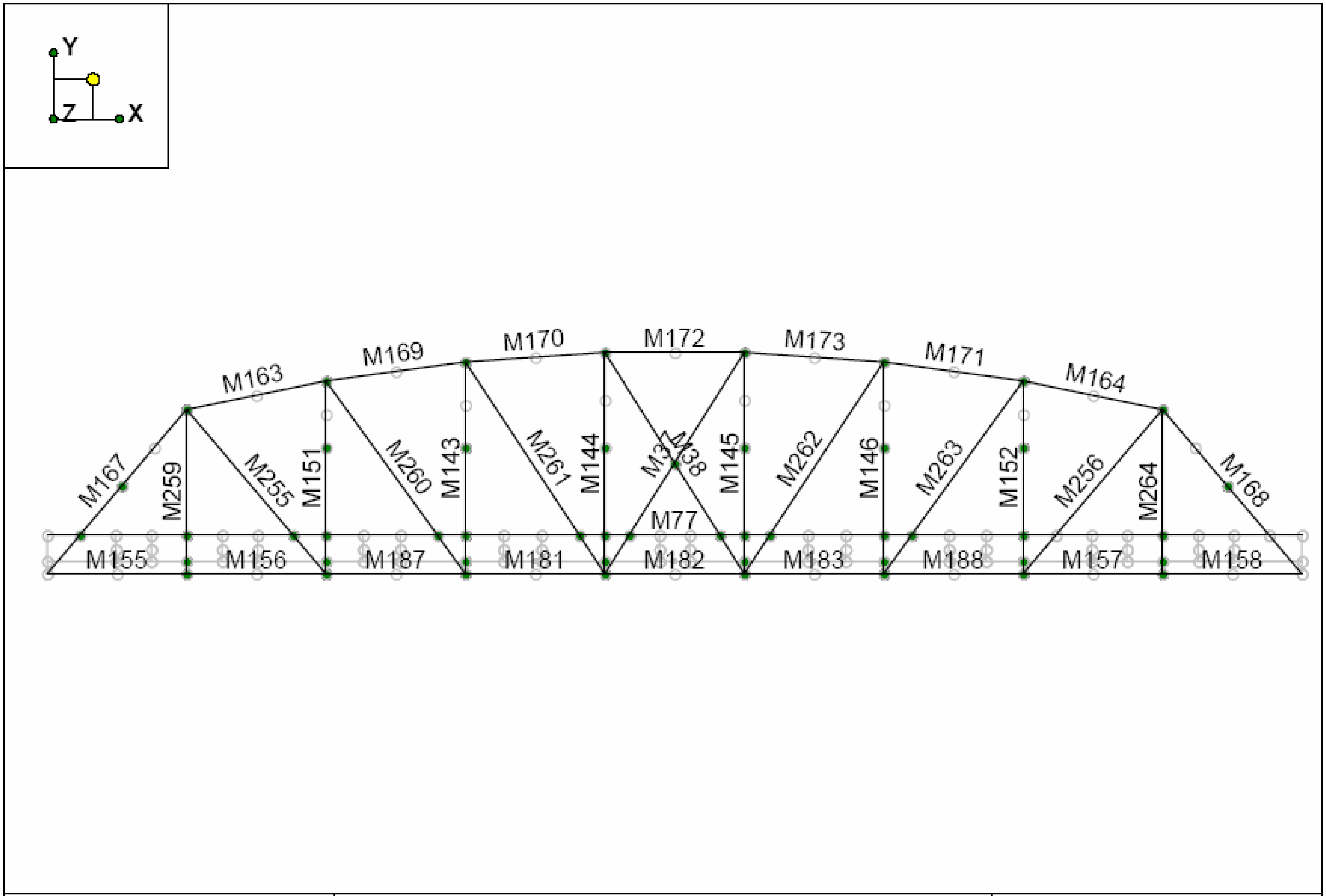
Llano Truss Bridge Structural Analysis

William Williams, P.E.

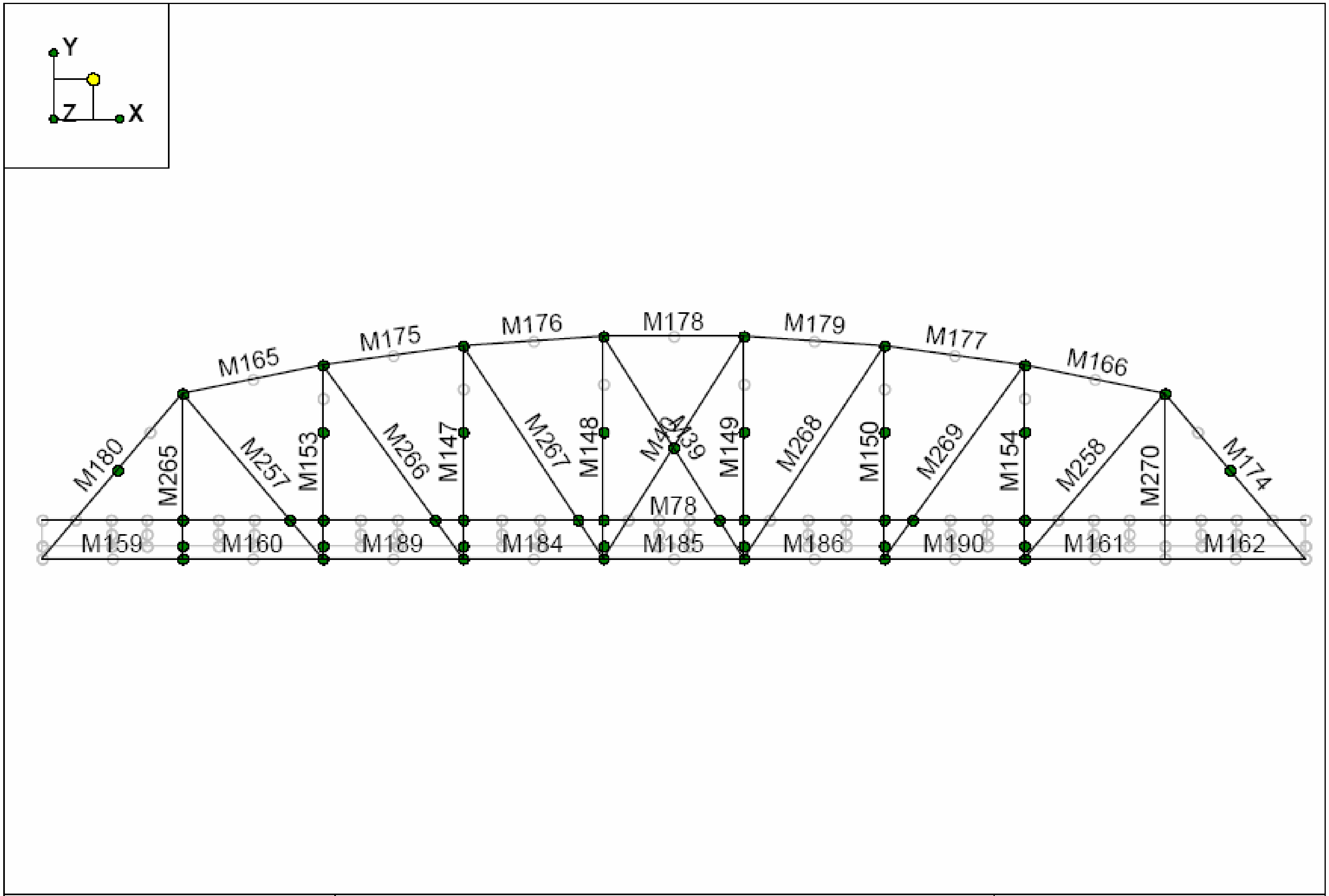
June 10, 2005 at 9:48 AM

6.75 KIPS/FT OVER 4 FT. = 27 KIPS @ MEMBER L2-U2

Llano_RisaModel4.r3d



	Llano Truss Bridge Structural Analysis	
William Williams, P.E.		Oct 5, 2005 at 4:11 PM
	Member Numbers (Loaded Left Truss)	Llano_RisaModel4.r3d



William Williams, P.E.

Llano Truss Bridge Structural Analysis

Member Numbers (Non-loaded Right Truss)

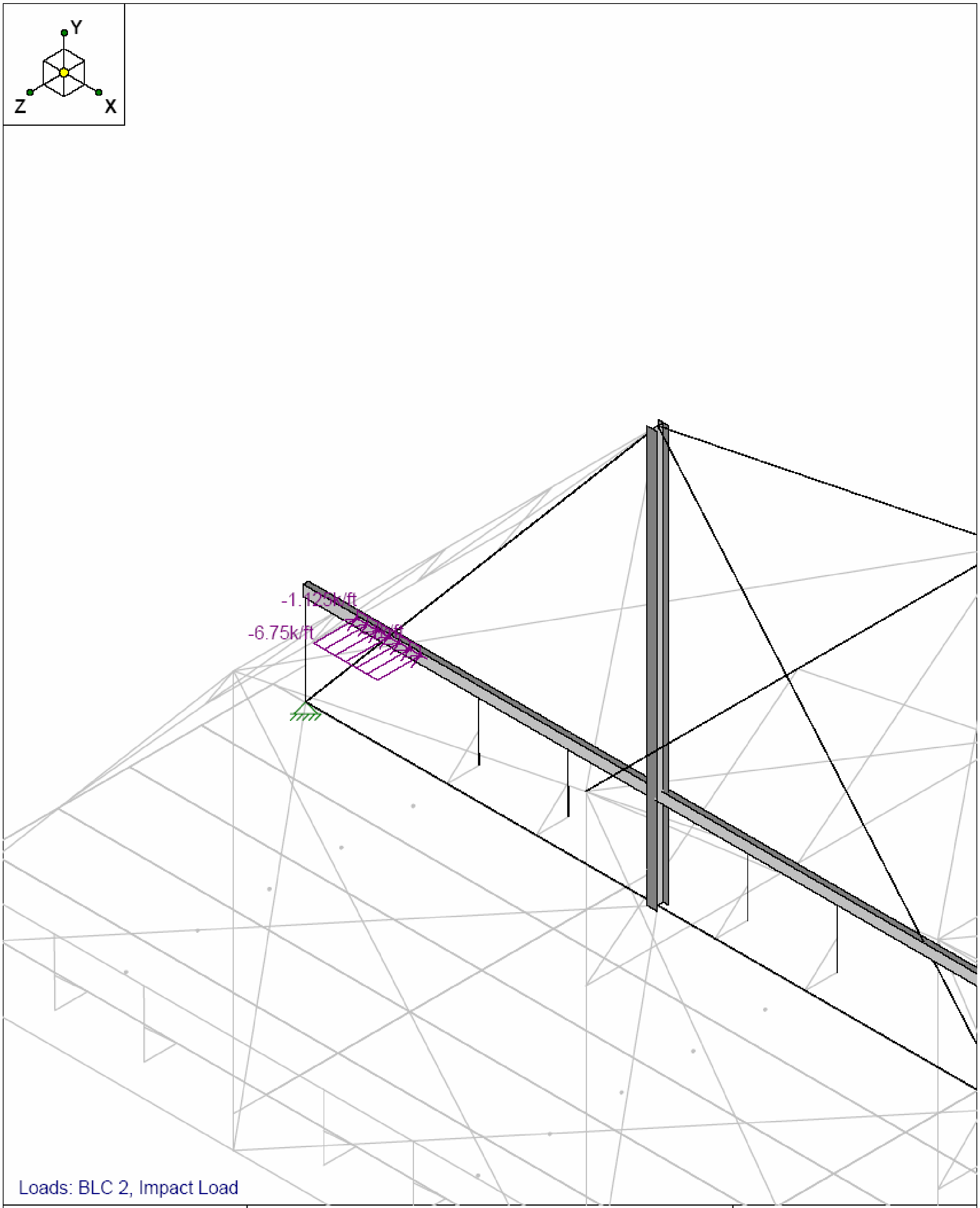
Oct 5, 2005 at 4:24 PM

Llano_RisaModel4.r3d

State Highway 16, Roy B. Inks Bridge over Llano River

Three-Dimensional RISA-3D Structural Model and Analysis Results

Load Applied at First Truss Member (Node 1)



Loads: BLC 2, Impact Load

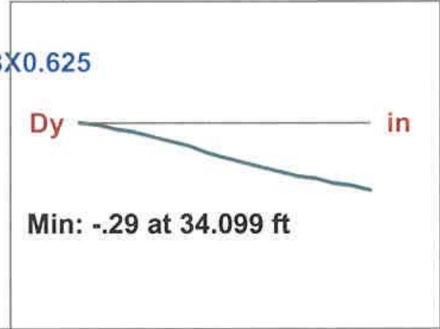
	Llano Truss Bridge Structural Analysis	
William Williams, P.E.		June 10, 2005 at 3:46 PM
	TL-2 Impact Loads on Member L0-U1(Memb. M167)	Llano_RisaModel1.r3d

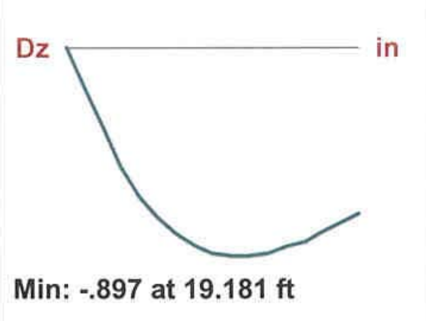
Member: *M167*

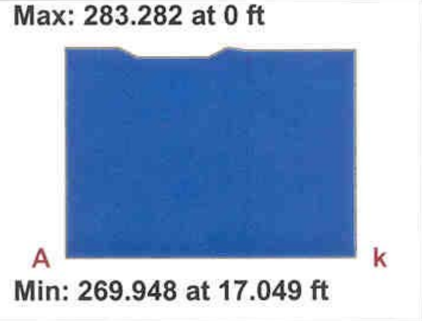
Shape: Llano_2C15X50 _PL18X0.625
Length: 34.099 ft
I Joint: N1
J Joint: N102

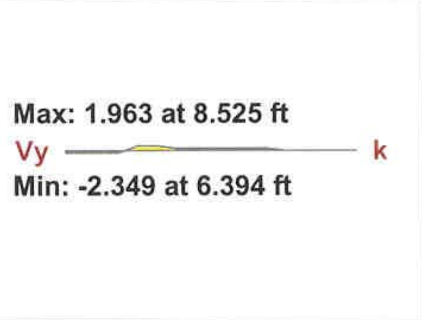
LC 1: Impact + 1.25*DL

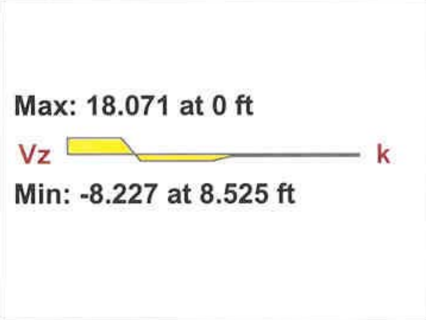
Code Check: No Calc
Report Based On 17 Sections

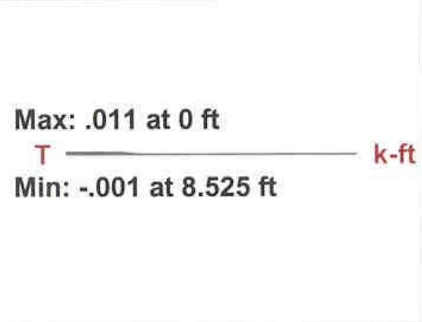


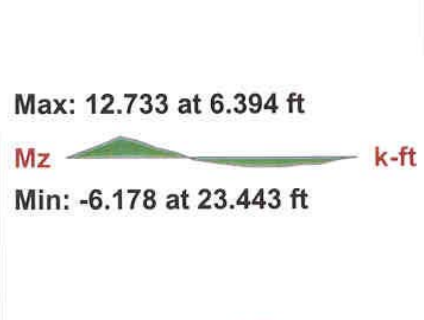


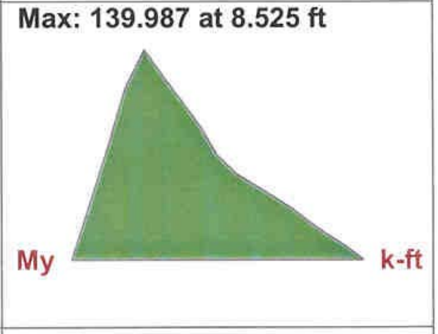


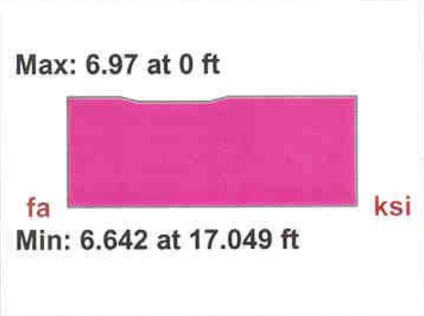


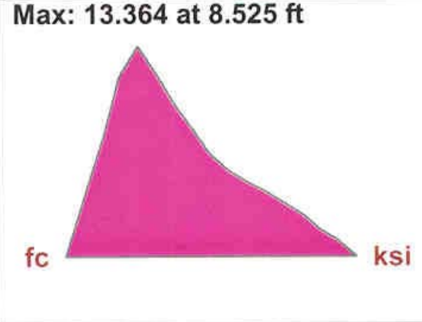


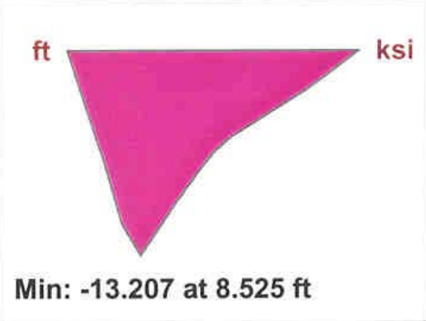












AISC LRFD 2nd Ed. Code Check

- Steel code check not calculated -

Max Defl Ratio L/753

TEXAS TRANSPORTATION INSTITUTE

Texas Transportation Institute
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College Station, Texas

PAGE *1 of 2*
JOB NO. *444193*
Date: *07-27-03*

SUBJECT *Llano S.H. 16 Truss Bridge*
LRFD Beam Check: NODE 1
BY: *W. Williams*
CKD:
CLIENT *TTI*

Beam Column Tension Analysis in Accordance with Section 6, LRFD

1.) Load Data:

$P_u := 283.0 \cdot \text{kips}$ ***"Compression"***

$M_z := 12.7 \cdot \text{kip} \cdot \text{ft}$

$M_y := 140.0 \cdot \text{kip} \cdot \text{ft}$

2.) Member Data: **4~C15x50 w/18.0 in x 0.625 in Plate**

$S_y := 132.24 \cdot \text{in}^3$ $S_z := 135.05 \cdot \text{in}^3$ $A_g := 40.643 \cdot \text{in}^2$

$Z_y := 209.404\text{in}^3$ $Z_z := 206.533\text{in}^3$ $\psi_c := 0.90$ ***from LRFD C6.5.4.2***

$K_y := 1.0$ $K_z := 1.0$

$L_y := 18.26\text{ft}$ $L_z := 26\text{ft}$

$r_y := 5.411\text{in}$ $r_z := 5.666\text{in}$

$E := 29000 \cdot \text{ksi}$ $F_y := 30 \cdot \text{ksi}$ $F_u := 50\text{ksi}$ $\phi_b := 0.9$ ***(Ch. "F" LRFD)***

3.) Calculate the Nominal Axial Strength Based on Section 6, LRFD

$$\lambda_y := \left(\frac{K_y \cdot L_y}{r_y}\right)^2 \cdot \frac{F_y}{E} \qquad \lambda_y = 1.696$$

$$\lambda_z := \left(\frac{K_z \cdot L_z}{r_z}\right)^2 \cdot \frac{F_y}{E} \qquad \lambda_z = 3.137$$

$$\frac{K_y \cdot L_y}{r_y} = 40.495 \qquad \frac{K_z \cdot L_z}{r_z} = 55.065$$

From LRFD 6.9.4.1-2:

$$P_n := \frac{0.88 \cdot F_y \cdot A_g}{\lambda_y} \qquad P_n = 632.496 \text{kips}$$

From LRFD 6.9.2.1-1:

$$P_r := \psi_c \cdot P_n \qquad P_r = 569.246 \text{kips}$$

SUBJECT Llano S.H. 16 Truss Bridge
LRFD Beam Check: NODE 1
CLIENT TTI

BY: W. Williams
CKD:

4.) Determine P_u / P_r Ratio:

$$\frac{P_u}{P_r} = 0.497$$ .... *therefore ....*

$M_{uZ} := M_z$

$M_{uY} := M_y$

$M_{nZ} := F_y \cdot Z_z$

$M_{nY} := F_y \cdot Z_y$

...... and ...

$$\frac{P_u}{P_r} + \frac{8}{9} \cdot \left(\frac{M_{uZ}}{\phi_b \cdot M_{nZ}} + \frac{M_{uY}}{\phi_b \cdot M_{nY}} \right) = 0.786$$

...... This is O.K. ...... less than 1.0 !!

GLOBAL DEFINITION OF UNITS:

$g = 32.174 \frac{ft}{sec^2}$

$plf \equiv \frac{lb}{ft}$

$kip \equiv 1000 \cdot lb$

$pcf \equiv \frac{lb}{ft^3}$

$psi \equiv \frac{lb}{in^2}$

$psf \equiv \frac{lb}{ft^2}$

$klf \equiv \frac{kip}{ft}$

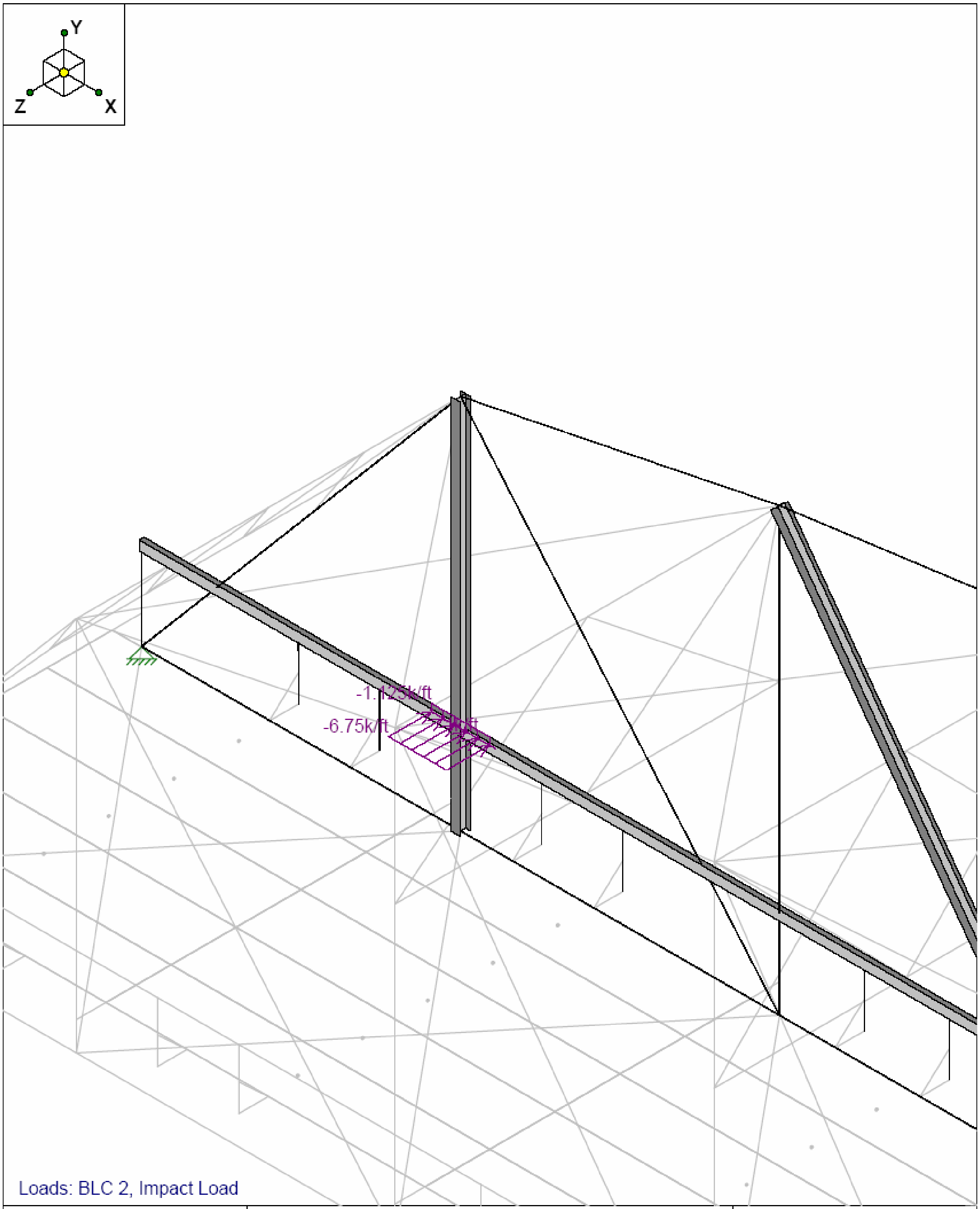
$ksi \equiv \frac{kip}{in^2}$

$kcf \equiv \frac{kip}{ft^3}$

State Highway 16, Roy B. Inks Bridge over Llano River

Three-Dimensional RISA-3D Structural Model and Analysis Results

Load Applied at Second Truss Member (Node 2)



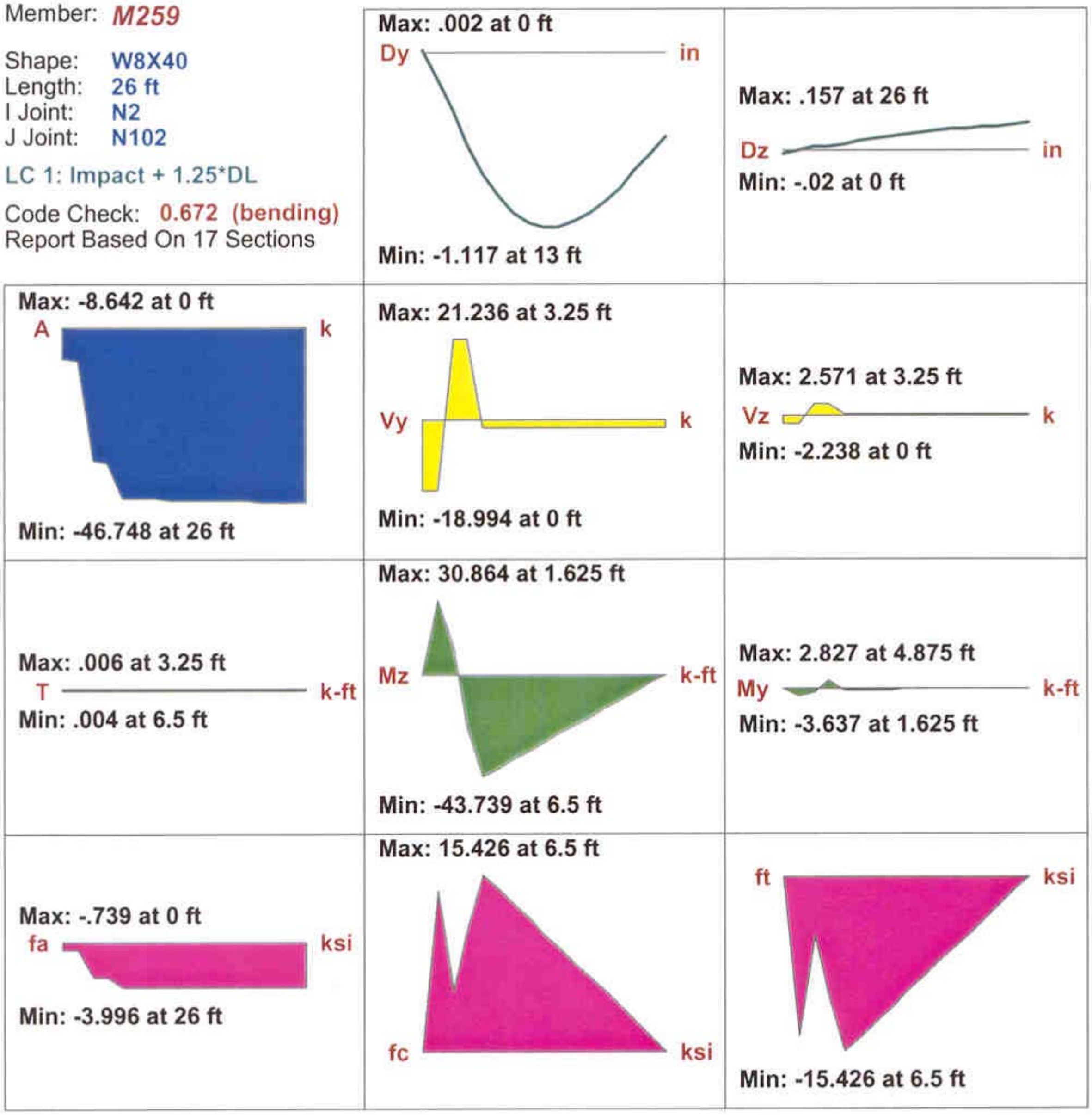
	Llano Truss Bridge Structural Analysis	
William Williams, P.E.		June 10, 2005 at 3:49 PM
	TL-2 Impact Loads on Member L1-U1 (Memb. M259)	Llano_RisaModel2.r3d

Member: **M259**

Shape:	W8X40
Length:	26 ft
I Joint:	N2
J Joint:	N102

LC 1: Impact + 1.25*DL

Code Check: 0.672 (bending)
Report Based On 17 Sections



AISC LRFD 2nd Ed. Code Check

Max Bending Check	0.672	Max Shear Check	0.443 (y)
Location	6.5 ft	Location	3.25 ft
Equation	H1-1b	Max Defl Ratio	L/364

Compact
Seismic Provisions

Fy	30 ksi
phi*Pnc	107.619 k
phi*Pnt	315.9 k
phi*Mny	41.069 k-ft
phi*Mnz	74.774 k-ft
phi*Vny	48.114 k
phi*Vnz	146.422 k
Cb	1

	Y-Y	Z-Z
Lb	26 ft	26 ft
KL/r	152.302	88.322
Sway	No	No

L Comp Flange 26 ft

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Texas Transportation Institute
Texas A&M University System
College Station, Texas

PAGE 1 of 2
JOB NO. 444193
Date: 07-27-03

SUBJECT *Llano S.H. 16 Truss Bridge*
LRFD Beam Check: NODE 2

BY: *W. Williams*
CKD:

CLIENT *TTI*

Beam Column Tension Analysis in Accordance with Chapter H, LRFD

1.) Load Data:

$P_u := 46.75 \cdot \text{kip}$ ***"Tension"***

$M_z := 43.74 \cdot \text{kip} \cdot \text{ft}$

$M_y := 3.6 \cdot \text{kip} \cdot \text{ft}$

2.) Member Data: W8x40

$S_y := 12.2 \cdot \text{in}^3$ $S_z := 35.5 \cdot \text{in}^3$ $A_g := 11.7 \cdot \text{in}^2$

$Z_y := 18.5 \text{in}^3$ $Z_z := 39.8 \text{in}^3$

$K_y := 1.0$ $K_z := 1.0$

$L_y := 26 \text{ft}$ $L_z := 26 \text{ft}$ $t_f := .5 \text{in}$

$r_y := 2.04 \text{in}$ $r_z := 3.53 \text{in}$

$E := 29000 \cdot \text{ksi}$ $F_y := 30 \cdot \text{ksi}$ $F_u := 50 \text{ksi}$ $\phi_b := 0.9$ ***(Ch. "F" LRFD)***

3.) Calculate the Nominal Axial Tension Strength Based on Chapter D, LRFD

D1-1: For Yielding in the Gross Section:

$\phi_{tG} := 0.9$

$P_{nGross} := F_y \cdot A_g$

$\phi P_{nG} := \phi_{tG} \cdot P_{nGross}$

$\phi P_{nG} = 315.9 \text{kip}$

D1-2: For Fracture in the net Section (Assume 4 ~ 7/8" Dia. holes):

$\phi_{tN} := 0.75$ $A_{Net} := A_g - 4 \cdot (t_f \cdot 0.875 \text{in})$

$A_{Net} = 9.95 \text{in}^2$

$P_{nNet} := F_u \cdot A_{Net}$

$\phi P_{nN} := \phi_{tN} \cdot P_{nNet}$

$\phi P_{nN} = 373.125 \text{kip}$

SUBJECT Llano S.H. 16 Truss Bridge
LRFD Beam Check: NODE 2
CLIENT TTI

BY: W. Williams
CKD:

Take the lesser of the two above ..... therefore:

$$\phi P_n := \phi P_{nG}$$

4.) Determine $P_u / \phi P_n$ Ratio:

$$\frac{P_u}{\phi P_n} = \mathbf{0.148}$$

.... therefore ....

(if greater than 0.2, then delete "2" in the denominator and change the 2nd denominator to a "9")

$$M_{uZ} := M_z$$

$$M_{uY} := M_y$$

$$M_{nZ} := F_y \cdot Z_z$$

$$M_{nY} := F_y \cdot Z_y$$

...... and ...

$$\frac{P_u}{2\phi P_n} + \frac{8}{8} \cdot \left(\frac{M_{uZ}}{\phi_b \cdot M_{nZ}} + \frac{M_{uY}}{\phi_b \cdot M_{nY}} \right) = \mathbf{0.649}$$

...... This is O.K. ...... less than 1.0 !!

GLOBAL DEFINITION OF UNITS:

$g = \mathbf{32.174} \frac{ft}{sec^2}$

$plf \equiv \frac{lb}{ft}$

$kip \equiv \mathbf{1000} \cdot lb$

$pcf \equiv \frac{lb}{ft^3}$

$psi \equiv \frac{lb}{in^2}$

$psf \equiv \frac{lb}{ft^2}$

$klf \equiv \frac{kip}{ft}$

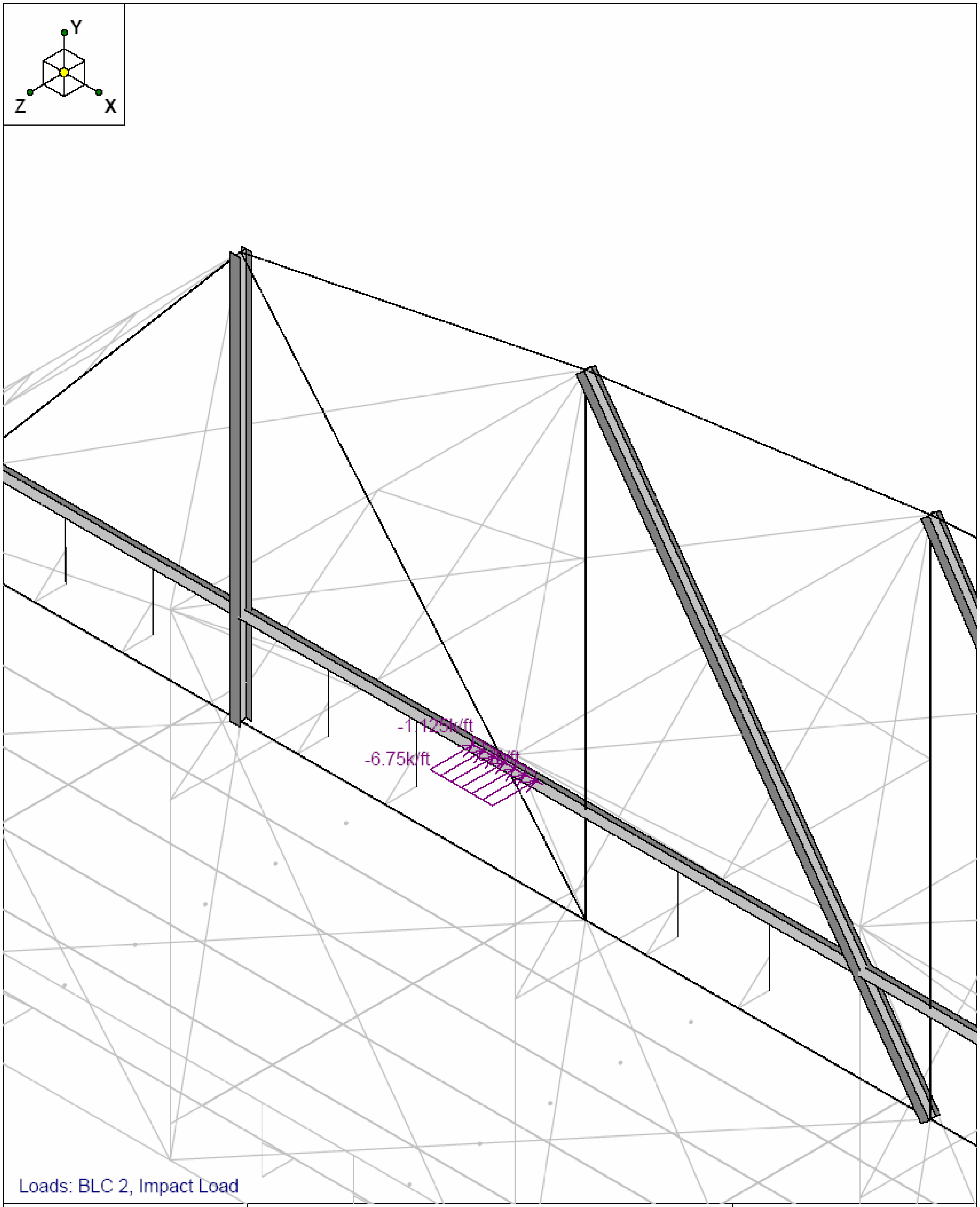
$ksi \equiv \frac{kip}{in^2}$

$kcf \equiv \frac{kip}{ft^3}$

State Highway 16, Roy B. Inks Bridge over Llano River

Three-Dimensional RISA-3D Structural Model and Analysis Results

Load Applied at Third Truss Member (Node 3)



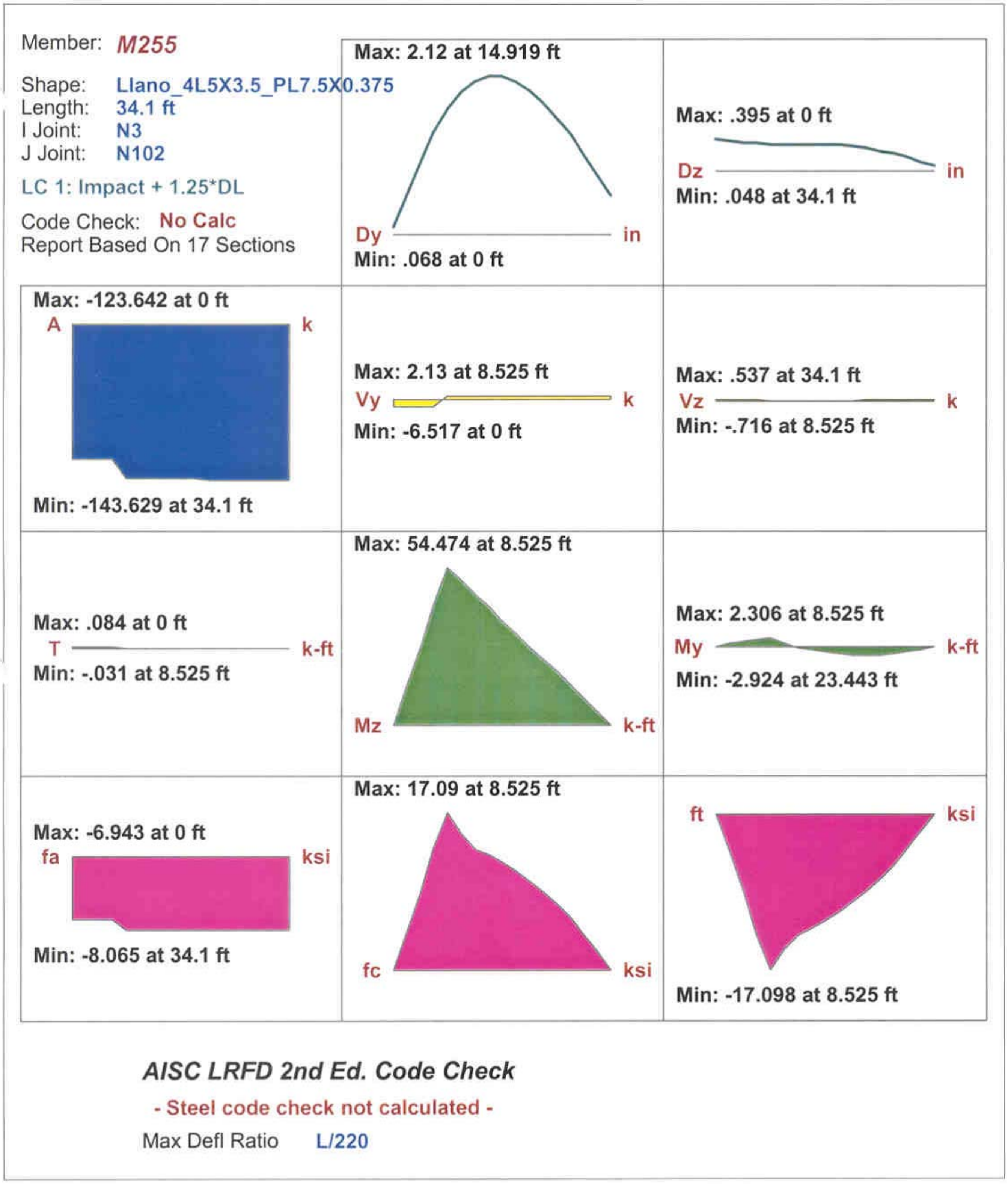
Llano Truss Bridge Structural Analysis

William Williams, P.E.

June 10, 2005 at 4:18 PM

TL-2 Impact Loads on Member L2-U1 (Memb. M255)

Llano_RisaModel3.r3d



Member: **M255**

Shape: Llano_4L5X3.5_PL7.5X0.375
Length: 34.1 ft
I Joint: N3
J Joint: N102

LC 1: Impact + 1.25*DL

Code Check: No Calc
Report Based On 17 Sections

Max: 2.12 at 14.919 ft
Dy **in**
Min: .068 at 0 ft

Max: .395 at 0 ft
Dz **in**
Min: .048 at 34.1 ft

Max: -123.642 at 0 ft
A **k**
Min: -143.629 at 34.1 ft

Max: 2.13 at 8.525 ft
Vy **k**
Min: -6.517 at 0 ft

Max: .537 at 34.1 ft
Vz **k**
Min: -.716 at 8.525 ft

Max: .084 at 0 ft
T **k-ft**
Min: -.031 at 8.525 ft

Max: 54.474 at 8.525 ft
Mz **k-ft**

Max: 2.306 at 8.525 ft
My **k-ft**
Min: -2.924 at 23.443 ft

Max: -6.943 at 0 ft
fa **ksi**
Min: -8.065 at 34.1 ft

Max: 17.09 at 8.525 ft
fc **ksi**

ft **ksi**
Min: -17.098 at 8.525 ft

AISC LRFD 2nd Ed. Code Check

- Steel code check not calculated -

Max Defl Ratio L/220

SUBJECT Llano S.H. 16 Truss Bridge
LRFD Beam Check: NODE 3

BY: W. Williams
CKD:

CLIENT TTI

Beam Column Tension Analysis in Accordance with Chapter H, LRFD

1.) Load Data: $P_u := 143.6 \cdot kip$ "Tension"

$M_z := 54.7 \cdot kip \cdot ft$

$M_y := 2.9 \cdot kip \cdot ft$

2.) Member Data: 4~L5x3.5x1/2 w/ 7.5x0.375 Plate

$S_y := 132.24 \cdot in^3$ $S_z := 135.05 \cdot in^3$ $A_g := 40.643 \cdot in^2$

$Z_y := 209.404 in^3$ $Z_z := 206.533 in^3$

$K_y := 1.0$ $K_z := 1.0$

$L_y := 34 ft$ $L_z := 34 ft$

$r_y := 5.411 in$ $r_z := 5.666 in$

$E := 29000 \cdot ksi$ $F_y := 30 \cdot ksi$ $F_u := 50 ksi$ $\phi_b := 0.9$ (Ch. "F" LRFD)

3.) Calculate the Nominal Axial Tension Strength Based on Chapter D, LRFD

D1-1: For Yielding in the Gross Section:

$\phi_{tG} := 0.9$

$P_{nGross} := F_y \cdot A_g$

$\phi P_{nG} := \phi_{tG} \cdot P_{nGross}$

$\phi P_{nG} = 1097.361 \, kip$

D1-2: For Fracture in the net Section (Assume 4 ~ 7/8" Dia. holes):

$\phi_{tN} := 0.75$ $A_{Net} := A_g - 4 \cdot (0.5 in \cdot 0.875 in)$

$A_{Net} = 38.893 \, in^2$

$P_{nNet} := F_u \cdot A_{Net}$

$\phi P_{nN} := \phi_{tN} \cdot P_{nNet}$

$\phi P_{nN} = 1458.488 \, kip$

SUBJECT *Llano S.H. 16 Truss Bridge*
LRFD Beam Check: NODE 3
BY: *W. Williams*
CKD:
CLIENT *TTI*

Take the lesser of the two above ..... therefore:

$$\phi P_n := \phi P_{nG}$$

4.) Determine $P_u / \phi P_n$ Ratio:

$$\frac{P_u}{\phi P_n} = \mathbf{0.131}$$ ***........ therefore ....***

$M_{uZ} := M_z$

$M_{uY} := M_y$

$M_{nZ} := F_y \cdot Z_z$

$M_{nY} := F_y \cdot Z_y$

...... and ...

$$\frac{P_u}{\phi P_n} + \frac{8}{9} \cdot \left(\frac{M_{uZ}}{\phi_b \cdot M_{nZ}} + \frac{M_{uY}}{\phi_b \cdot M_{nY}} \right) = \mathbf{0.241}$$

...... This is O.K. ...... less than 1.0 !!

GLOBAL DEFINITION OF UNITS:

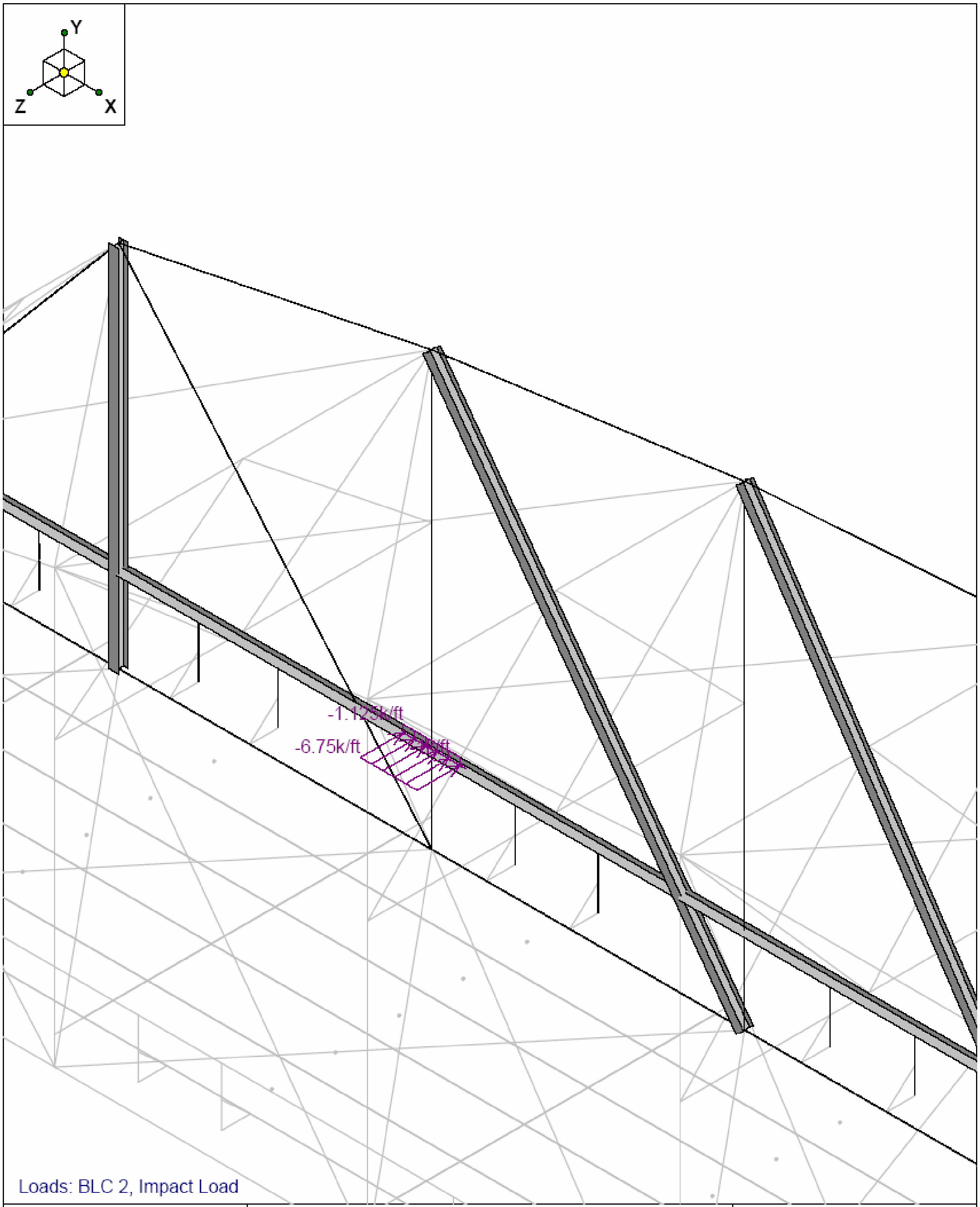
$g = \mathbf{32.174} \frac{ft}{sec^2}$ $plf \equiv \frac{lb}{ft}$ $kip \equiv \mathbf{1000} \cdot lb$ $pcf \equiv \frac{lb}{ft^3}$

$psi \equiv \frac{lb}{in^2}$ $psf \equiv \frac{lb}{ft^2}$ $klf \equiv \frac{kip}{ft}$ $ksi \equiv \frac{kip}{in^2}$ $kcf \equiv \frac{kip}{ft^3}$

State Highway 16, Roy B. Inks Bridge over Llano River

Three-Dimensional RISA-3D Structural Model and Analysis Results

Load Applied at Fourth Truss Member (Node 4)



	Llano Truss Bridge Structural Analysis	
William Williams, P.E.		June 10, 2005 at 4:33 PM
	TL-2 Impact Loads on Member L2-U2 (Memb. M155)	Llano_RisaModel4.r3d

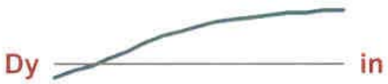
Member: *M151*

Shape: Llano_2C9X20
Length: 30.5 ft
I Joint: N3
J Joint: N104

LC 1: Impact + 1.25*DL

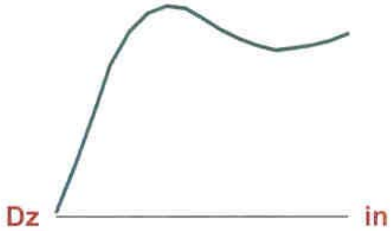
Code Check: No Calc
Report Based On 17 Sections

Max: .149 at 30.5 ft



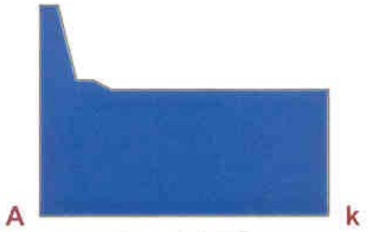
Min: -.041 at 0 ft

Max: .601 at 11.438 ft



Min: .009 at 0 ft

Max: 92.305 at 0 ft

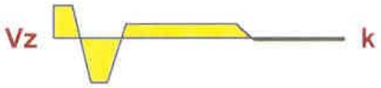


Min: 54.503 at 30.5 ft

Max: 5.226 at 3.812 ft

Min: -3.805 at 0 ft

Max: 13.147 at 0 ft



Min: -19.276 at 3.812 ft

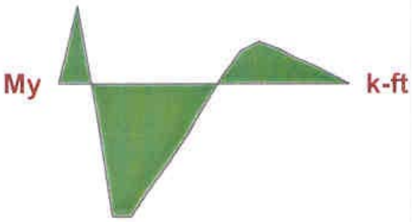
Max: .004 at 7.625 ft

Min: -.012 at 3.812 ft

Max: 7.253 at 1.906 ft

Min: -11.646 at 5.719 ft

Max: 25.061 at 1.906 ft

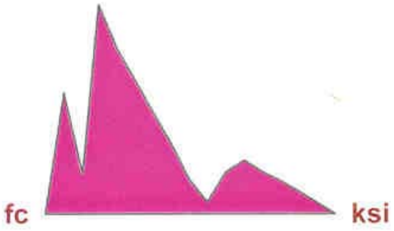


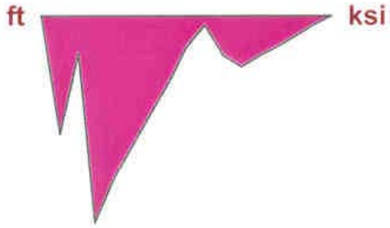
Min: -45.269 at 5.719 ft

Max: 7.851 at 0 ft

Min: 4.636 at 30.5 ft

Max: 20.862 at 5.719 ft





Min: -20.862 at 5.719 ft

AISC LRFD 2nd Ed. Code Check

- Steel code check not calculated -

Max Defl Ratio L/886

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College Station, Texas

PAGE 1 of 2
JOB NO. 444193
Date: 07-27-03

SUBJECT Llano S.H. 16 Truss Bridge
LRFD Beam Check: NODE 4

BY: W. Williams
CKD: ____________

CLIENT TTI

Beam Column Tension Analysis in Accordance with Section 6, LRFD

1.) Load Data:

$P_u := 92.3 \cdot kip$ "Compression"

$M_z := 11.65 \cdot kip \cdot ft$

$M_y := 45.3 \cdot kip \cdot ft$

2.) Member Data: 2~C9x20's

$S_y := 34.48 \cdot in^3$ $S_z := 27.07 \cdot in^3$ $A_g := 11.76 \cdot in^2$

$Z_y := 49.96 in^3$ $Z_z := 33.58 in^3$ $\psi_c := 0.90$ **from LRFD C6.5.4.2**

$K_y := 1.0$ $K_z := 1.0$

$L_y := 17.96 ft$ $L_z := 24.4 ft$

$r_y := 4.306 in$ $r_z := 3.22 in$

$E := 29000 \cdot ksi$ $F_y := 30 \cdot ksi$ $F_u := 50 ksi$ $\phi_b := 0.9$

3.) Calculate the Nominal Axial Strength Based on Section 6, LRFD

$$\lambda_y := \left(\frac{K_y \cdot L_y}{r_y}\right)^2 \cdot \frac{F_y}{E} \qquad \lambda_y = 2.591$$

$$\lambda_z := \left(\frac{K_z \cdot L_z}{r_z}\right)^2 \cdot \frac{F_y}{E} \qquad \lambda_z = 8.554$$

$$\frac{K_y \cdot L_y}{r_y} = 50.051 \qquad \frac{K_z \cdot L_z}{r_z} = 90.932$$

From LRFD 6.9.4.1-2:

$$P_n := \frac{0.88 \cdot F_y \cdot A_g}{\lambda_y} \qquad P_n = 119.801 \, kip$$

From LRFD 6.9.2.1-1:

$$P_r := \psi_c \cdot P_n \qquad P_r = 107.821 \, kip$$

PAGE 2 of 2
JOB NO. 444193
Date: 07-27-03

SUBJECT Llano S.H. 16 Truss Bridge
LRFD Beam Check: NODE 4
BY: W. Williams
CKD:
CLIENT TTI

4.) Determine P_u / P_r Ratio:

$$\frac{P_u}{P_r} = 0.856$$ .... *therefore ....*

$M_{uZ} := M_z$

$M_{uY} := M_y$

$M_{nZ} := F_y \cdot Z_z$

$M_{nY} := F_y \cdot Z_y$

...... and ...

$$\frac{P_u}{P_r} + \frac{8}{9} \cdot \left(\frac{M_{uZ}}{\phi_b \cdot M_{nZ}} + \frac{M_{uY}}{\phi_b \cdot M_{nY}} \right) = 1.351$$

...... This is N.G.!! ...... greater than 1.0 !!

GLOBAL DEFINITION OF UNITS:

$g = 32.174 \frac{ft}{sec^2}$

$plf \equiv \frac{lb}{ft}$

$kip \equiv 1000 \cdot lb$

$pcf \equiv \frac{lb}{ft^3}$

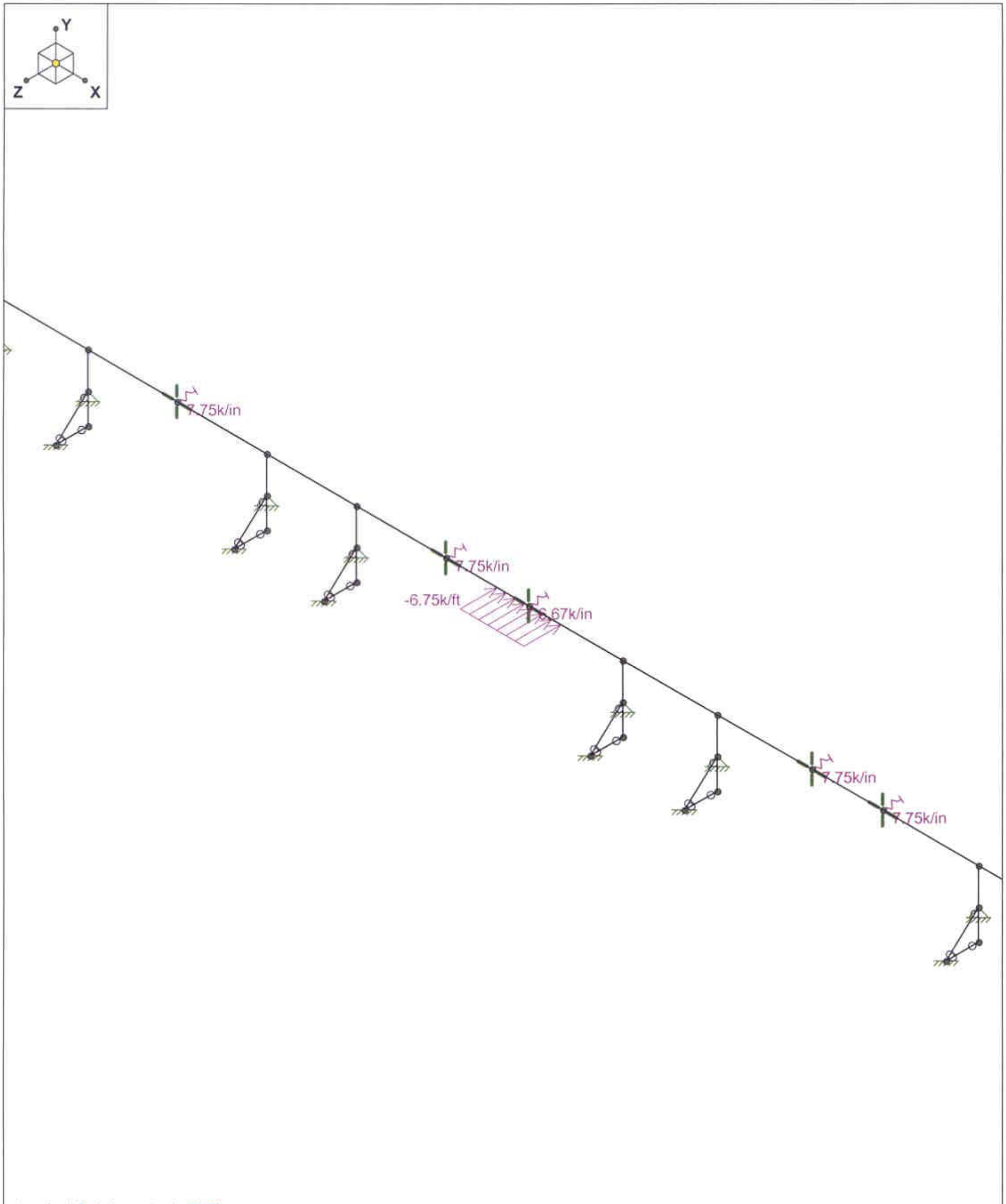
$psi \equiv \frac{lb}{in^2}$

$psf \equiv \frac{lb}{ft^2}$

$klf \equiv \frac{kip}{ft}$

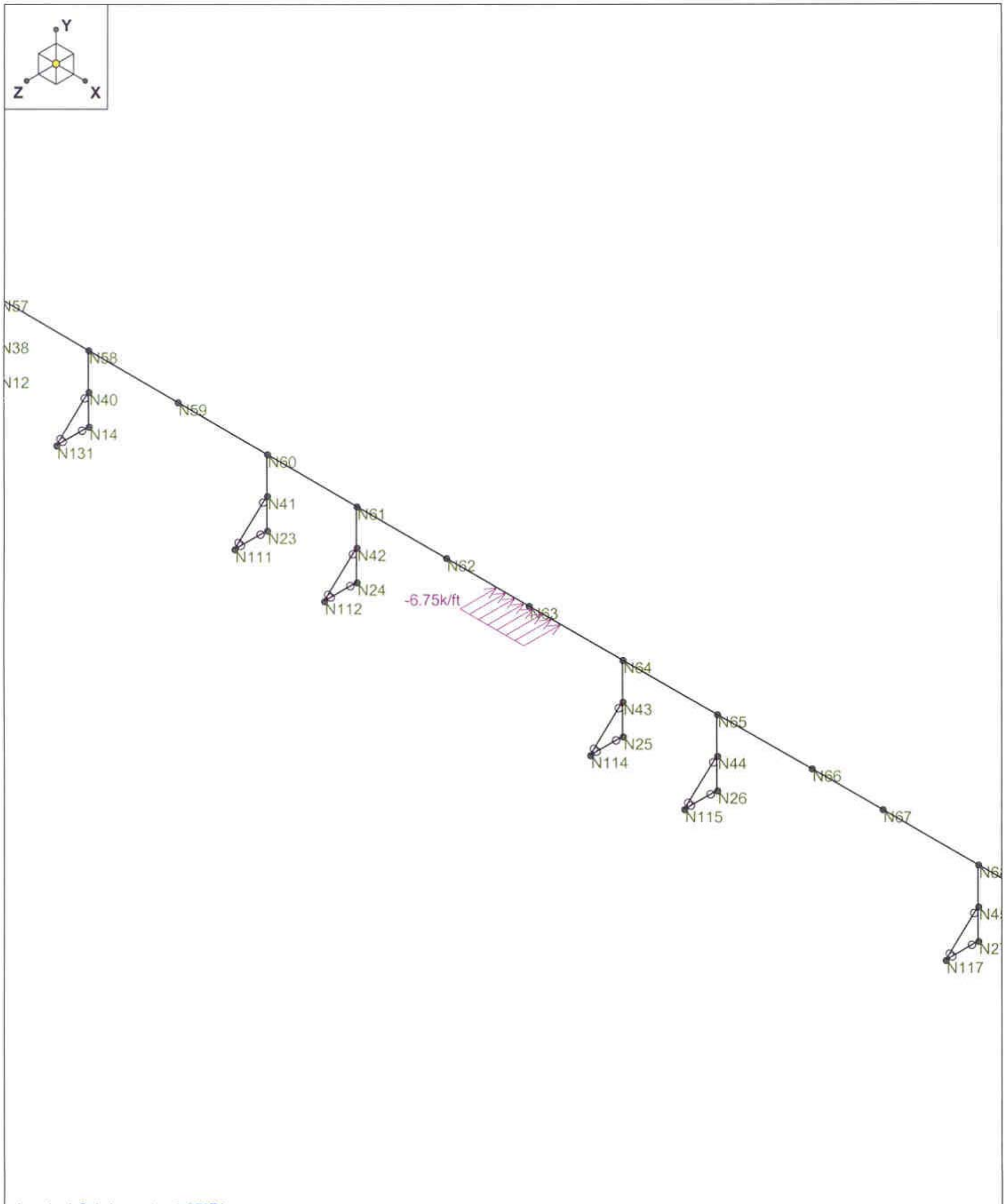
$ksi \equiv \frac{kip}{in^2}$

$kcf \equiv \frac{kip}{ft^3}$



Loads: LC 1, Impact + 1.25*DL
Results for LC 1, Impact + 1.25*DL

William Williams	Llano Truss Bridge Structural Analysis	
William Williams, P.E.		Feb 25, 2004 at 2:13 PM
	Lateral Spring Supports & Impact Loads over 4 feet	LlanoRAILONLY4.r3d



Loads: LC 1, Impact + 1.25*DL
Results for LC 1, Impact + 1.25*DL

William Williams

William Williams, P.E.

Llano Truss Bridge Structural Analysis

Nodes & Geometry

Feb 25, 2004 at 2:13 PM

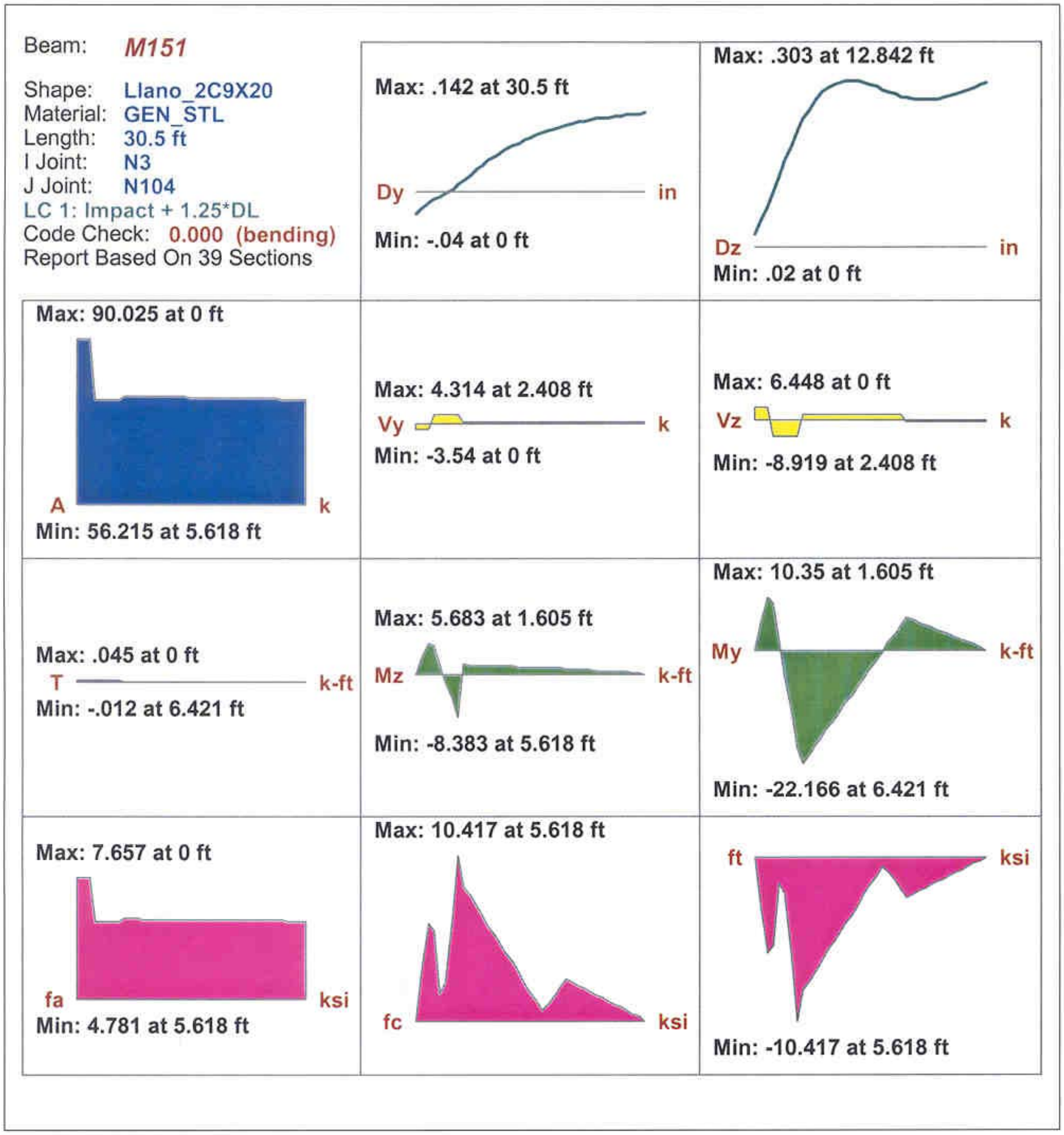
LlanoRAILONLY4.r3d

Joint Reactions (By Combination)

	LC	Joint Label	X [k]	Y [k]	Z [k]	MX [k-ft]	MY [k-ft]	MZ [k-ft]
1	1	N10	0	.054	0	0	0	0
2	1	N1	0	.054	.002	0	0	0
3	1	N38	0	.393	.013	0	0	0
4	1	N40	-.002	.398	.109	0	0	0
5	1	N41	.001	.396	-2.987	0	0	0
6	1	N42	-.005	.401	7.381	0	0	0
7	1	N43	.012	.411	26.268	0	0	0
8	1	N44	.001	.415	-4.866	0	0	0
9	1	N46	.001	.419	.081	0	0	0
10	1	N45	-.001	.421	.101	0	0	0
11	1	N47	0	.365	.002	0	0	0
12	1	N48	0	.365	.007	0	0	0
13	1	N49	-.001	.419	0	0	0	0
14	1	N50	.001	.421	0	0	0	0
15	1	N51	-.001	.416	0	0	0	0
16	1	N52	0	.411	0	0	0	0
17	1	N54	0	.396	0	0	0	0
18	1	N53	0	.401	0	0	0	0
19	1	N39	-.002	.398	0	0	0	0
20	1	N37	-.004	.393	0	0	0	0
21	1	N133	0	.032	-.019	0	0	0
22	1	N131	0	.032	-.068	0	0	0
23	1	N111	0	.032	.854	0	0	0
24	1	N112	0	.032	-3.712	0	-.008	.009
25	1	N114	0	.032	-12.011	0	.018	-.02
26	1	N115	0	.032	.939	0	0	0
27	1	N117	0	.032	-.132	0	0	0
28	1	N118	0	.032	-.07	0	0	0
29	1	N137	0	.032	-.006	0	0	0
30	1	N120	0	.032	-.005	0	0	0
31	1	N122	0	.032	0	0	0	0
32	1	N123	0	.032	0	0	0	0
33	1	N125	0	.032	0	0	0	0
34	1	N126	0	.032	0	0	0	0
35	1	N128	0	.032	0	0	0	0
36	1	N129	0	.032	0	0	0	0
37	1	N132	0	.032	0	0	0	0
38	1	N134	0	.032	0	0	0	0
39	1	N56	.001	.254	.001	0	0	0
40	1	N91	.003	.254	0	0	0	0
41	1	N88	.003	.236	0	0	0	0
42	1	N85	0	.221	0	0	0	0
43	1	N84	0	.229	0	0	0	0
44	1	N81	0	.207	0	0	0	0
45	1	N80	0	.211	0	0	0	0
46	1	N77	0	.22	0	0	0	0
47	1	N76	0	.138	0	0	0	0
48	1	N75	0	.187	0	0	0	0
49	1	N72	0	.187	0	0	0	0
50	1	N71	0	.138	0	0	0	0
51	1	N70	0	.22	.003	0	0	0
52	1	N67	0	.211	.042	0	0	0
53	1	N66	-.005	.207	.058	0	0	0
54	1	N63	-.008	.229	8.647	0	0	0
55	1	N62	.003	.221	6.346	0	0	0
56	1	N59	.003	.236	.024	0	0	0

Joint Reactions (By Combination) (Continued)

	LC	Joint Label	X [k]	Y [k]	Z [k]	MX [k-ft]	MY [k-ft]	MZ [k-ft]
57	1	N370	0	.36	-.003	0	0	0
58	1	N372	0	.36	0	0	0	0
59	1	Totals:	0	12.446	27			
60	1	COG (ft):	X: 99.287	Y: 5.198	Z: .094			



Beam: *M151*

Shape: Llano_2C9X20
Material: GEN_STL
Length: 30.5 ft
I Joint: N3
J Joint: N104
LC 1: Impact + 1.25*DL
Code Check: 0.000 (bending)
Report Based On 39 Sections

TEXAS TRANSPORTATION INSTITUTE

Texas Transportation Institute
Texas A&M University System
College Station, Texas

PAGE 1 of 2
JOB NO. 444193
Date: 07-27-03

SUBJECT Llano S.H. 16 Truss Bridge
LRFD Beam Check: NODE 4
BY: W. Williams
CKD:
CLIENT TTI

Beam Column Tension Analysis in Accordance with Section 6, LRFD

1.) Load Data:

$P_u := 90.03 \cdot kip$ **"Compression"**

$M_z := 8.38 \cdot kip \cdot ft$

$M_y := 22.16 \cdot kip \cdot ft$

2.) Member Data: 2~C9x20's

$S_y := 34.48 \cdot in^3$ $S_z := 27.07 \cdot in^3$ $A_g := 11.76 \cdot in^2$

$Z_y := 49.96 in^3$ $Z_z := 33.58 in^3$ $\psi_c := 0.90$ **from LRFD C6.5.4.2**

$K_y := 1.0$ $K_z := 1.0$

$L_y := 16.5 ft$ $L_z := 24.4 ft$

$r_y := 4.306 in$ $r_z := 3.22 in$

$E := 29000 \cdot ksi$ $F_y := 30 \cdot ksi$ $F_u := 50 ksi$ $\phi_b := 0.9$

3.) Calculate the Nominal Axial Strength Based on Section 6, LRFD

$$\lambda_y := \left(\frac{K_y \cdot L_y}{r_y}\right)^2 \cdot \frac{F_y}{E} \qquad \lambda_y = 2.187$$

$$\lambda_z := \left(\frac{K_z \cdot L_z}{r_z}\right)^2 \cdot \frac{F_y}{E} \qquad \lambda_z = 8.554$$

$$\frac{K_y \cdot L_y}{r_y} = 45.982 \qquad \frac{K_z \cdot L_z}{r_z} = 90.932$$

From LRFD 6.9.4.1-2:

$$P_n := \frac{0.88 \cdot F_y \cdot A_g}{\lambda_y} \qquad P_n = 141.94\, kip$$

From LRFD 6.9.2.1-1:

$$P_r := \psi_c \cdot P_n \qquad P_r = 127.746\, kip$$

SUBJECT Llano S.H. 16 Truss Bridge
LRFD Beam Check: NODE 4
CLIENT TTI

BY: W. Williams
CKD:

4.) Determine P_u / P_r Ratio:

$$\frac{P_u}{P_r} = 0.705$$

.... ***therefore ....***

$$M_{uZ} := M_z$$

$$M_{uY} := M_y$$

$$M_{nZ} := F_y \cdot Z_z$$

$$M_{nY} := F_y \cdot Z_y$$

...... ***and ...***

$$\frac{P_u}{P_r} + \frac{8}{9} \cdot \left(\frac{M_{uZ}}{\phi_b \cdot M_{nZ}} + \frac{M_{uY}}{\phi_b \cdot M_{nY}} \right) = 0.979$$

...... ***This is O.K.***

GLOBAL DEFINITION OF UNITS:

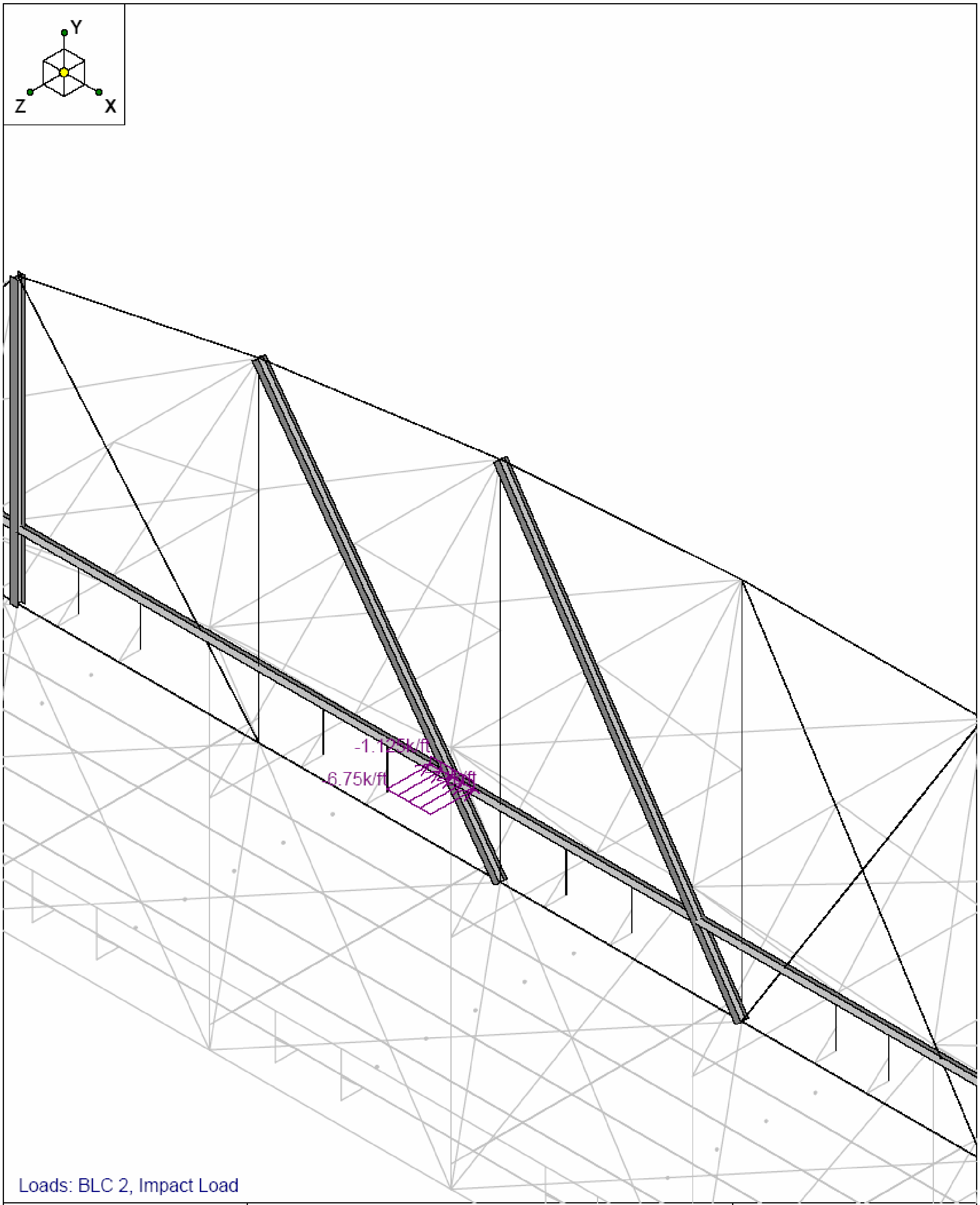
$g = 32.174 \frac{ft}{sec^2}$ $plf \equiv \frac{lb}{ft}$ $kip \equiv 1000 \cdot lb$ $pcf \equiv \frac{lb}{ft^3}$

$psi \equiv \frac{lb}{in^2}$ $psf \equiv \frac{lb}{ft^2}$ $klf \equiv \frac{kip}{ft}$ $ksi \equiv \frac{kip}{in^2}$ $kcf \equiv \frac{kip}{ft^3}$

State Highway 16, Roy B. Inks Bridge over Llano River

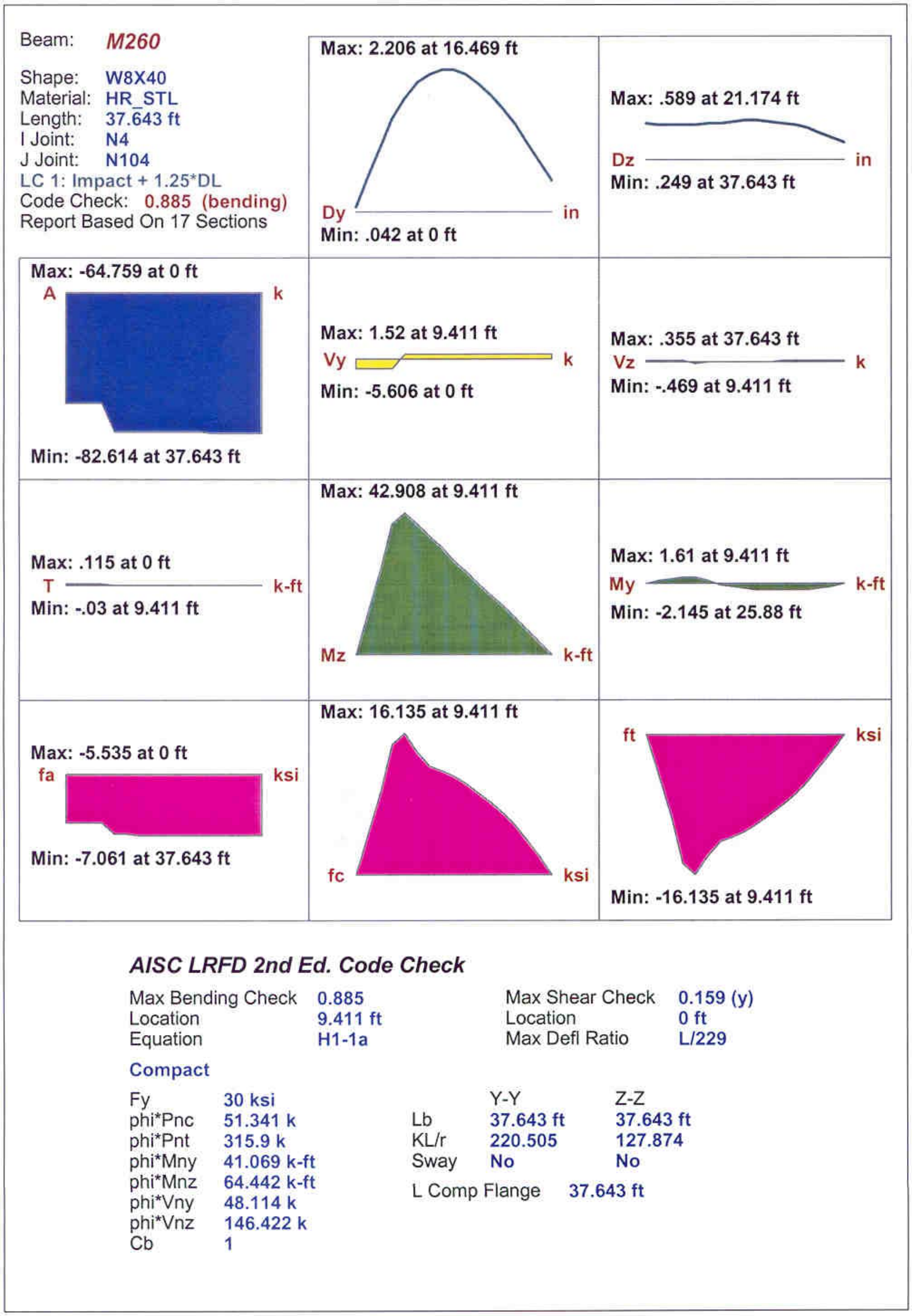
Three-Dimensional RISA-3D Structural Model and Analysis Results

Load Applied at Fifth Truss Member (Node 5)



Loads: BLC 2, Impact Load

	Llano Truss Bridge Structural Analysis	
William Williams, P.E.		June 10, 2005 at 4:36 PM
	TL-2 Impact Loads on Member L3-U2 (Memb. M260)	Llano_RisaModel5.r3d



Beam: ***M260***

Shape: W8X40
Material: HR_STL
Length: 37.643 ft
I Joint: N4
J Joint: N104
LC 1: Impact + 1.25*DL
Code Check: 0.885 (bending)
Report Based On 17 Sections

AISC LRFD 2nd Ed. Code Check

Max Bending Check	0.885	Max Shear Check	0.159 (y)
Location	9.411 ft	Location	0 ft
Equation	H1-1a	Max Defl Ratio	L/229

Compact

Fy	30 ksi
phi*Pnc	51.341 k
phi*Pnt	315.9 k
phi*Mny	41.069 k-ft
phi*Mnz	64.442 k-ft
phi*Vny	48.114 k
phi*Vnz	146.422 k
Cb	1

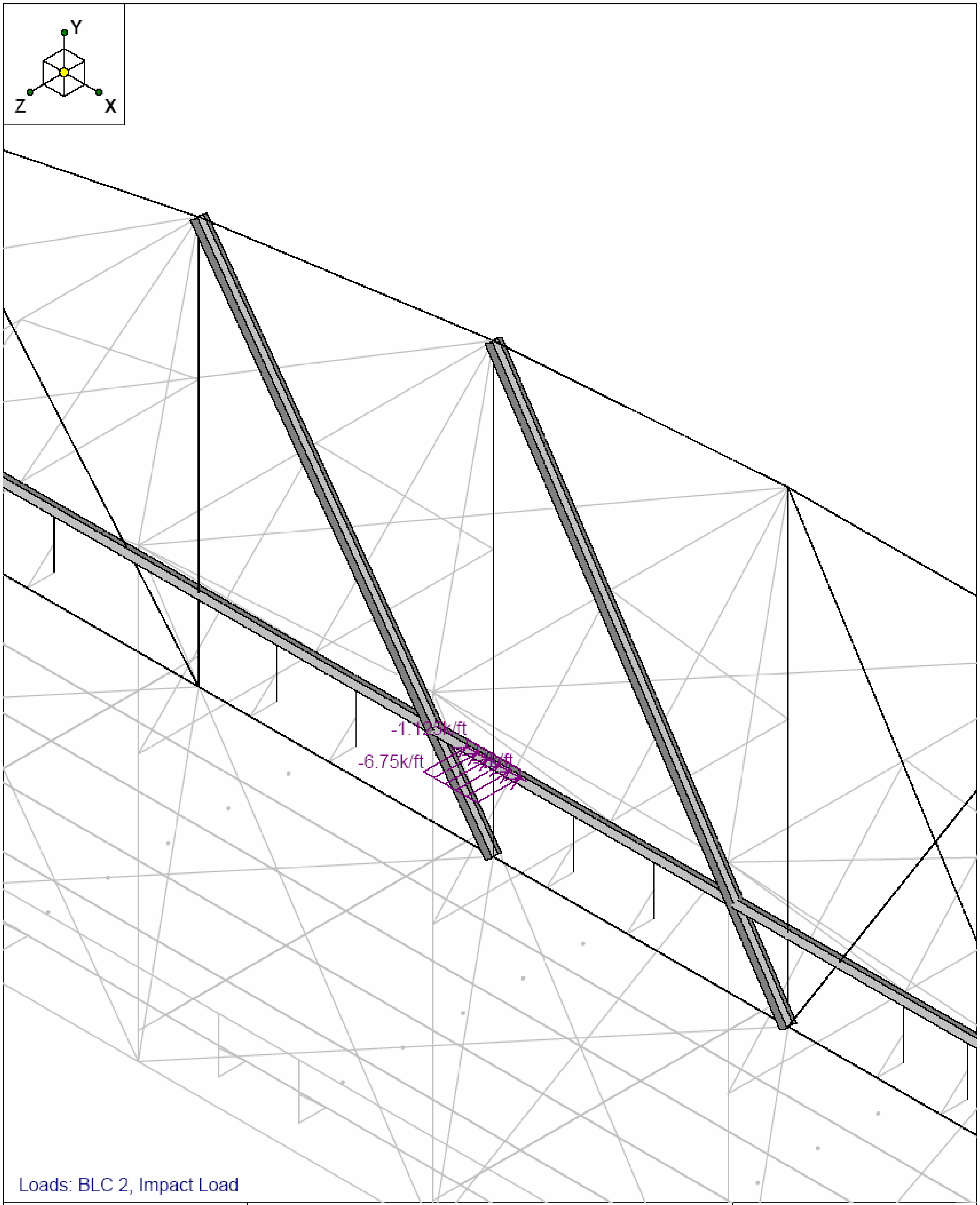
	Y-Y	Z-Z
Lb	37.643 ft	37.643 ft
KL/r	220.505	127.874
Sway	No	No

L Comp Flange 37.643 ft

State Highway 16, Roy B. Inks Bridge over Llano River

Three-Dimensional RISA-3D Structural Model and Analysis Results

Load Applied at Sixth Truss Member (Node 6)



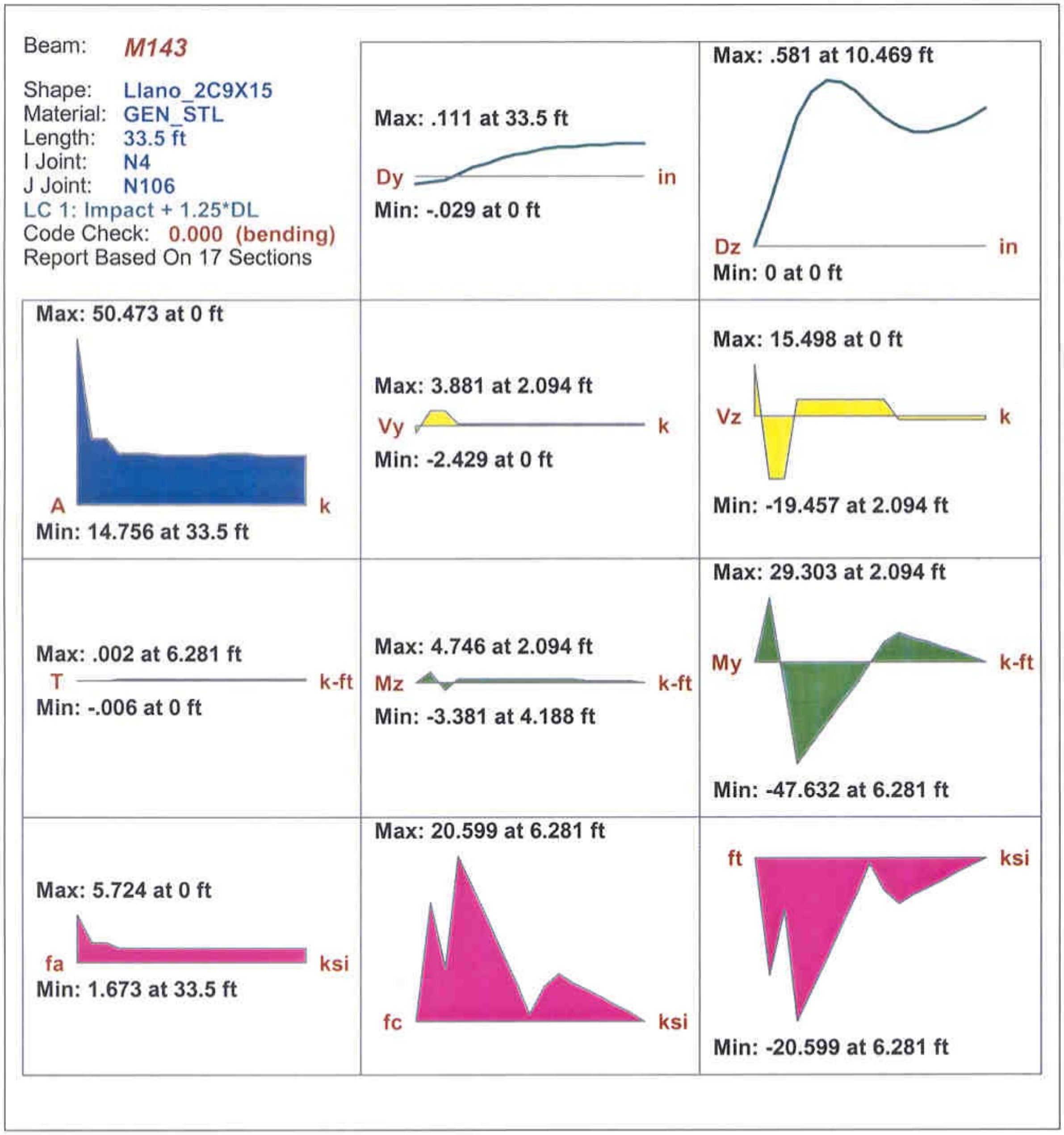
Llano Truss Bridge Structural Analysis

William Williams, P.E.

June 10, 2005 at 4:39 PM

TL-2 Impact Loads on Member L3-U3 (Memb. M143)

Llano_RisaModel6.r3d



Beam: *M143*

Shape: Llano_2C9X15
Material: GEN_STL
Length: 33.5 ft
I Joint: N4
J Joint: N106
LC 1: Impact + 1.25*DL
Code Check: 0.000 (bending)
Report Based On 17 Sections

SUBJECT Llano S.H. 16 Truss Bridge
LRFD Beam Check: NODE 6
BY: W. Williams
CKD:
CLIENT TTI

Beam Column Tension Analysis in Accordance with Section 6, LRFD

1.) Load Data:

$P_u := 50.5 \cdot kip$ "Compression"

$M_z := 4.8 \cdot kip \cdot ft$

$M_y := 47.6 \cdot kip \cdot ft$

2.) Member Data: 2~C9x15's

$S_y := 28.7 \cdot in^3$ $S_z := 22.67 \cdot in^3$ $A_g := 8.82 \cdot in^2$

$Z_y := 39.92 in^3$ $Z_z := 27.10 in^3$ $\psi_c := 0.90$ **from LRFD C6.5.4.2**

$K_y := 1.0$ $K_z := 1.0$

$L_y := 17.96 ft$ $L_z := 27.5 ft$

$r_y := 4.57 in$ $r_z := 3.40 in$

$E := 29000 \cdot ksi$ $F_y := 30 \cdot ksi$ $F_u := 50 ksi$ $\phi_b := 0.9$

3.) Calculate the Nominal Axial Strength Based on Section 6, LRFD

$$\lambda_y := \left(\frac{K_y \cdot L_y}{r_y}\right)^2 \cdot \frac{F_y}{E} \qquad \lambda_y = 2.301$$

$$\lambda_z := \left(\frac{K_z \cdot L_z}{r_z}\right)^2 \cdot \frac{F_y}{E} \qquad \lambda_z = 9.745$$

$$\frac{K_y \cdot L_y}{r_y} = 47.16 \qquad \frac{K_z \cdot L_z}{r_z} = 97.059$$

From LRFD 6.9.4.1-2:

$$P_n := \frac{0.88 \cdot F_y \cdot A_g}{\lambda_y} \qquad P_n = 101.206 \, kip$$

From LRFD 6.9.2.1-1:

$$P_r := \psi_c \cdot P_n \qquad P_r = 91.085 \, kip$$

SUBJECT Llano S.H. 16 Truss Bridge
LRFD Beam Check: NODE 6
BY: W. Williams
CKD:
CLIENT TTI

4.) Determine P_u / P_r Ratio:

$$\frac{P_u}{P_r} = 0.554$$ .... **therefore ....**

$M_{uZ} := M_z$

$M_{uY} := M_y$

$M_{nZ} := F_y \cdot Z_z$

$M_{nY} := F_y \cdot Z_y$

...... and ...

$$\frac{P_u}{P_r} + \frac{8}{9} \cdot \left(\frac{M_{uZ}}{\phi_b \cdot M_{nZ}} + \frac{M_{uY}}{\phi_b \cdot M_{nY}} \right) = 1.095$$

...... This is N.G.!! ...... greater than 1.0 !!

GLOBAL DEFINITION OF UNITS:

$g = 32.174 \frac{ft}{sec^2}$

$plf \equiv \frac{lb}{ft}$

$kip \equiv 1000 \cdot lb$

$pcf \equiv \frac{lb}{ft^3}$

$psi \equiv \frac{lb}{in^2}$

$psf \equiv \frac{lb}{ft^2}$

$klf \equiv \frac{kip}{ft}$

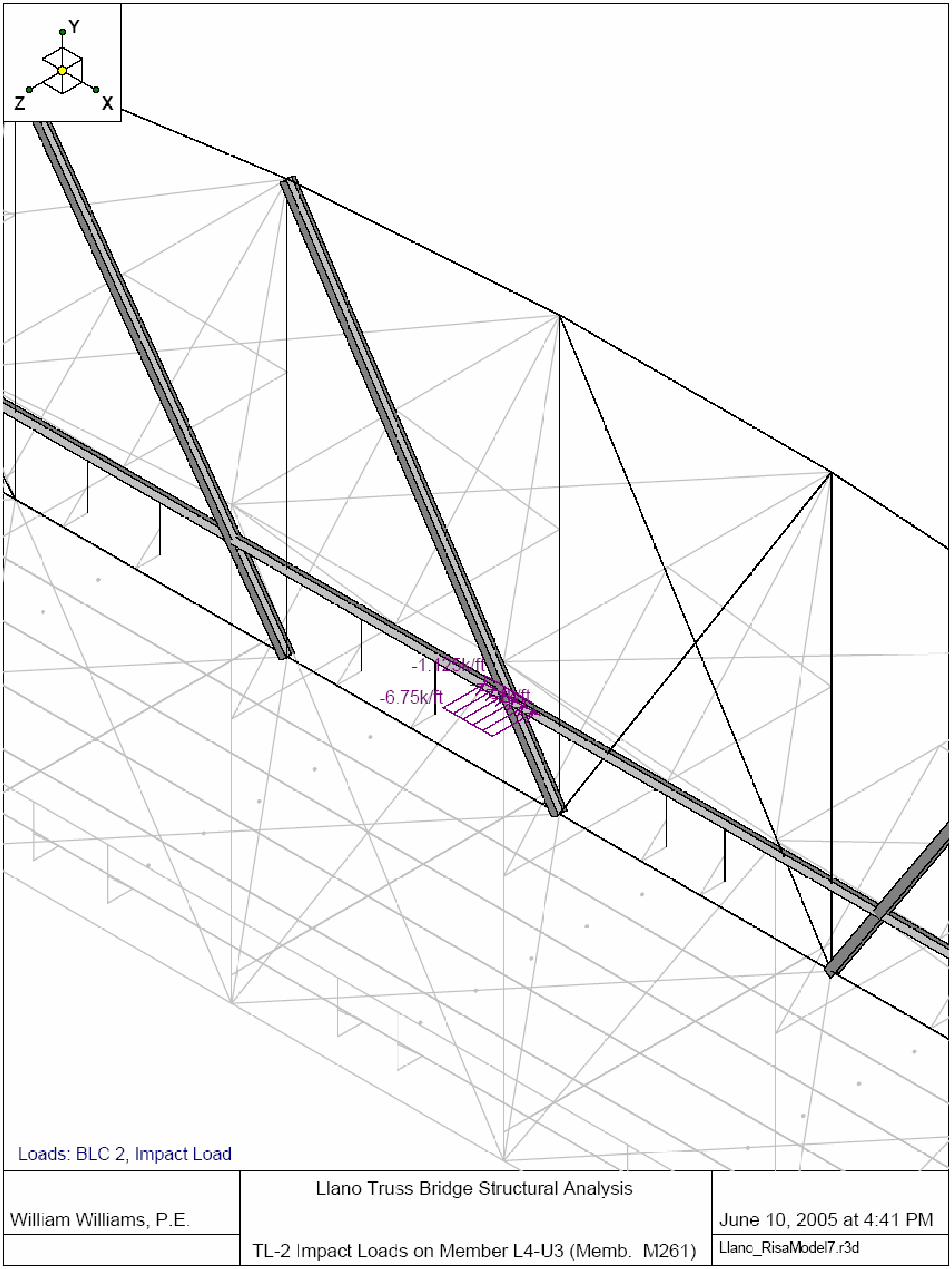
$ksi \equiv \frac{kip}{in^2}$

$kcf \equiv \frac{kip}{ft^3}$

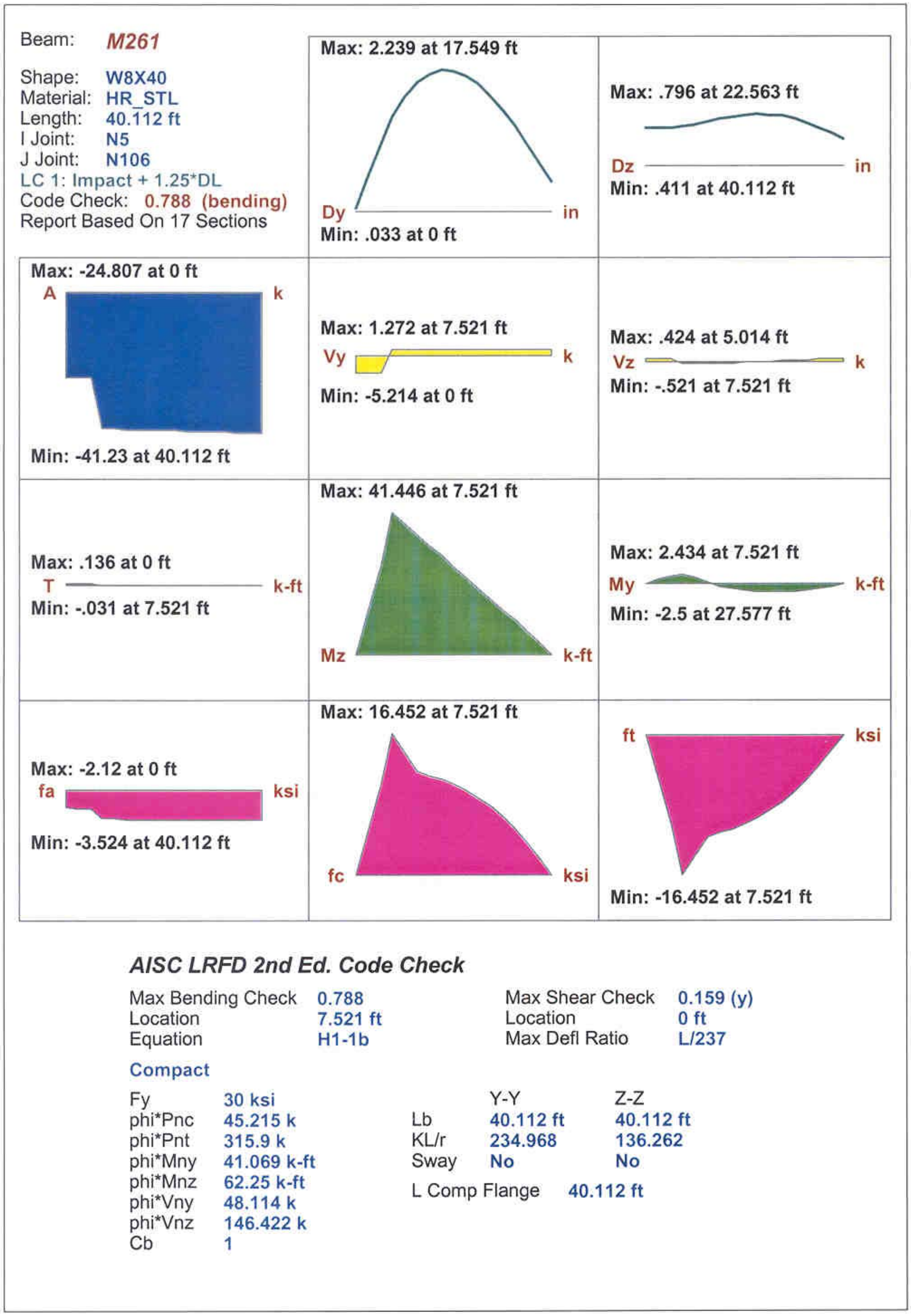
State Highway 16, Roy B. Inks Bridge over Llano River

Three-Dimensional RISA-3D Structural Model and Analysis Results

Load Applied at Seventh Truss Member (Node 7)



	Llano Truss Bridge Structural Analysis	
William Williams, P.E.		June 10, 2005 at 4:41 PM
	TL-2 Impact Loads on Member L4-U3 (Memb. M261)	Llano_RisaModel7.r3d



Beam:	*M261*
Shape:	W8X40
Material:	HR_STL
Length:	40.112 ft
I Joint:	N5
J Joint:	N106

LC 1: Impact + 1.25*DL

Code Check: 0.788 (bending)

Report Based On 17 Sections

AISC LRFD 2nd Ed. Code Check

Max Bending Check	0.788	Max Shear Check	0.159 (y)
Location	7.521 ft	Location	0 ft
Equation	H1-1b	Max Defl Ratio	L/237

Compact

Fy	30 ksi
phi*Pnc	45.215 k
phi*Pnt	315.9 k
phi*Mny	41.069 k-ft
phi*Mnz	62.25 k-ft
phi*Vny	48.114 k
phi*Vnz	146.422 k
Cb	1

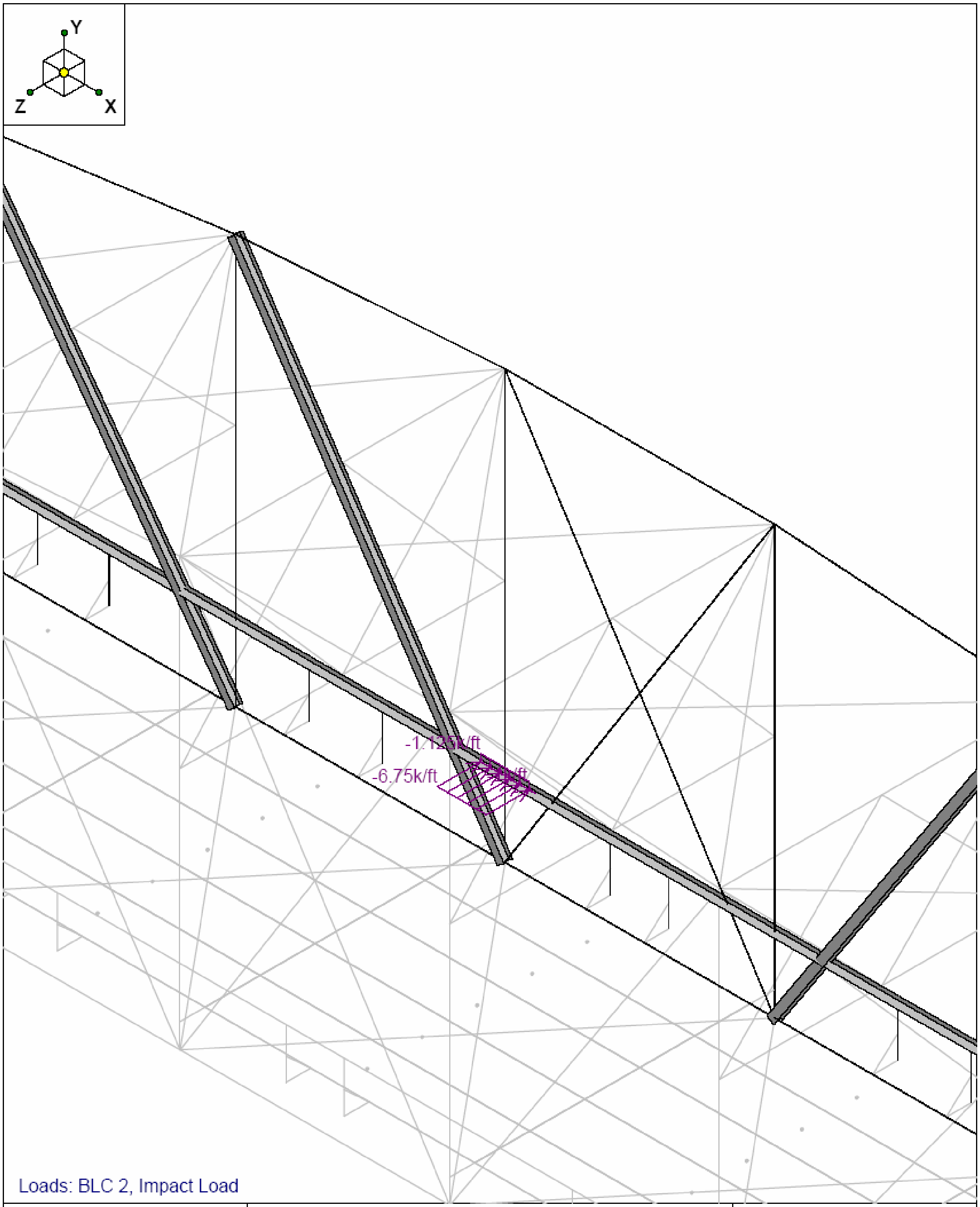
	Y-Y	Z-Z
Lb	40.112 ft	40.112 ft
KL/r	234.968	136.262
Sway	No	No

L Comp Flange 40.112 ft

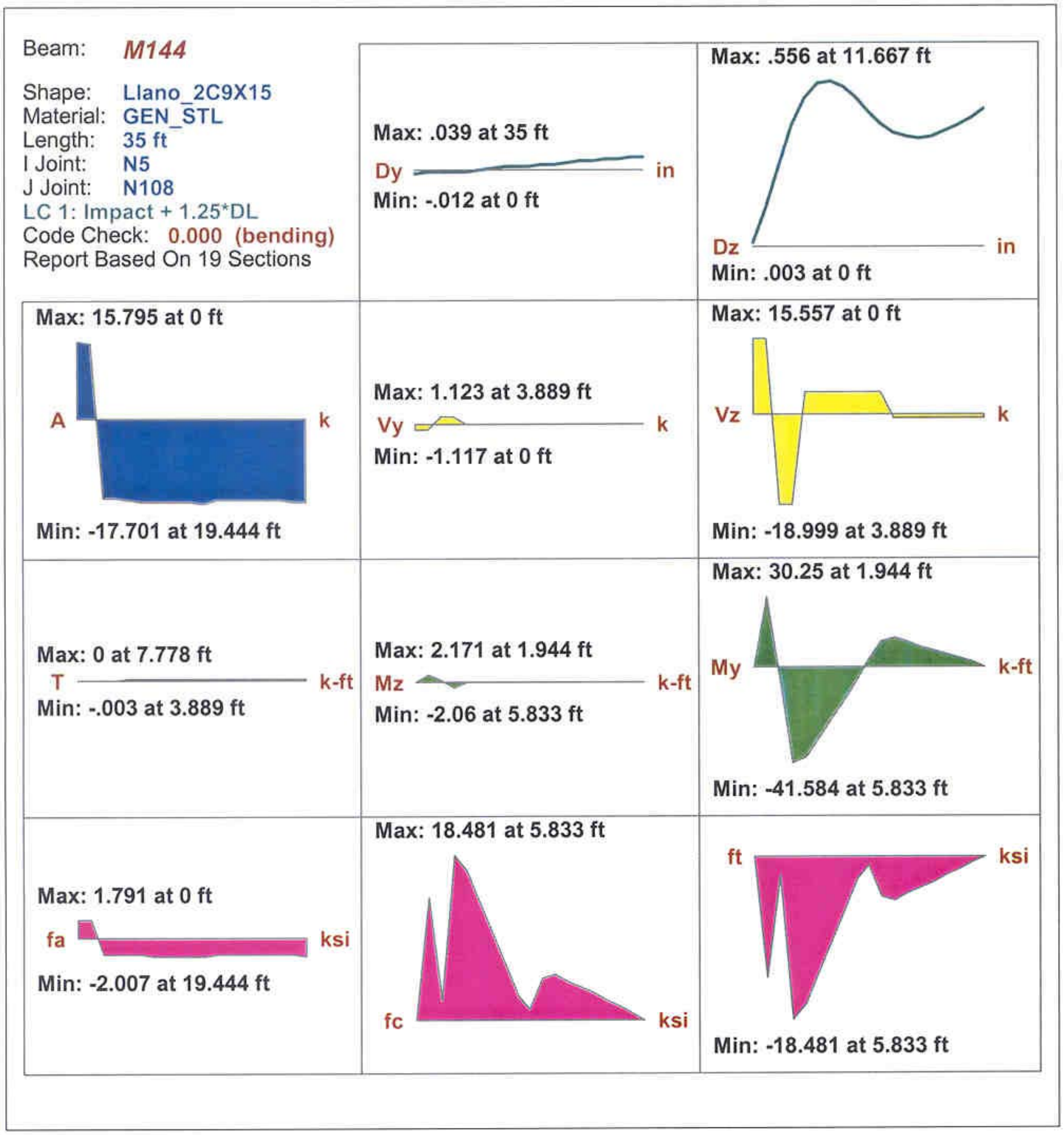
State Highway 16, Roy B. Inks Bridge over Llano River

Three-Dimensional RISA-3D Structural Model and Analysis Results

Load Applied at Eighth Truss Member (Node 8)



William Williams	Llano Truss Bridge Structural Analysis	
William Williams, P.E.		June 10, 2005 at 4:44 PM
	TL-2 Impact Loads on Member L4-U4 (Memb. M144)	Llano_RisaModel8.r3d



Beam: *M144*

Shape: Llano_2C9X15
Material: GEN_STL
Length: 35 ft
I Joint: N5
J Joint: N108
LC 1: Impact + 1.25*DL
Code Check: 0.000 (bending)
Report Based On 19 Sections

SUBJECT Llano S.H. 16 Truss Bridge
LRFD Beam Check: NODE 8

BY: W. Williams
CKD:

CLIENT TTI

Beam Column Tension Analysis in Accordance with Section 6, LRFD

1.) Load Data:

$P_u := 16 \cdot kip$ **"Compression"**

$M_z := 2.0 \cdot kip \cdot ft$

$M_y := 44.9 \cdot kip \cdot ft$

2.) Member Data: 2~C9x15's

$S_y := 28.7 \cdot in^3$ $S_z := 22.67 \cdot in^3$ $A_g := 8.82 \cdot in^2$

$Z_y := 39.92 in^3$ $Z_z := 27.10 in^3$ $\psi_c := 0.90$ **from LRFD C6.5.4.2**

$K_y := 1.0$ $K_z := 1.0$

$L_y := 17.96 ft$ $L_z := 27.5 ft$

$r_y := 4.57 in$ $r_z := 3.40 in$

$E := 29000 \cdot ksi$ $F_y := 30 \cdot ksi$ $F_u := 50 ksi$ $\phi_b := 0.9$

3.) Calculate the Nominal Axial Strength Based on Section 6, LRFD

$$\lambda_y := \left(\frac{K_y \cdot L_y}{r_y}\right)^2 \cdot \frac{F_y}{E} \qquad \lambda_y = 2.301$$

$$\lambda_z := \left(\frac{K_z \cdot L_z}{r_z}\right)^2 \cdot \frac{F_y}{E} \qquad \lambda_z = 9.745$$

$$\frac{K_y \cdot L_y}{r_y} = 47.16 \qquad \frac{K_z \cdot L_z}{r_z} = 97.059$$

From LRFD 6.9.4.1-2:

$$P_n := \frac{0.88 \cdot F_y \cdot A_g}{\lambda_y} \qquad P_n = 101.206 \, kip$$

From LRFD 6.9.2.1-1:

$$P_r := \psi_c \cdot P_n \qquad P_r = 91.085 \, kip$$

SUBJECT Llano S.H. 16 Truss Bridge
LRFD Beam Check: NODE 8
CLIENT TTI

BY: W. Williams
CKD:

4.) Determine P_u / P_r Ratio:

$$\frac{P_u}{P_r} = 0.176$$ .... *therefore ....*

$M_{uZ} := M_z$

$M_{uY} := M_y$

$M_{nZ} := F_y \cdot Z_z$

$M_{nY} := F_y \cdot Z_y$

...... and ...

$$\frac{P_u}{P_r} + \frac{8}{9} \cdot \left(\frac{M_{uZ}}{\phi_b \cdot M_{nZ}} + \frac{M_{uY}}{\phi_b \cdot M_{nY}} \right) = 0.649$$

...... This is o.k.! ...... less than 1.0 !!

GLOBAL DEFINITION OF UNITS:

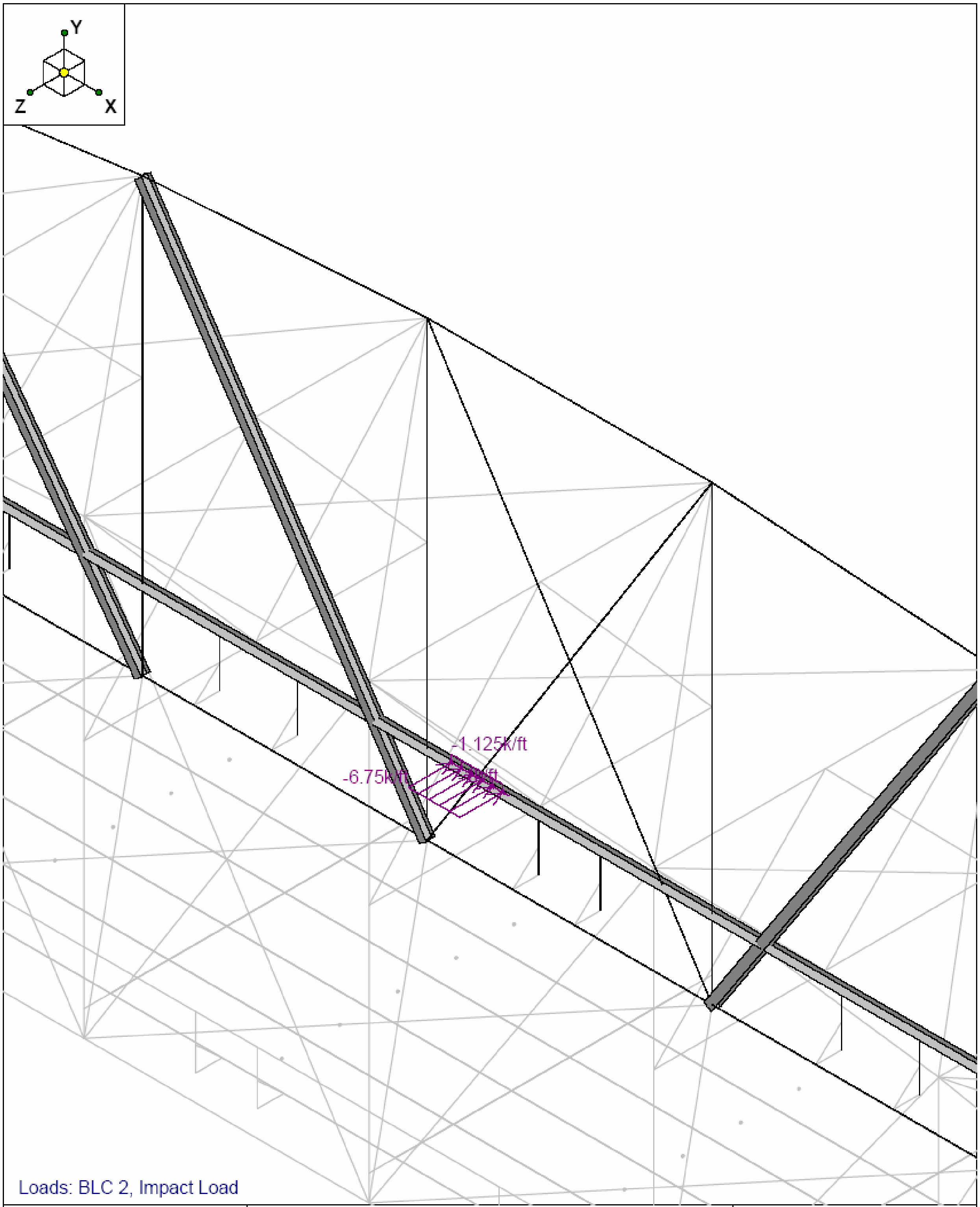
$g = 32.174 \frac{ft}{sec^2}$ $\quad plf \equiv \frac{lb}{ft}$ $\quad kip \equiv 1000 \cdot lb$ $\quad pcf \equiv \frac{lb}{ft^3}$

$psi \equiv \frac{lb}{in^2}$ $\quad psf \equiv \frac{lb}{ft^2}$ $\quad klf \equiv \frac{kip}{ft}$ $\quad ksi \equiv \frac{kip}{in^2}$ $\quad kcf \equiv \frac{kip}{ft^3}$

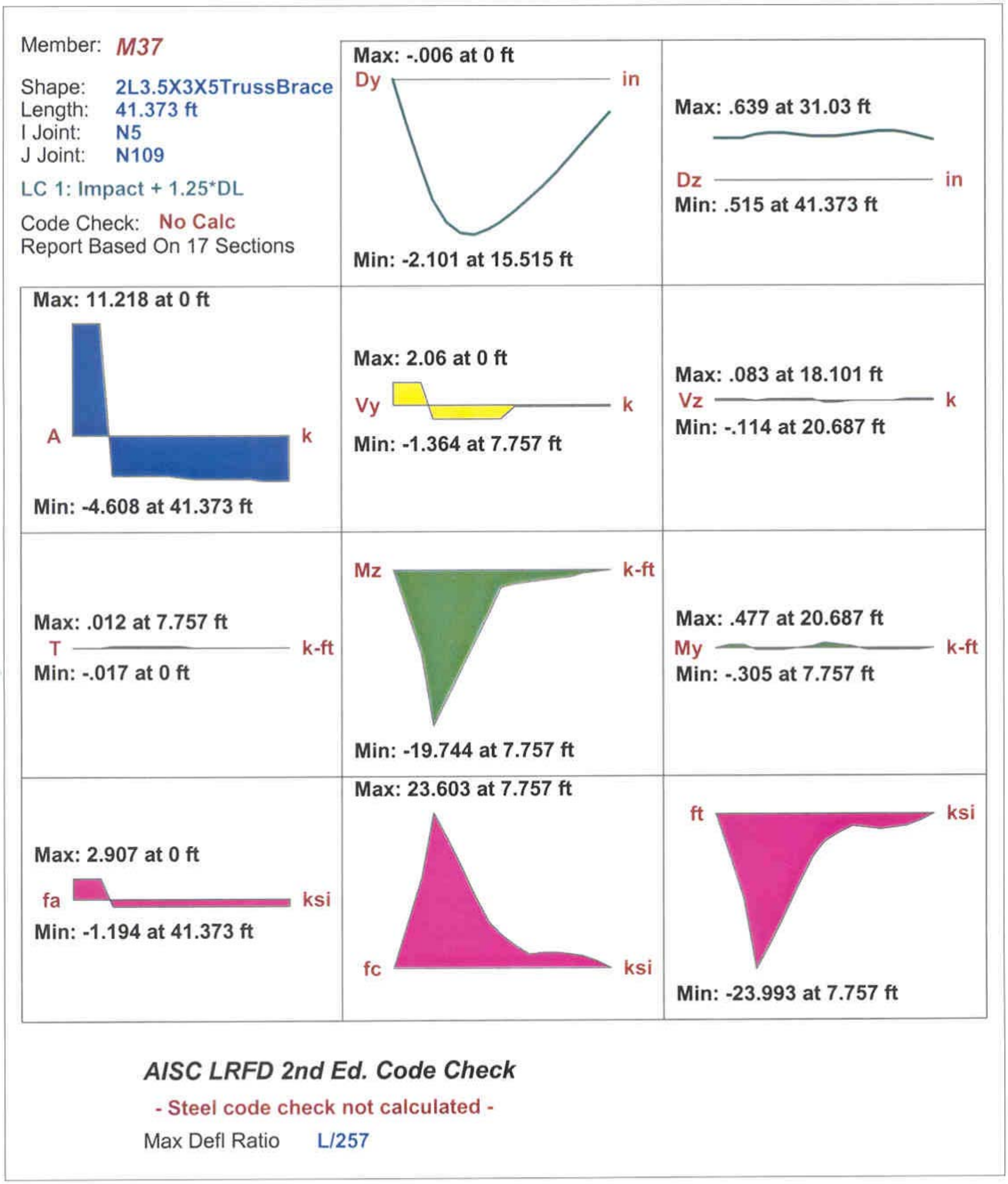
State Highway 16, Roy B. Inks Bridge over Llano River

Three-Dimensional RISA-3D Structural Model and Analysis Results

Load Applied at Ninth Truss Member (Node 9)



	Llano Truss Bridge Structural Analysis	
William Williams, P.E.		June 10, 2005 at 4:49 PM
	TL-2 Impact Loads on Member L4-U4 Diag. Brace	Llano_RisaModel9.r3d



Member: *M37*

Shape: 2L3.5X3X5TrussBrace
Length: 41.373 ft
I Joint: N5
J Joint: N109

LC 1: Impact + 1.25*DL

Code Check: No Calc
Report Based On 17 Sections

Max: -.006 at 0 ft
Min: -2.101 at 15.515 ft

Max: .639 at 31.03 ft
Min: .515 at 41.373 ft

Max: 11.218 at 0 ft
Min: -4.608 at 41.373 ft

Max: 2.06 at 0 ft
Min: -1.364 at 7.757 ft

Max: .083 at 18.101 ft
Min: -.114 at 20.687 ft

Max: .012 at 7.757 ft
Min: -.017 at 0 ft

Min: -19.744 at 7.757 ft

Max: .477 at 20.687 ft
Min: -.305 at 7.757 ft

Max: 2.907 at 0 ft
Min: -1.194 at 41.373 ft

Max: 23.603 at 7.757 ft

Min: -23.993 at 7.757 ft

AISC LRFD 2nd Ed. Code Check

- Steel code check not calculated -

Max Defl Ratio L/257

TEXAS TRANSPORTATION INSTITUTE

Texas Transportation Institute
Texas A&M University System
College Station, Texas

PAGE 1 of 2
JOB NO. 444193
Date: 07-27-03

SUBJECT *Llano S.H. 16 Truss Bridge*
LRFD Beam Check: NODE 9
CLIENT *TTI*

BY: *W. Williams*
CKD:

Beam Column Tension Analysis in Accordance with Chapter H, LRFD

1.) Load Data:

$P_u := 11.2 \cdot \text{kip}$ **"Tension"**

$M_z := 19.7 \cdot \text{kip} \cdot \text{ft}$

$M_y := 0.30 \cdot \text{kip} \cdot \text{ft}$

2.) Member Data: 2~L3.5x3.0x5/16

$S_y := 1.93 \cdot \text{in}^3$ $S_z := 10.56 \cdot \text{in}^3$ $A_g := 3.86 \cdot \text{in}^2$

$Z_y := 3.72\text{in}^3$ $Z_z := 12.68\text{in}^3$

$K_y := 1.0$ $K_z := 1.0$

$L_y := 40\text{ft}$ $L_z := 34\text{ft}$

$r_y := 1.103\text{in}$ $r_z := 3.31\text{in}$

$E := 29000 \cdot \text{ksi}$ $F_y := 30 \cdot \text{ksi}$ $F_u := 50\text{ksi}$ $\phi_b := 0.9$ *(Ch. "F" LRFD)*

3.) Calculate the Nominal Axial Tension Strength Based on Chapter D, LRFD

D1-1: For Yielding in the Gross Section:

$\phi_{tG} := 0.9$

$P_{nGross} := F_y \cdot A_g$

$\phi P_{nG} := \phi_{tG} \cdot P_{nGross}$

$\phi P_{nG} = 104.22\,\text{kip}$

D1-2: For Fracture in the net Section (Assume 2 ~ 7/8" Dia. holes):

$\phi_{tN} := 0.75$ $A_{Net} := A_g - 2 \cdot (0.3125\text{in} \cdot 0.875\text{in})$

$A_{Net} = 3.313\,\text{in}^2$

$P_{nNet} := F_u \cdot A_{Net}$

$\phi P_{nN} := \phi_{tN} \cdot P_{nNet}$

$\phi P_{nN} = 124.242\,\text{kip}$

SUBJECT Llano S.H. 16 Truss Bridge
LRFD Beam Check: NODE 9

BY: W. Williams
CKD:

CLIENT TTI

Take the lesser of the two above ..... therefore:

$$\phi P_n := \phi P_{nG}$$

4.) Determine $P_u / \phi P_n$ Ratio:

$$\frac{P_u}{\phi P_n} = 0.107$$

.... therefore ....

$$M_{uZ} := M_z$$

$$M_{uY} := M_y$$

$$M_{nZ} := F_y \cdot Z_z$$

$$M_{nY} := F_y \cdot Z_y$$

...... and ...

$$\frac{P_u}{2 \cdot \phi P_n} + \frac{8}{8} \cdot \left(\frac{M_{uZ}}{\phi_b \cdot M_{nZ}} + \frac{M_{uY}}{\phi_b \cdot M_{nY}} \right) = 0.78$$

...... O.K.... Less than 1.0 !!

GLOBAL DEFINITION OF UNITS:

$g = 32.174 \frac{ft}{sec^2}$

$plf \equiv \frac{lb}{ft}$

$kip \equiv 1000 \cdot lb$

$pcf \equiv \frac{lb}{ft^3}$

$psi \equiv \frac{lb}{in^2}$

$psf \equiv \frac{lb}{ft^2}$

$klf \equiv \frac{kip}{ft}$

$ksi \equiv \frac{kip}{in^2}$

$kcf \equiv \frac{kip}{ft^3}$

APPENDIX D. TEST VEHICLE PROPERTIES AND INFORMATION

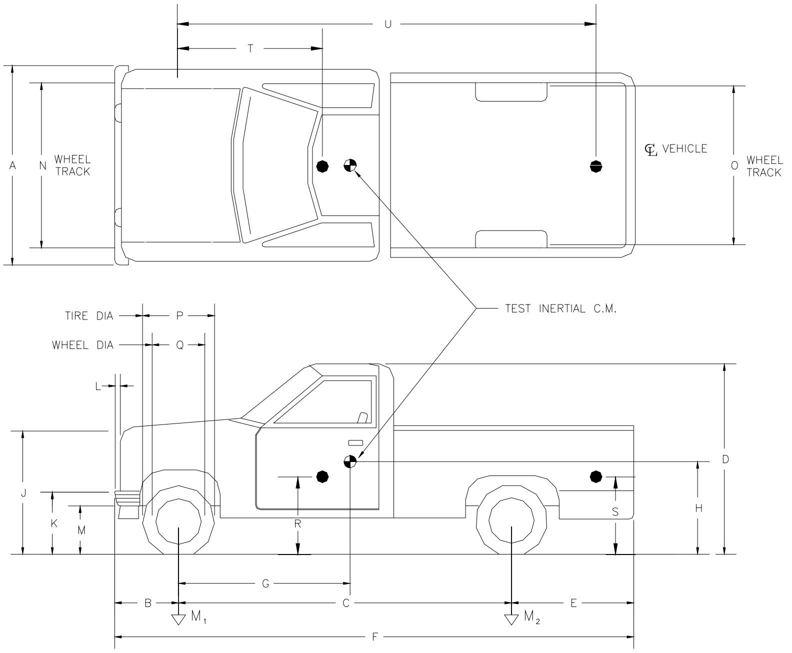
DATE: 05/30/03 TEST NO.: 444193-1 VIN NO.: 1GCGC24R4WZ145024

YEAR: 1998 MAKE: Chevrolet MODEL: Cheyenne 2500 Pickup

TIRE INFLATION PRESSURE: ________ ODOMETER: 143426 TIRE SIZE: 245 75 R16

MASS DISTRIBUTION (kg) LF 598 RF 598 LR 446 RR 437

DESCRIBE ANY DAMAGE TO VEHICLE PRIOR TO TEST:



● Denotes accelerometer location.

NOTES: ________

ENGINE TYPE: V-8

ENGINE CID: 5.7 L

TRANSMISSION TYPE:

X AUTO

__ MANUAL

OPTIONAL EQUIPMENT:

DUMMY DATA:

TYPE: ________

MASS: ________

SEAT POSITION: ________

GEOMETRY – (mm)

A	1880	E	1310	J	1038	N	1590	R	750
B	810	F	5470	K	635	O	1610	S	900
C	3350	G	1422.8	L	70	P	725	T	1460
D	1820	H		M	415	Q	440	U	3360

MASS – (kg)	CURB	TEST INERTIAL	GROSS STATIC
M_1	1210	1196	
M_2	807	883	
M_T	2017	2079	

Figure 55. Vehicle Properties for Test 444193-1.

Table 7. Exterior Crush Measurements for Test 444193-1.

VEHICLE CRUSH MEASUREMENT SHEET[1]

Complete When Applicable	
End Damage	Side Damage
Undeformed end width ________	Bowing: B1 _____ X1 _____
Corner shift: A1 ________	B2 _____ X2 _____
A2 ________	
End shift at frame (CDC)	Bowing constant
(check one)	$\frac{X1+X2}{2} =$ _______
< 4 inches ________	
$\geq$ 4 inches ________	

Note: Measure C_1 to C_6 from Driver to Passenger side in Front or Rear impacts – Rear to Front in Side Impacts.

Specific Impact Number	Plane* of C-Measurements	Direct Damage		Field L**	C_1	C_2	C_3	C_4	C_5	C_6	±D
		Width** (CDC)	Max*** Crush								
1	At front bumper	1000	360	720	0	50	130	240	300	360	-360
2	At front bumper	1000	400	1270	0	85	N/A		330	400	+1660

[1] Table taken from National Accident Sampling System (NASS).

*Identify the plane at which the C-measurements are taken (e.g., at bumper, above bumper, at sill, above sill, at beltline, etc.) or label adjustments (e.g., free space).

Free space value is defined as the distance between the baseline and the original body contour taken at the individual C locations. This may include the following: bumper lead, bumper taper, side protrusion, side taper, etc.
Record the value for each C-measurement and maximum crush.

**Measure and document on the vehicle diagram the beginning or end of the direct damage width and field L (e.g., side damage with respect to undamaged axle).

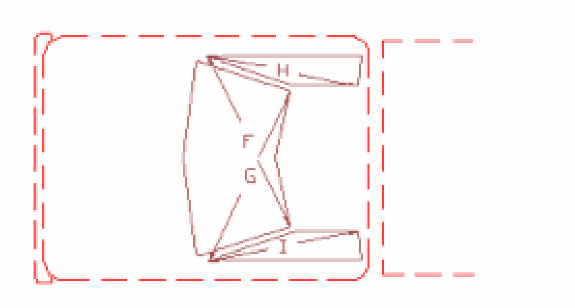
***Measure and document on the vehicle diagram the location of the maximum crush.

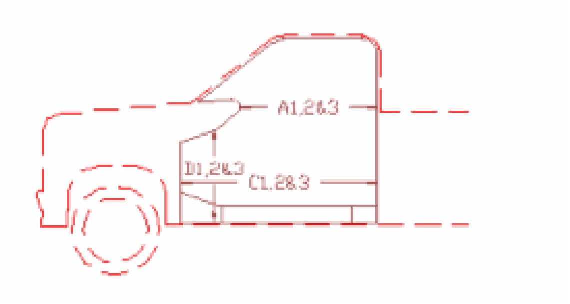
Note: Use as many lines/columns as necessary to describe each damage profile.

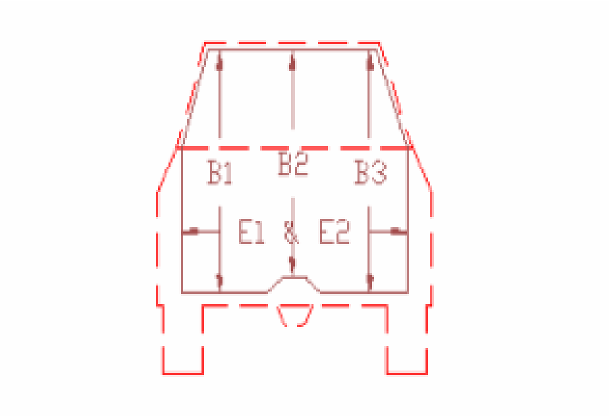
Table 8. Occupant Compartment Measurements for Test 444193-1.

Truck

Occupant Compartment Deformation







	BEFORE	AFTER
A1	870	873
A2	946	941
A3	932	931
B1	1075	1075
B2	1002	998
B3	1072	1067
C1	1375	1375
C2		
C3	1370	1370
D1	329	329
D2	160	149
D3	310	304
E1	1590	1590
E2	1590	1592
F	1470	1470
G	1470	1470
H	1250	1250
I	1240	1240
J*	1523	1513

*Lateral area across the cab from
driver's side kickpanel to passenger's side kickpanel.

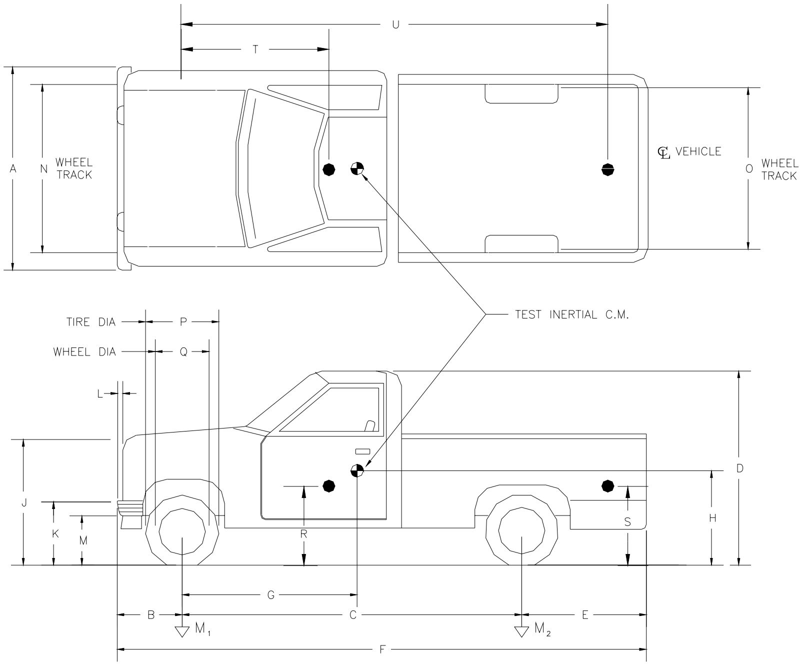
DATE: 07/30/03 TEST NO.: 444193-2 VIN NO.: 1GCGC24R8XR702843

YEAR: 1999 MAKE: Chevrolet MODEL: Cheyenne 2500 P/U

TIRE INFLATION PRESSURE: ______ ODOMETER: 204739 TIRE SIZE: 245 75R16

MASS DISTRIBUTION (kg) LF 594 RF 586 LR 435 RR 444

DESCRIBE ANY DAMAGE TO VEHICLE PRIOR TO TEST:



● Denotes accelerometer location.

NOTES: ______

ENGINE TYPE: V-8

ENGINE CID: 5.7 L

TRANSMISSION TYPE:

_ AUTO

X MANUAL

OPTIONAL EQUIPMENT:

8 LUGS

DUMMY DATA:

TYPE: ______

MASS: ______

SEAT POSITION: ______

GEOMETRY - (mm)

A	1880	E	1310	J	1038	N	1590	R	750
B	810	F	5470	K	635	O	1610	S	900
C	3350	G	1430.1	L	70	P	725	T	1460
D	1820	H		M	415	Q	440	U	3360

MASS - (kg)	CURB	TEST INERTIAL	GROSS STATIC
M_1	1225	1180	
M_2	880	879	
M_T	2105	2059	

Figure 56. Vehicle Properties for Test 444193-2.

Table 9. Exterior Crush Measurements for Test 444193-2.

VEHICLE CRUSH MEASUREMENT SHEET[1]

Complete When Applicable

End Damage	Side Damage
Undeformed end width ________	Bowing: B1 _____ X1 _____
Corner shift: A1 ________	B2 _____ X2 _____
A2 ________	
End shift at frame (CDC)	Bowing constant
(check one)	$\frac{X1+X2}{2} =$ _______
< 4 inches ________	
≥ 4 inches ________	

Note: Measure C_1 to C_6 from Driver to Passenger side in Front or Rear impacts – Rear to Front in Side Impacts.

Specific Impact Number	Plane* of C-Measurements	Direct Damage		Field L**	C_1	C_2	C_3	C_4	C_5	C_6	±D
		Width** (CDC)	Max*** Crush								
1	At front bumper	800	550	700	0	55	140	250	350	550	+350
2	At front bumper	800	645	1600	80	125	200	N/A	480	645	+1650

[1] Table taken from National Accident Sampling System (NASS).

*Identify the plane at which the C-measurements are taken (e.g., at bumper, above bumper, at sill, above sill, at beltline, etc.) or label adjustments (e.g., free space).

Free space value is defined as the distance between the baseline and the original body contour taken at the individual C locations. This may include the following: bumper lead, bumper taper, side protrusion, side taper, etc. Record the value for each C-measurement and maximum crush.

**Measure and document on the vehicle diagram the beginning or end of the direct damage width and field L (e.g., side damage with respect to undamaged axle).

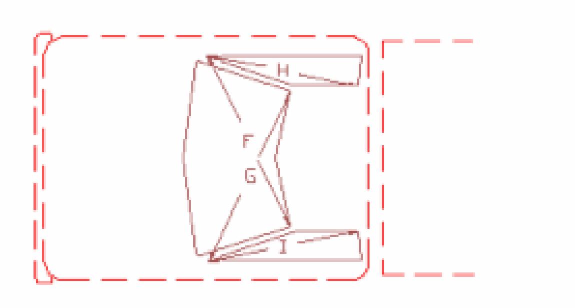
***Measure and document on the vehicle diagram the location of the maximum crush.

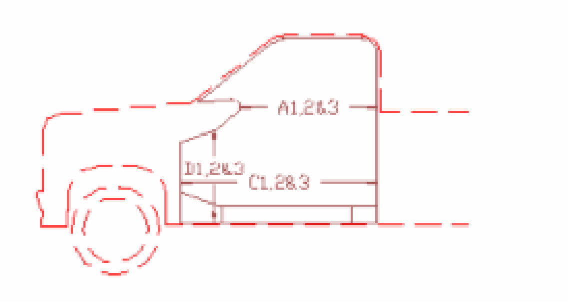
Note: Use as many lines/columns as necessary to describe each damage profile.

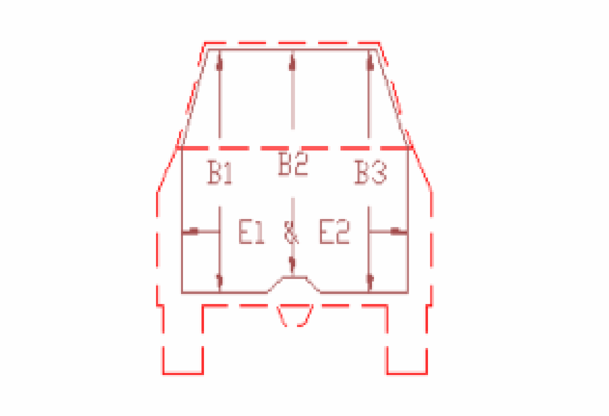
Table 10. Occupant Compartment Measurements for Test 444193-2.

Truck

Occupant Compartment Deformation



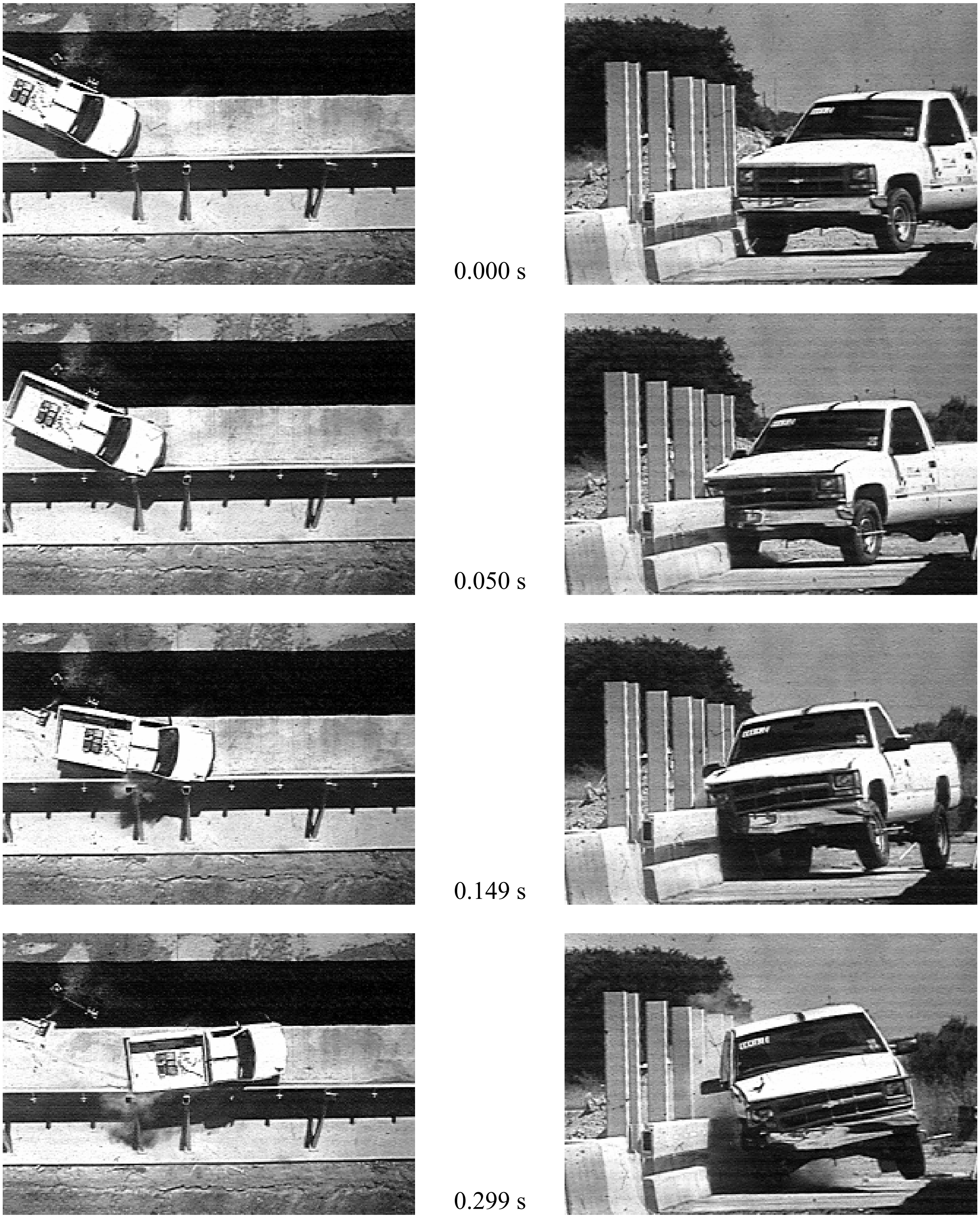




	BEFORE	AFTER
A1	865	873
A2	940	931
A3	939	923
B1	1078	1078
B2	960	930
B3	1064	1040
C1	1370	1370
C2		
C3	1360	1318
D1	324	336
D2	160	145
D3	307	330
E1	1590	1598
E2	1590	1615
F	1470	1470
G	1470	1470
H	1270	1270
I	1250	1250
J*	1523	1483

*Lateral area across the cab from
driver's side kickpanel to passenger's side kickpanel.

APPENDIX E. SEQUENTIAL PHOTOGRAPHS



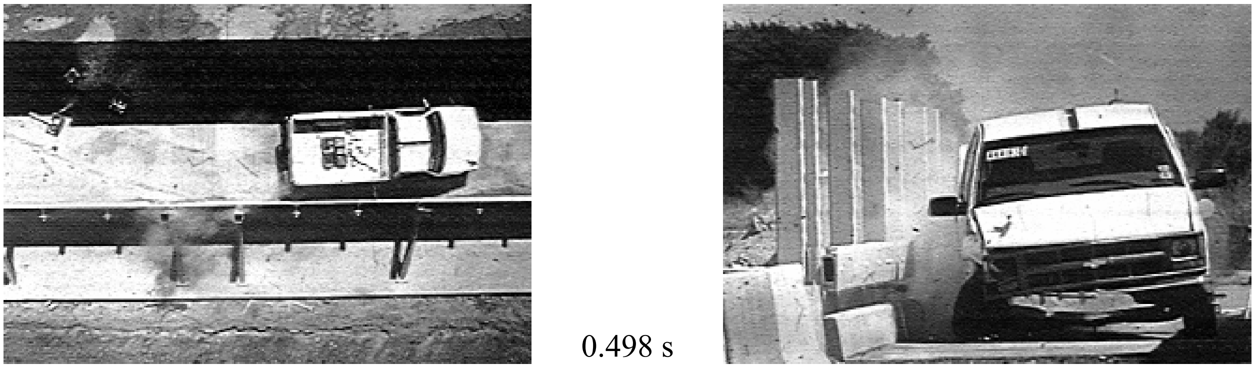
0.000 s

0.050 s

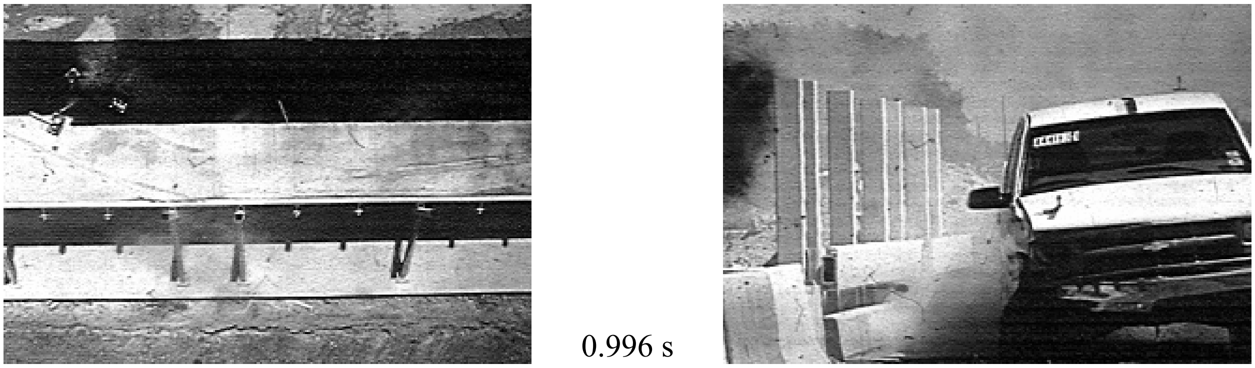
0.149 s

0.299 s

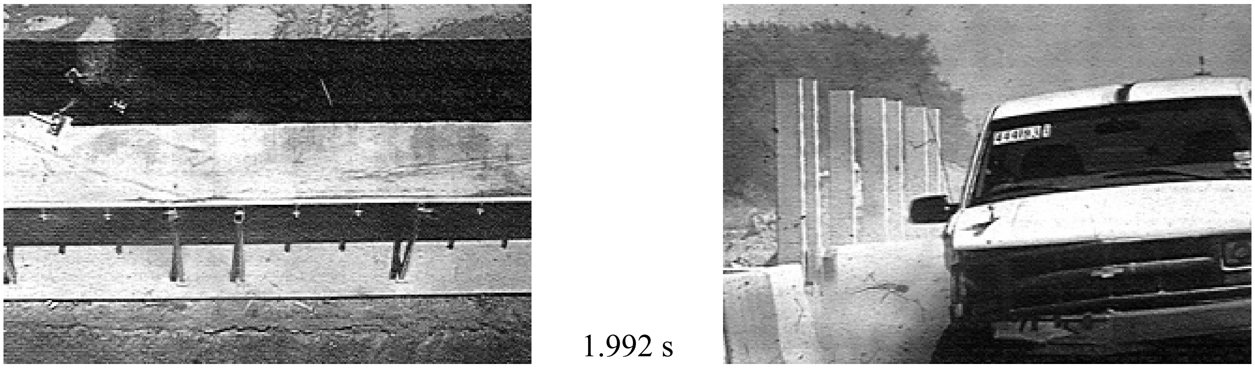
Figure 57. Sequential Photographs for Test 444193-1 (Overhead and Frontal Views).



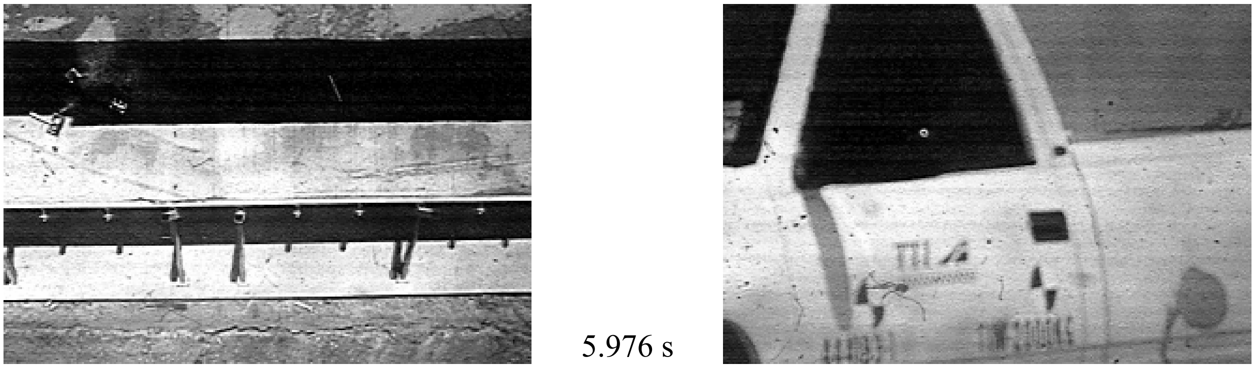
0.498 s



0.996 s

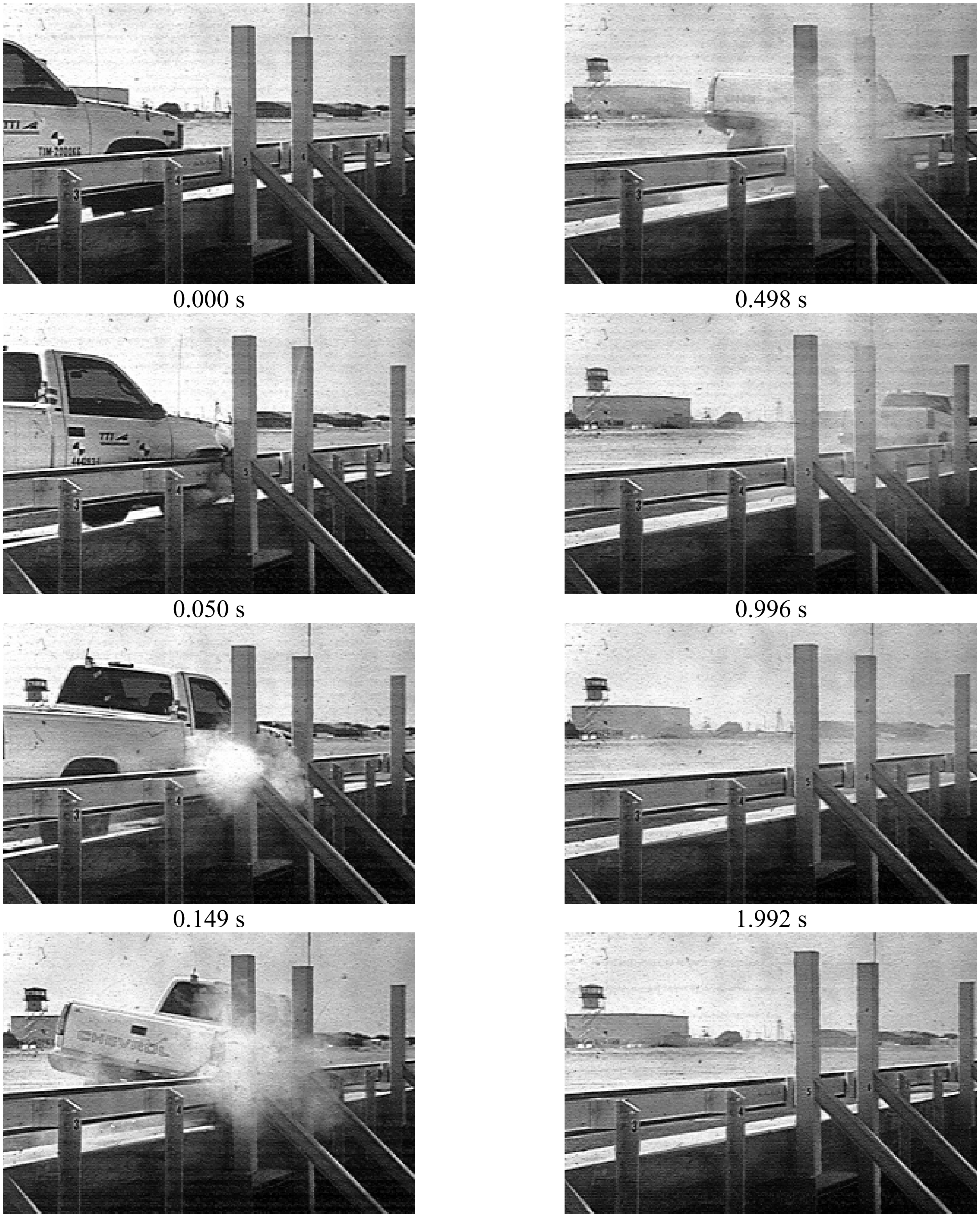


1.992 s



5.976 s

Figure 57. Sequential Photographs for Test 444193-1 (Overhead and Frontal Views) (continued).



0.000 s

0.050 s

0.149 s

0.299 s

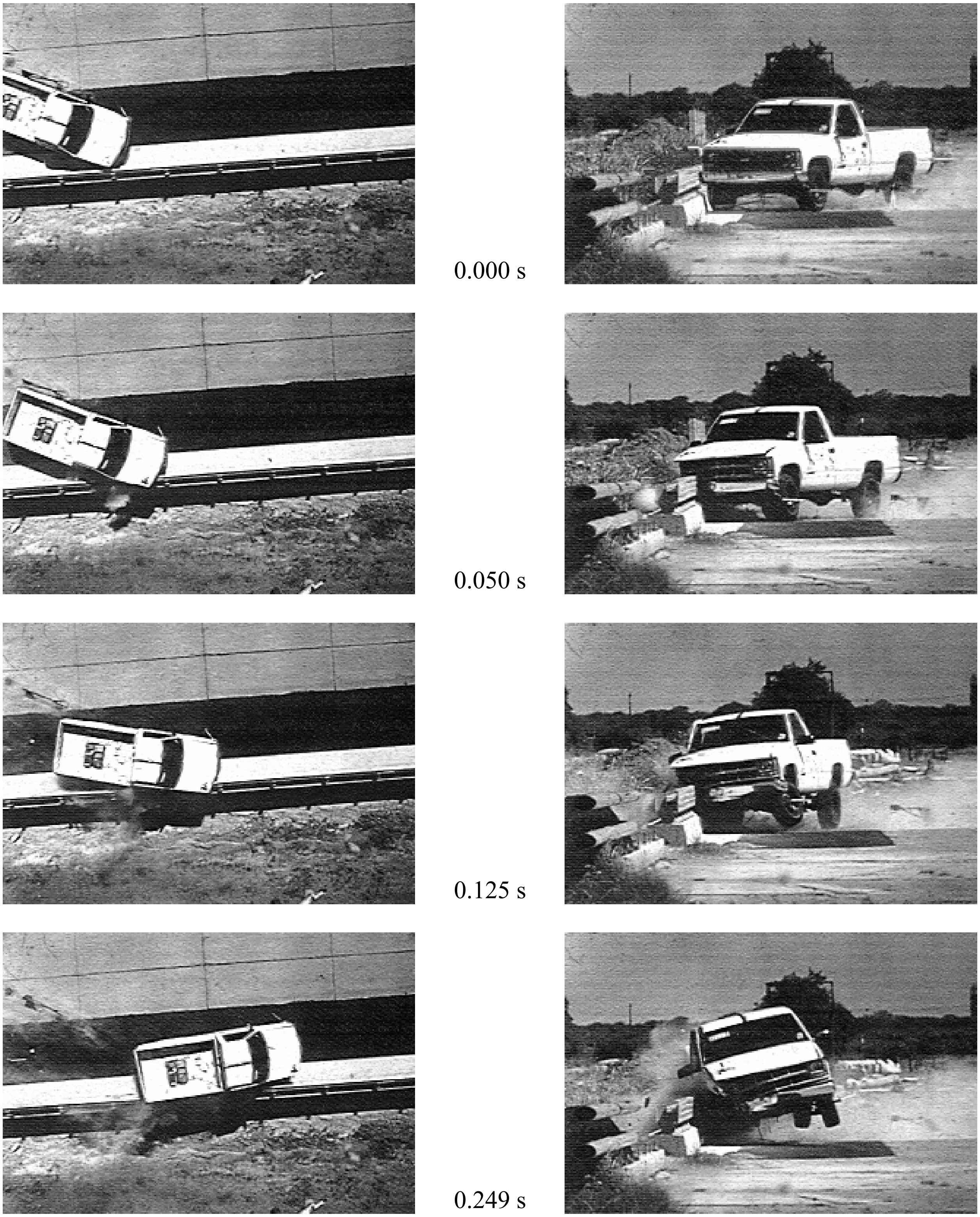
0.498 s

0.996 s

1.992 s

5.976 s

Figure 58. Sequential Photographs for Test 444193-1 (Rear View).



0.000 s

0.050 s

0.125 s

0.249 s

Figure 59. Sequential Photographs for Test 444193-2 (Overhead and Frontal Views).

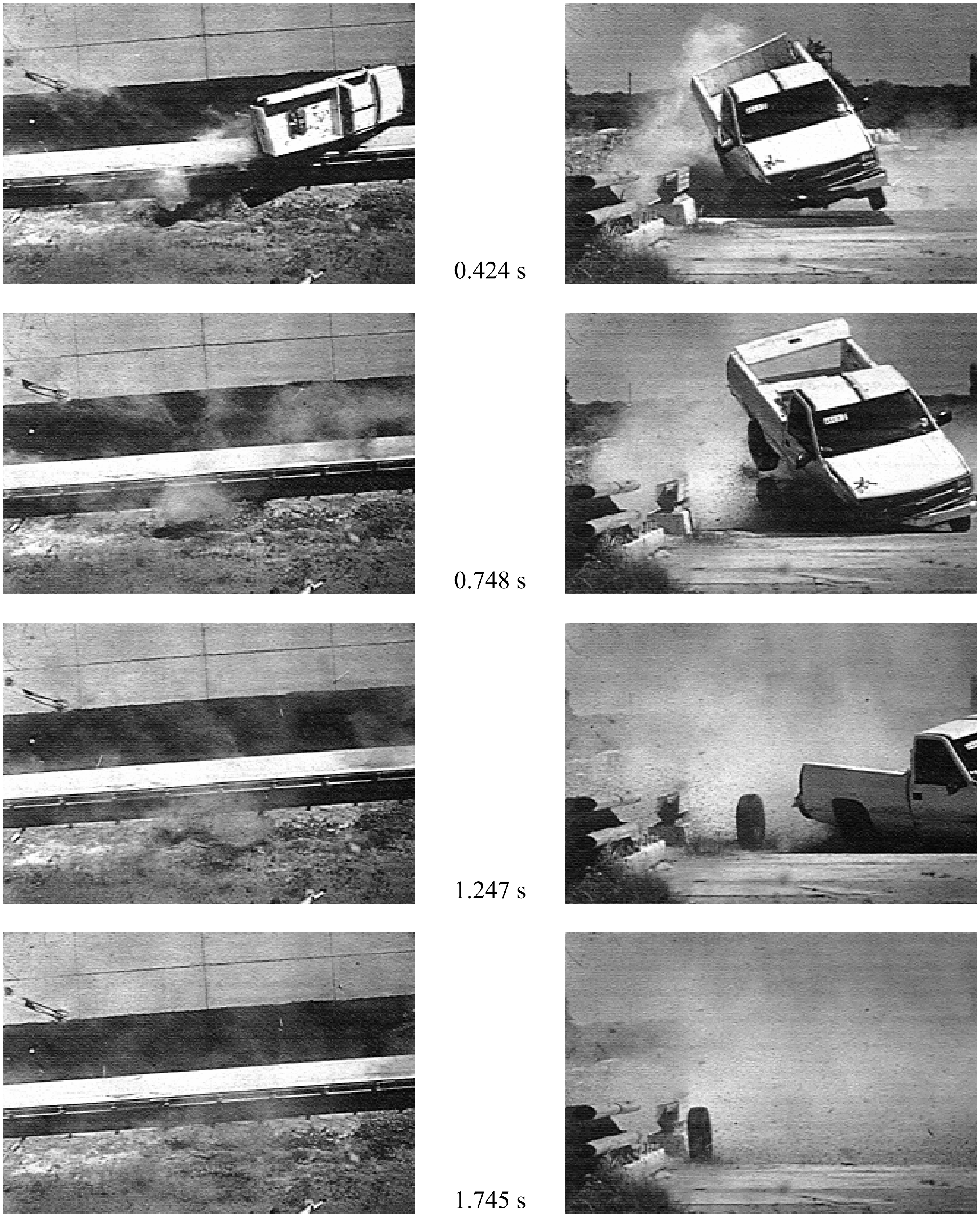
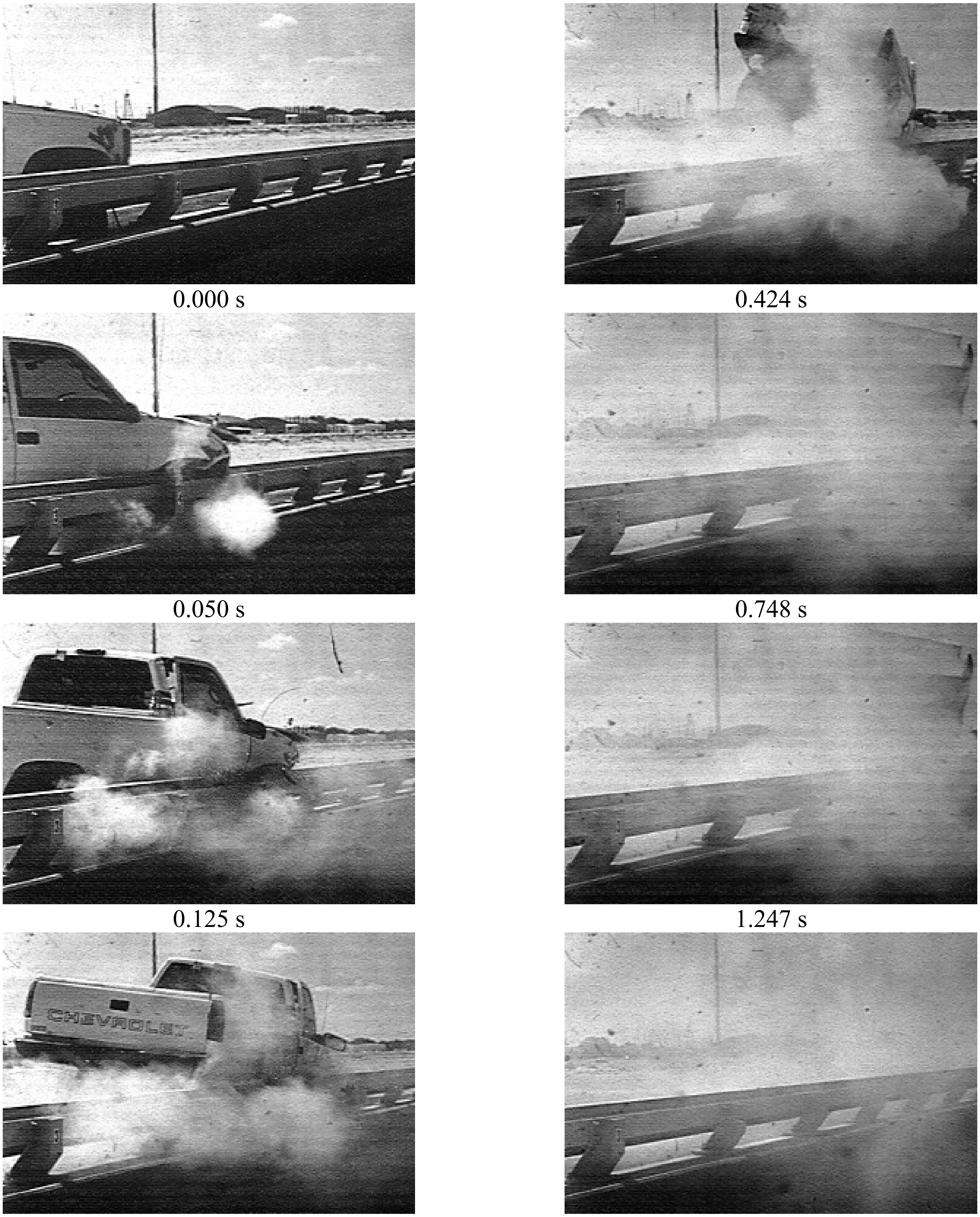


Figure 59. Sequential Photographs for Test 444193-2 (Overhead and Frontal Views) (continued).



0.000 s

0.050 s

0.125 s

0.249 s

0.424 s

0.748 s

1.247 s

1.754 s

Figure 60. Sequential Photographs for Test 444193-2 (Rear View).

APPENDIX F. VEHICLE ANGULAR DISPLACEMENTS AND ACCELERATIONS

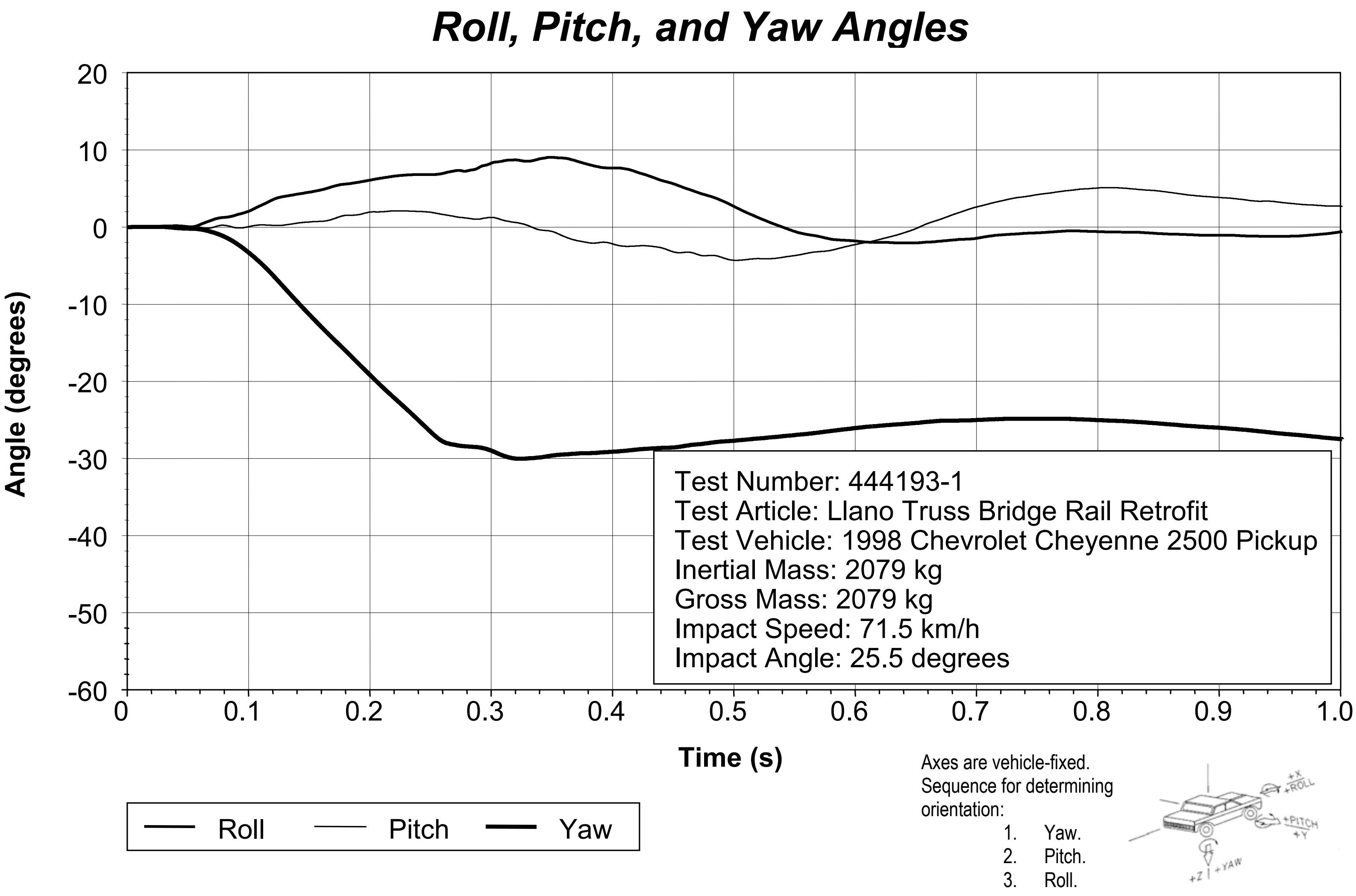


Figure 61. Vehicular Angular Displacements for Test 444193-1.

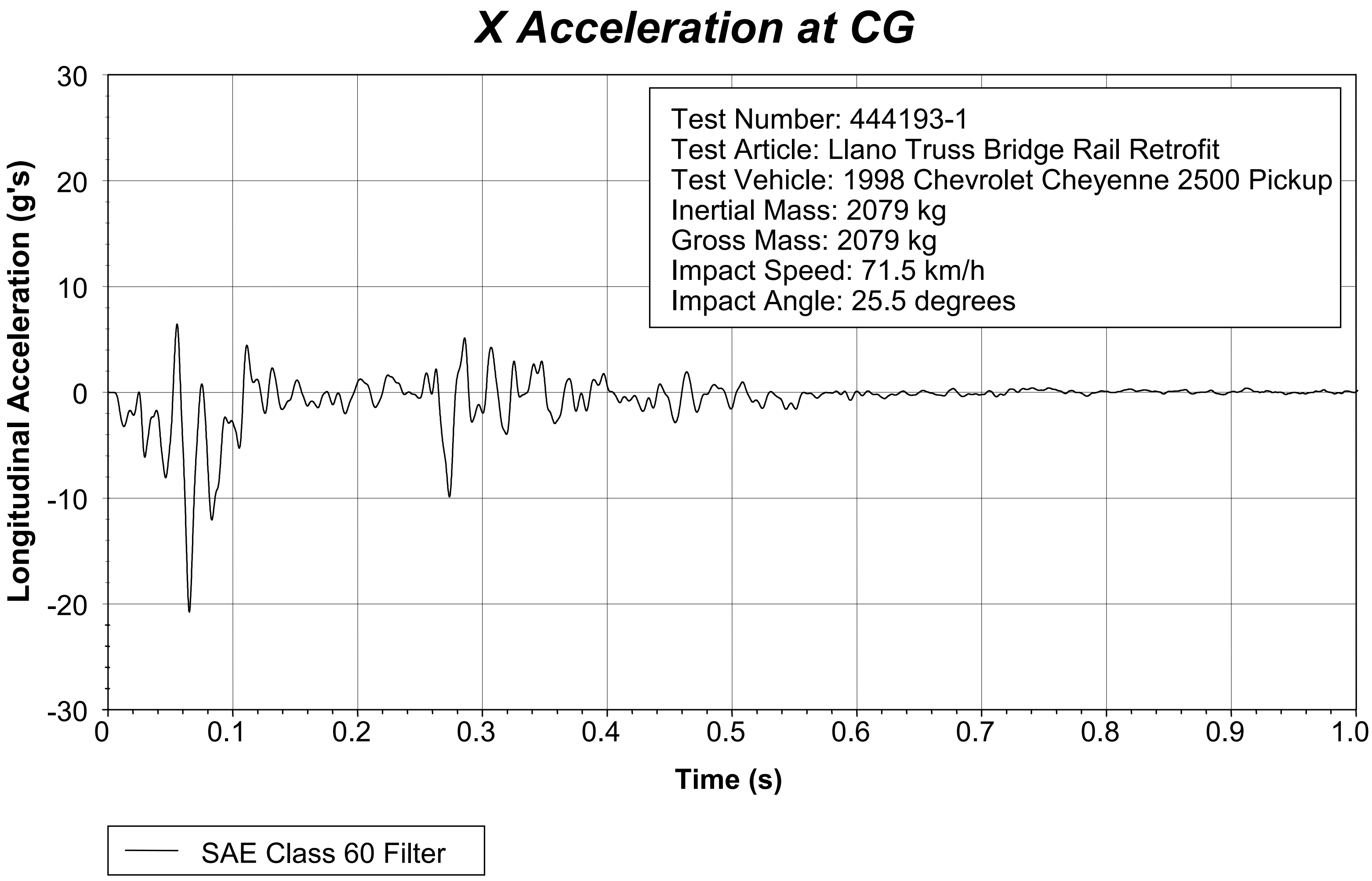


Figure 62. Vehicle Longitudinal Accelerometer Trace for Test 444193-1 (Accelerometer Located at Center of Gravity [CG]).

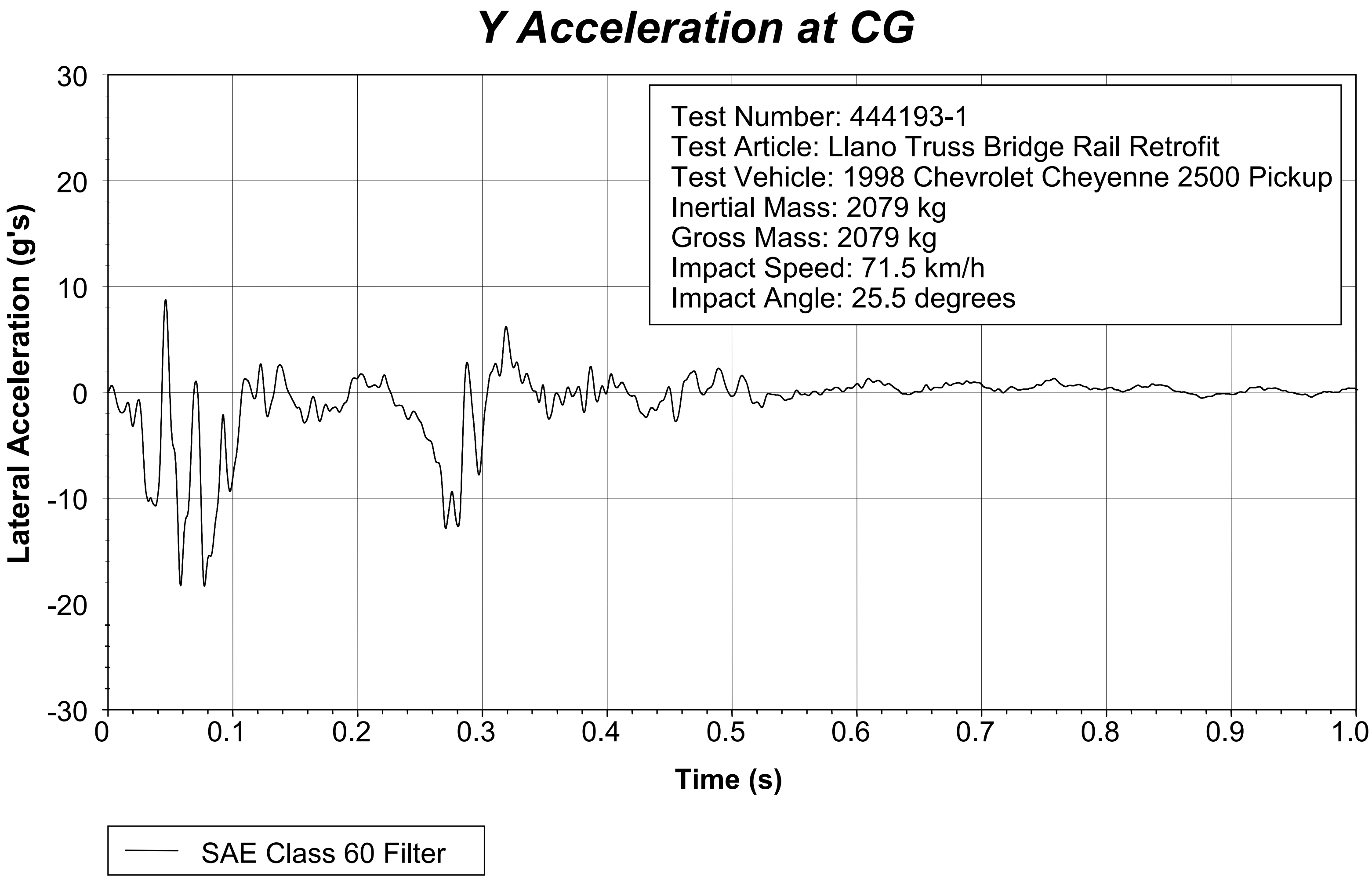


Figure 63. Vehicle Lateral Accelerometer Trace for Test 444193-1 (Accelerometer Located at Center of Gravity [CG]).

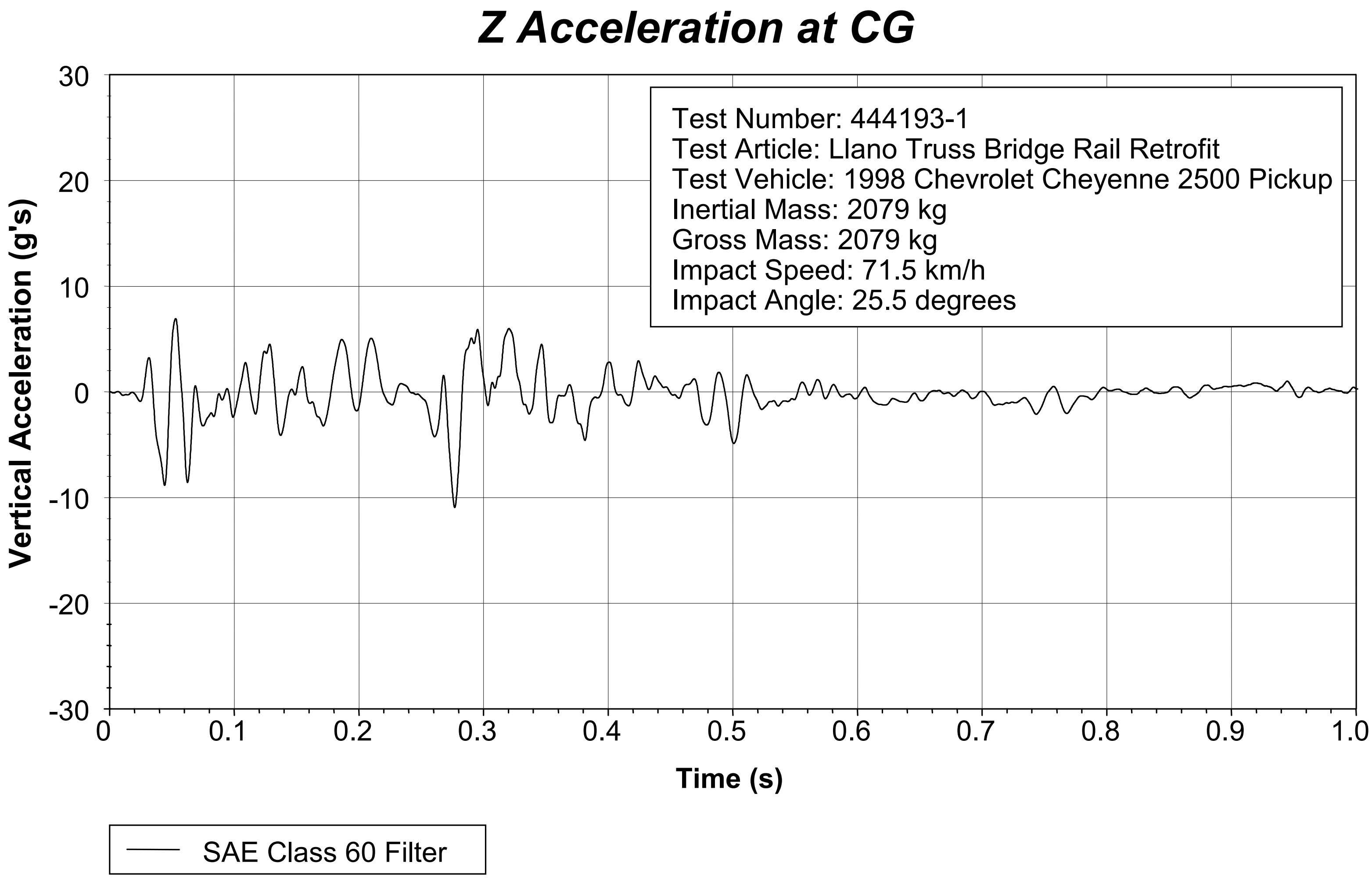


Figure 64. Vehicle Vertical Accelerometer Trace for Test 444193-1 (Accelerometer Located at Center of Gravity [CG]).

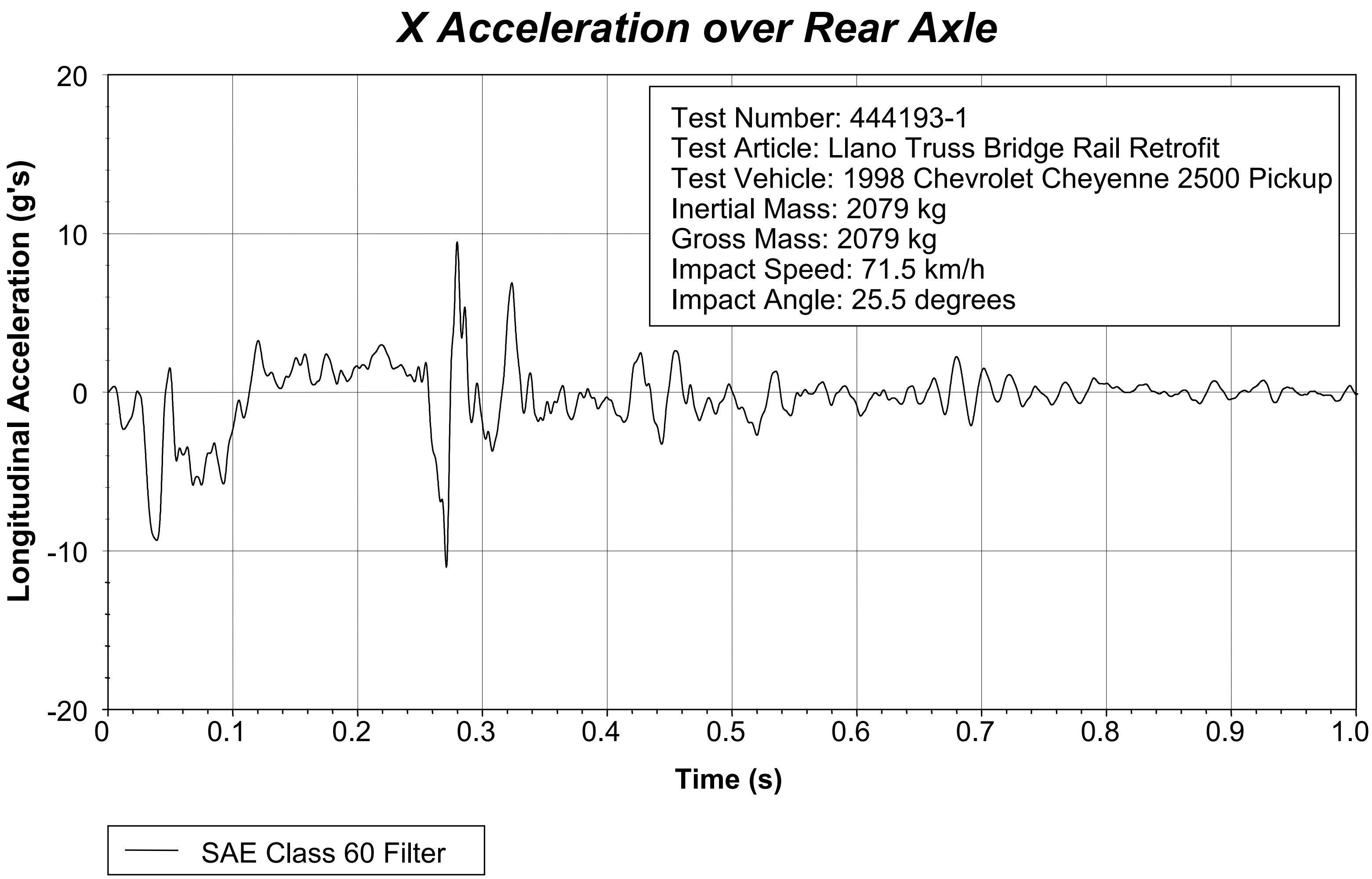


Figure 65. Vehicle Longitudinal Accelerometer Trace for Test 444193-1 (Accelerometer Located over Rear Axle).

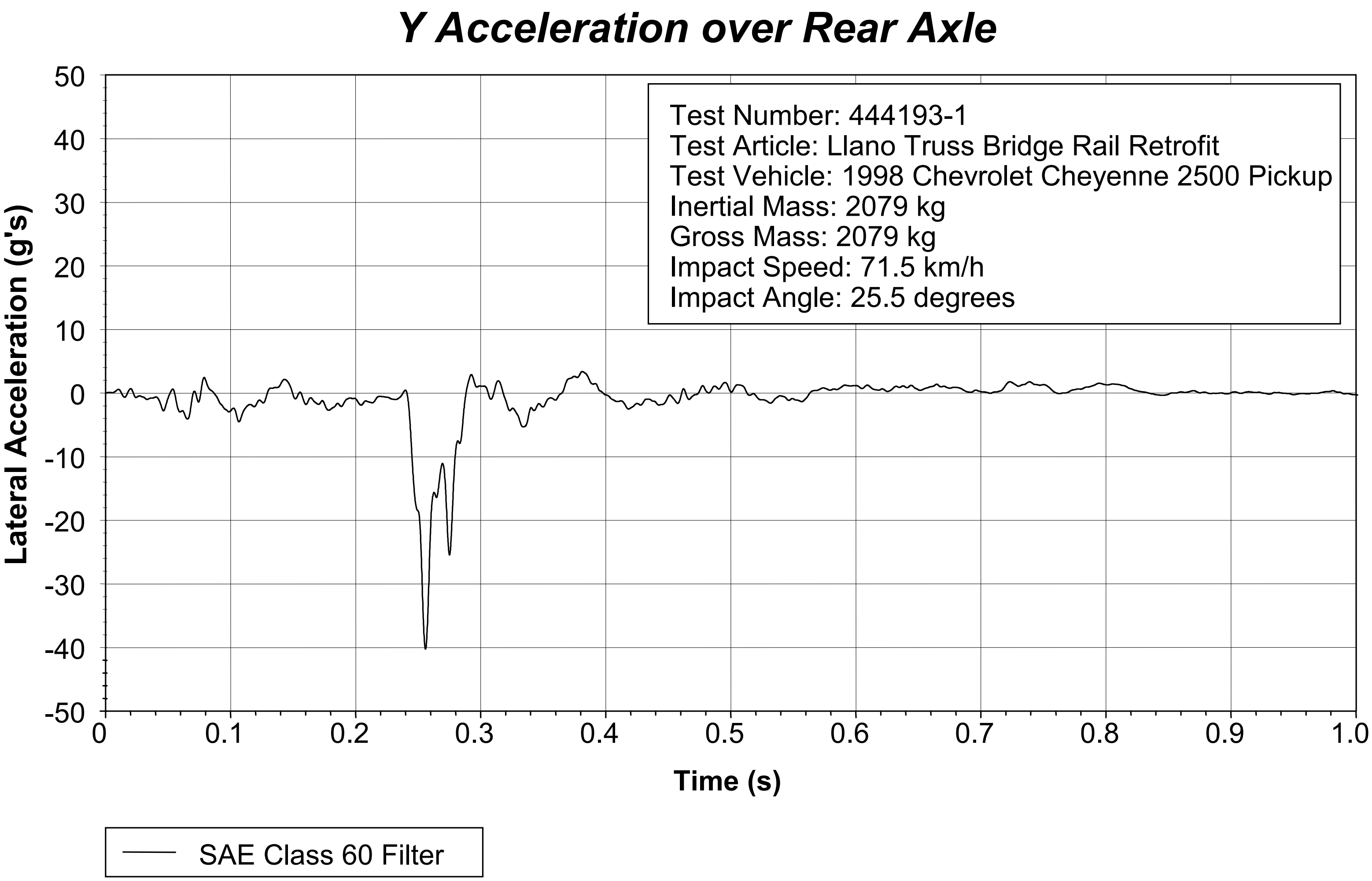


Figure 66. Vehicle Lateral Accelerometer Trace for Test 444193-1 (Accelerometer Located over Rear Axle).

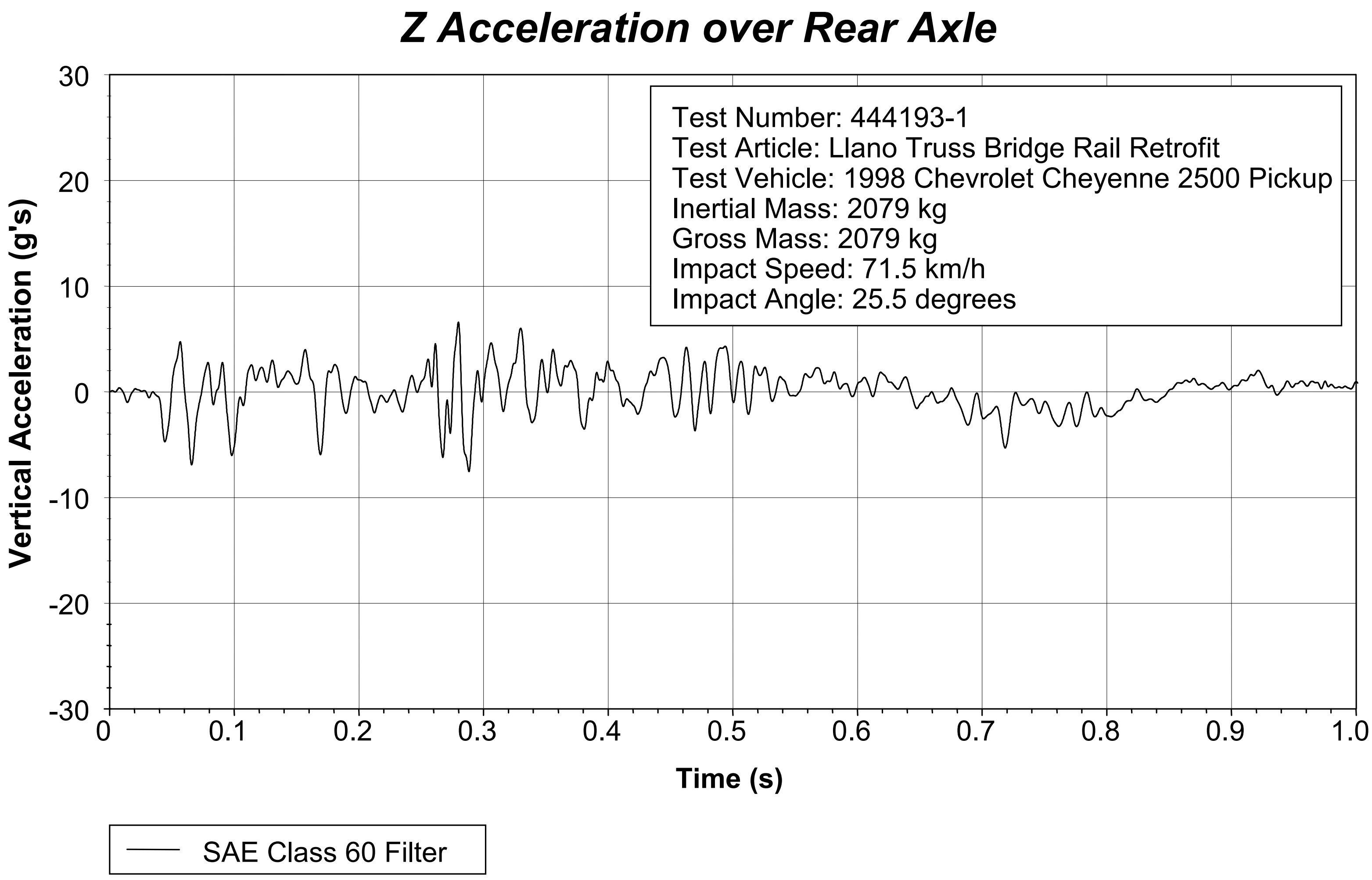


Figure 67. Vehicle Vertical Accelerometer Trace for Test 444193-1 (Accelerometer Located over Rear Axle).

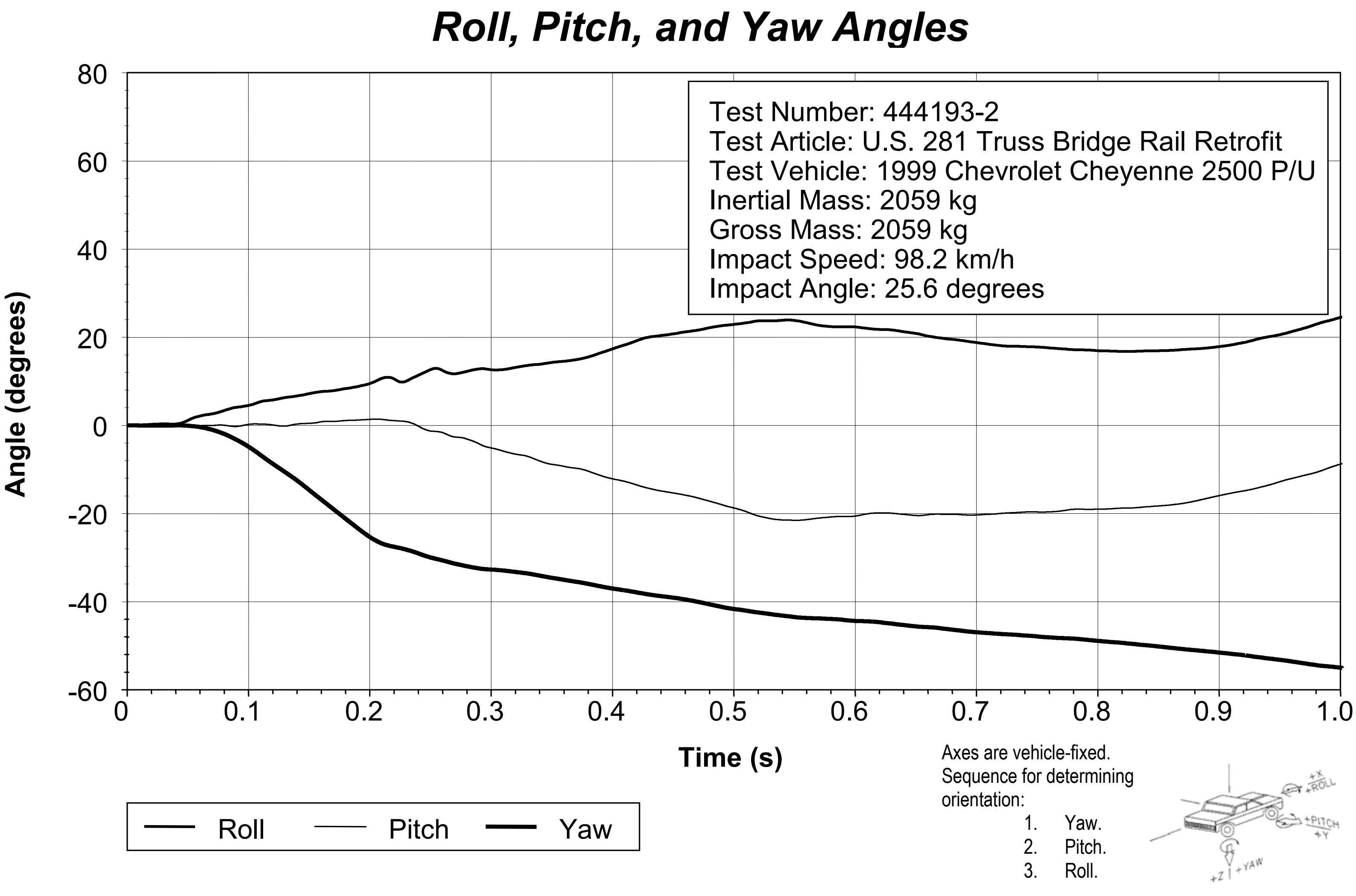


Figure 68. Vehicular Angular Displacements for Test 444193-2.

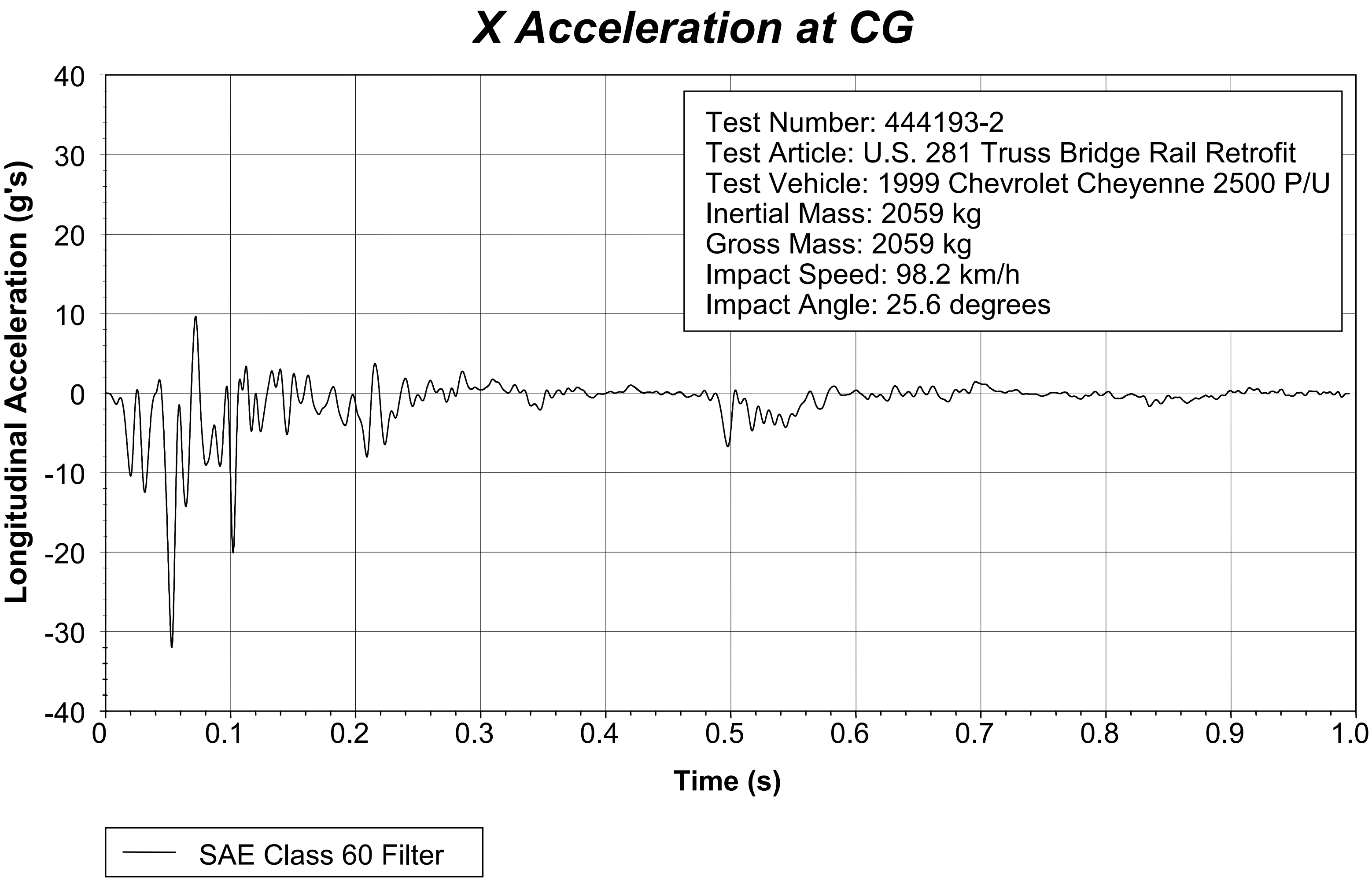


Figure 69. Vehicle Longitudinal Accelerometer Trace for Test 444193-2 (Accelerometer Located at Center of Gravity [CG]).

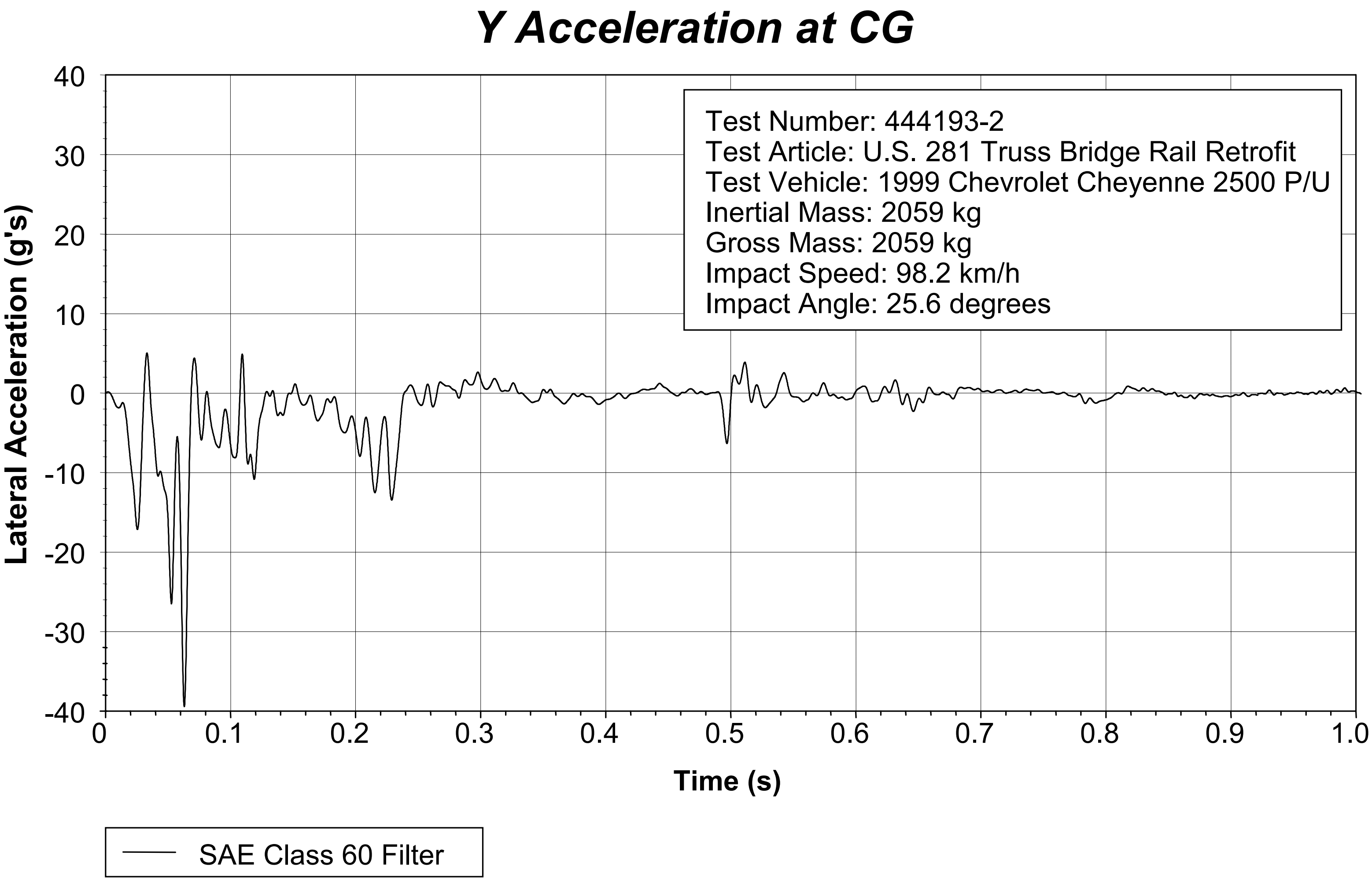


Figure 70. Vehicle Lateral Accelerometer Trace for Test 444193-2 (Accelerometer Located at Center of Gravity [CG]).

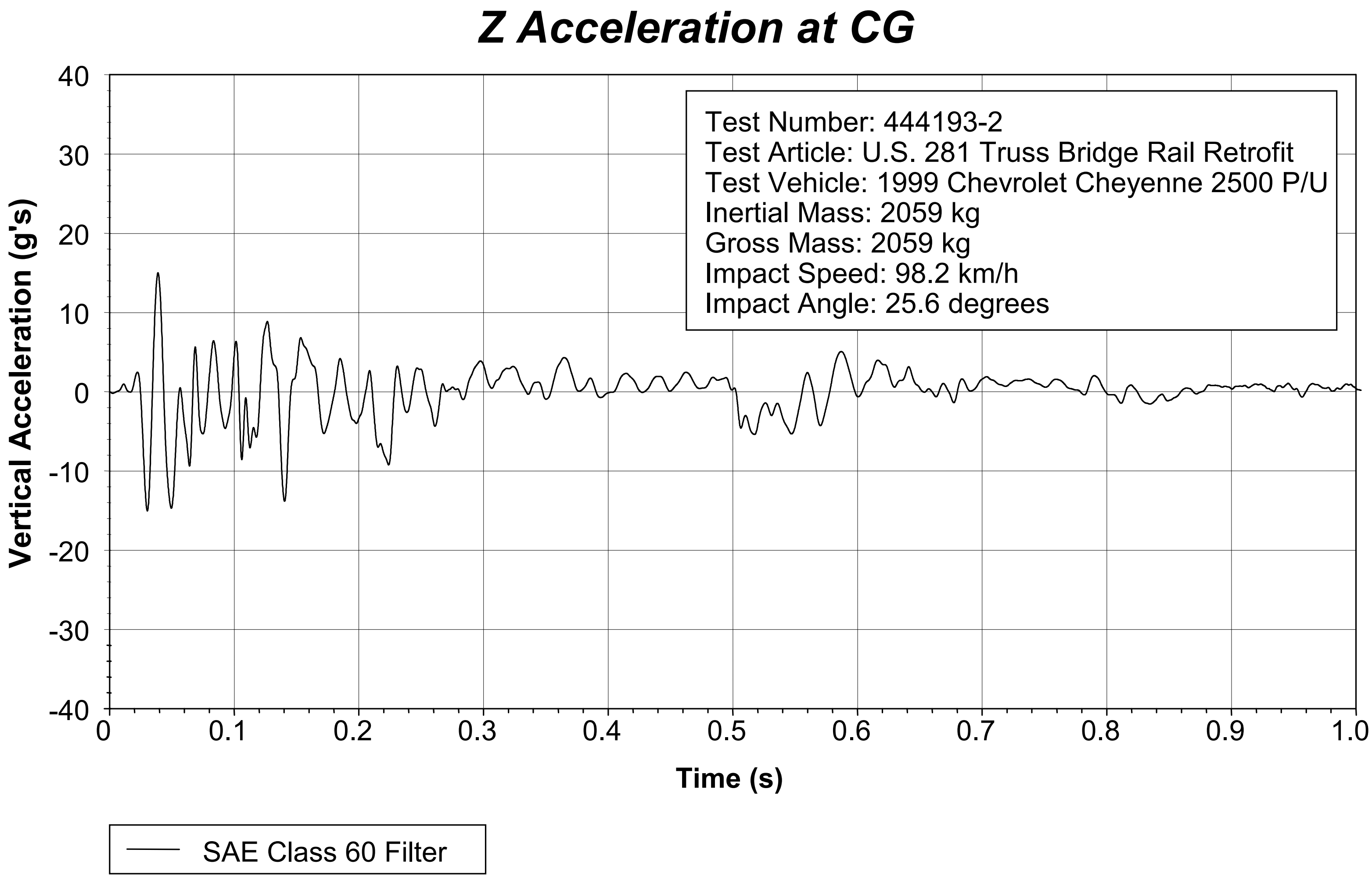


Figure 71. Vehicle Vertical Accelerometer Trace for Test 444193-2 (Accelerometer Located at Center of Gravity [CG]).

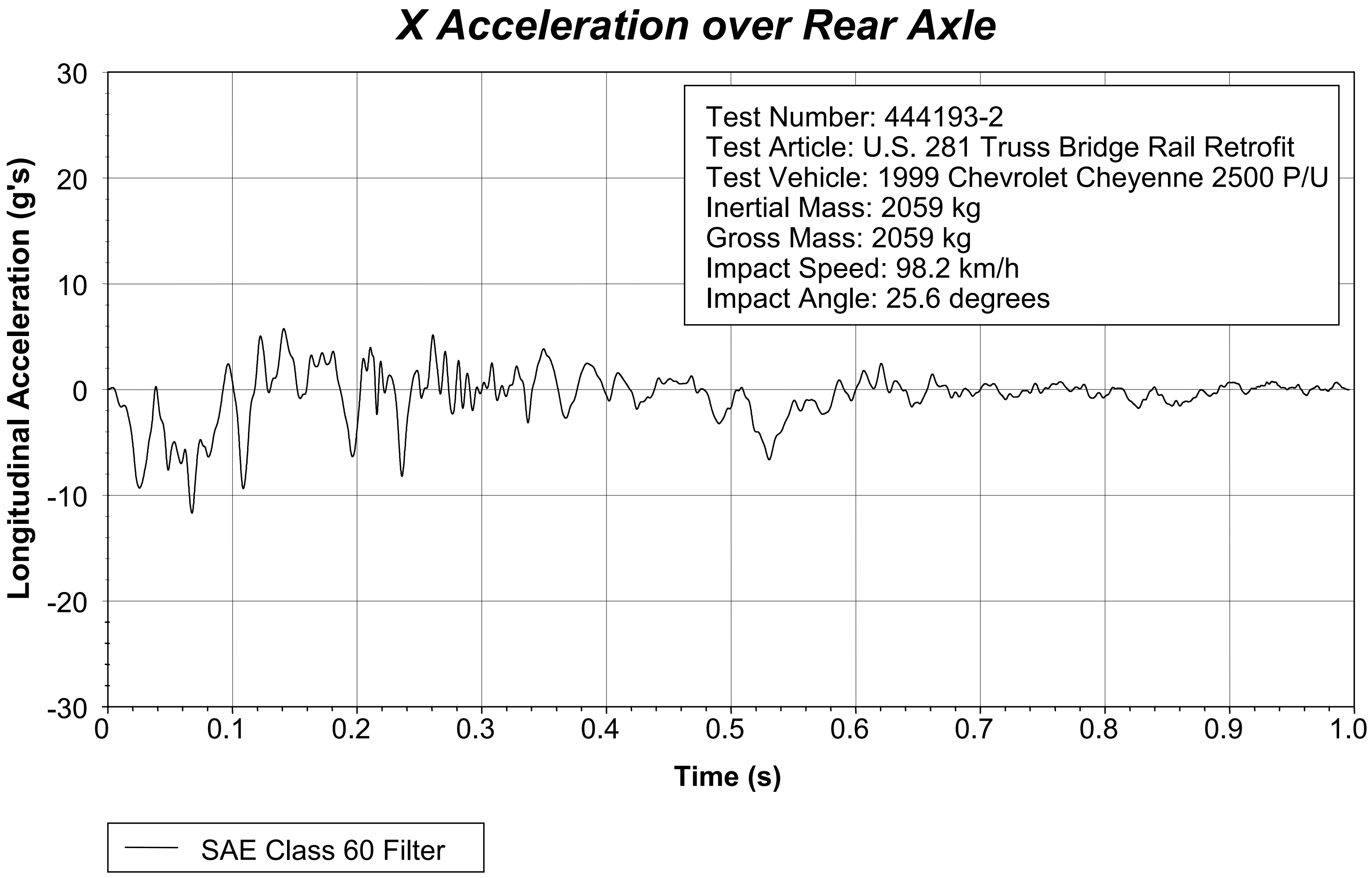


Figure 72. Vehicle Longitudinal Accelerometer Trace for Test 444193-2 (Accelerometer Located over Rear Axle).

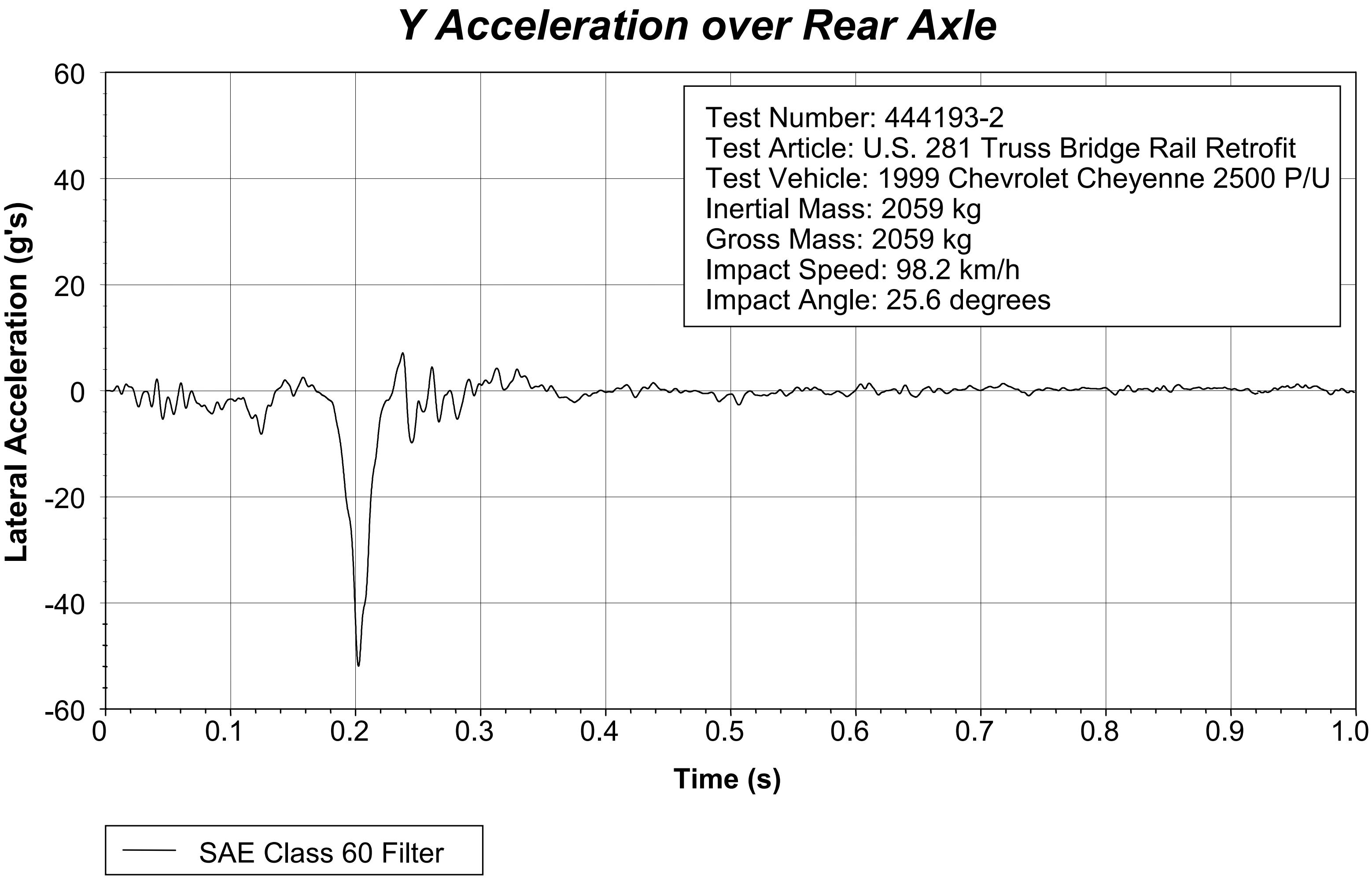


Figure 73. Vehicle Lateral Accelerometer Trace for Test 444193-2 (Accelerometer Located over Rear Axle).

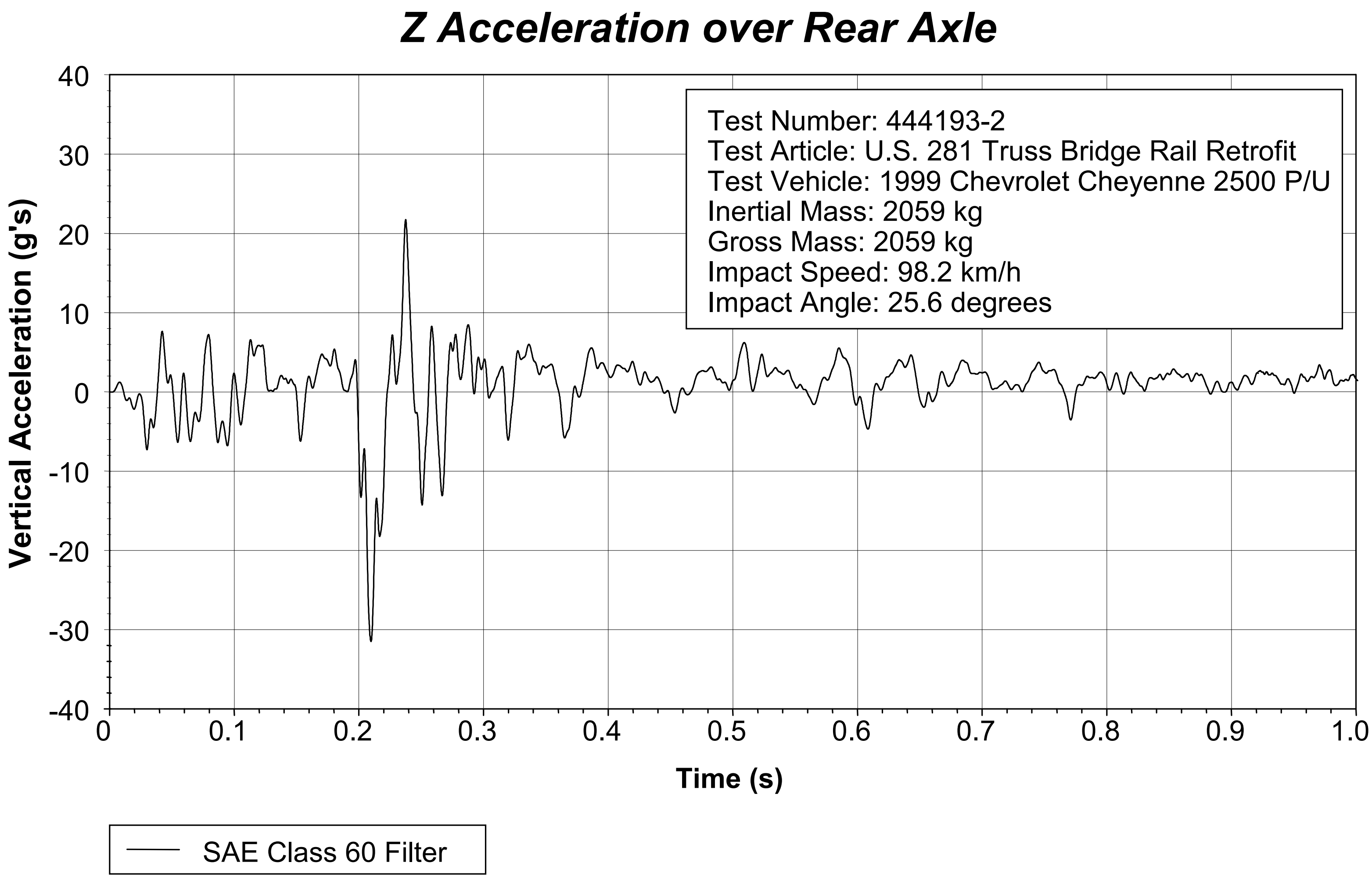


Figure 74. Vehicle Vertical Accelerometer Trace for Test 444193-2 (Accelerometer Located over Rear Axle).

APPENDIX G. DESIGN OF RETROFIT RAIL, U.S. 281 BRIDGE OVER BRAZOS RIVER, PALO PINTO COUNTY, TEXAS

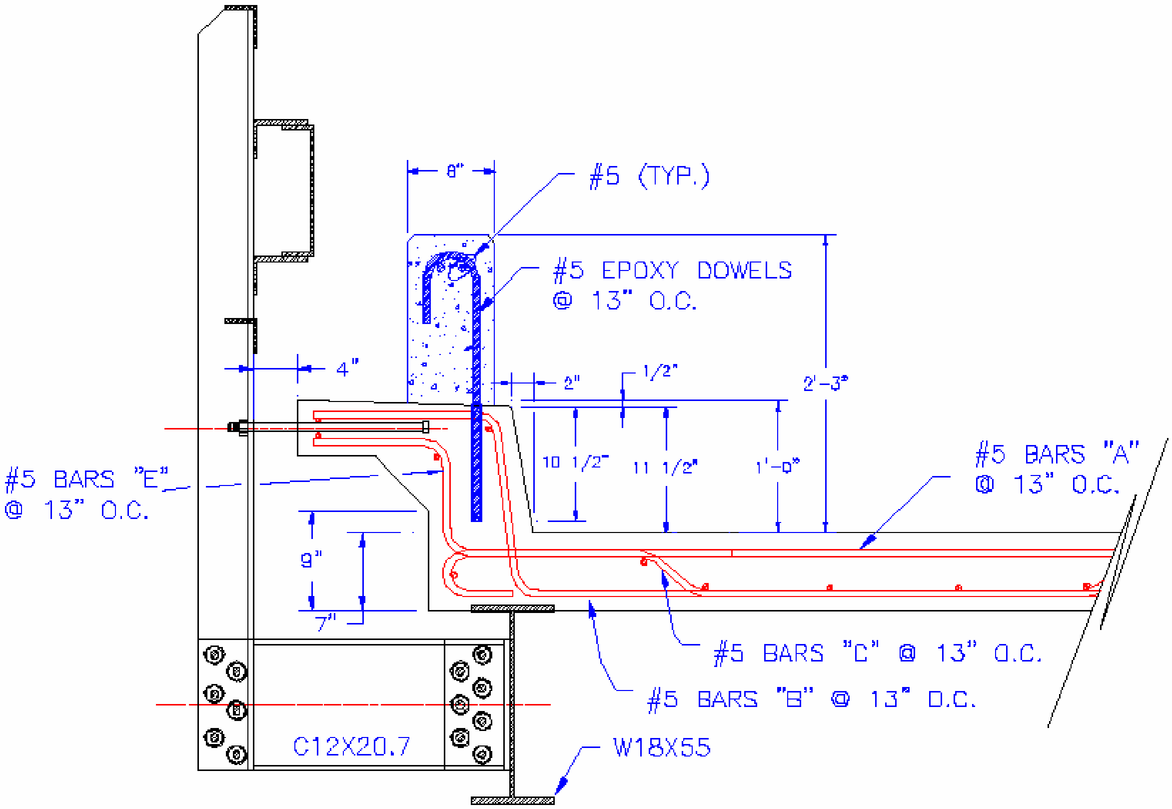
The calculations contained in this appendix pertain to the design of a retrofit bridge rail for the U.S. 281 Bridge in Palo Pinto County, Texas. This bridge structure was constructed with a 1 ft 7-1/2 inch (495 mm) wide concrete curb. This curb extended from the face of the existing traffic rail 1 ft 6 inches (457 mm). The clear roadway width between the top face of the curb is 24 ft (7.32 m). In lieu of designing a retrofit rail system to attach to the existing truss members similar to the current design, the researchers developed several options to utilize the wide curb to support a new rail system that did not attach to the existing truss members. The researchers developed several design alternatives, which were considered for this project. Five of the design alternatives were selected for more detailed evaluation and are presented in this appendix. The safety performance of each design as well as the aesthetic characteristics of each design were considered in the selection process. Option 4 with the addition of a C12×20.7 (C310×31) channel rail to the front face of the W6×20 (W150×30) was selected for final design for this project. The calculations presented in this appendix pertain to the railing design that was selected by the researchers and TxDOT personnel as the preferred alternative.

The bridge rail design selected for this project utilized a new rail system mounted on top of the existing concrete curb. An analysis was performed to determine the structural adequacy of the curb to support the new rail system. The retrofit rail attaches directly to the top of the curb using adhesive epoxy anchor bolts. Based on these analyses, TTI researchers recommend that the retrofit bridge rail be 30 inches (762 mm) in height with W6×20 (W150×30) posts. To maintain the existing structural appearance of the bridge, the existing C12×20.7 (C310×31) rail was retained. A new W6×20 (W150×30) rail was installed behind the channel to provide increased strength.

Texas
Transportation
Institute

PAGE 1 of 05
JOB NO. 444193
Date: 01-28-03

SUBJECT Retrofit Railing for U.S. 281
Truss Bridge over Brazos River
BY: W. Williams
CKD: ____________
CLIENT TxDOT

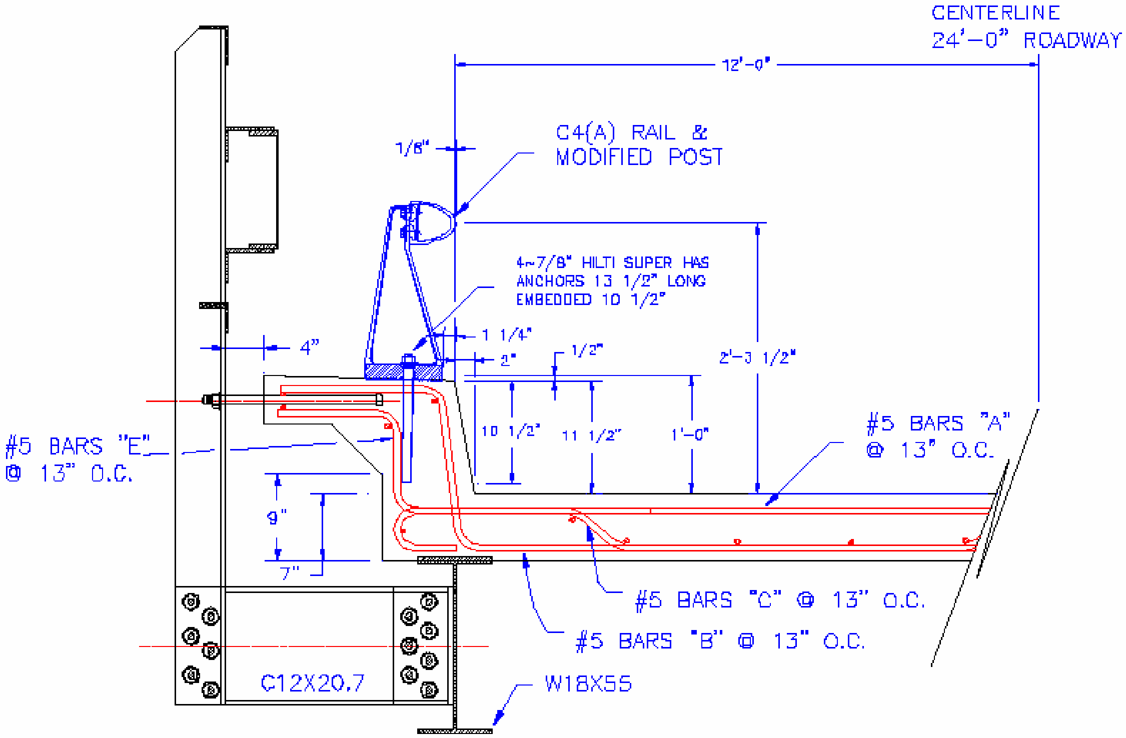


RETROFIT OPTION #1

Texas Transportation Institute

PAGE 2 of 05
JOB NO. 444193
Date: 01-28-03

SUBJECT Retrofit Railing for U.S. 281
Truss Bridge over Brazos River
BY: W. Williams
CKD:
CLIENT TxDOT



RETROFIT OPTION #2

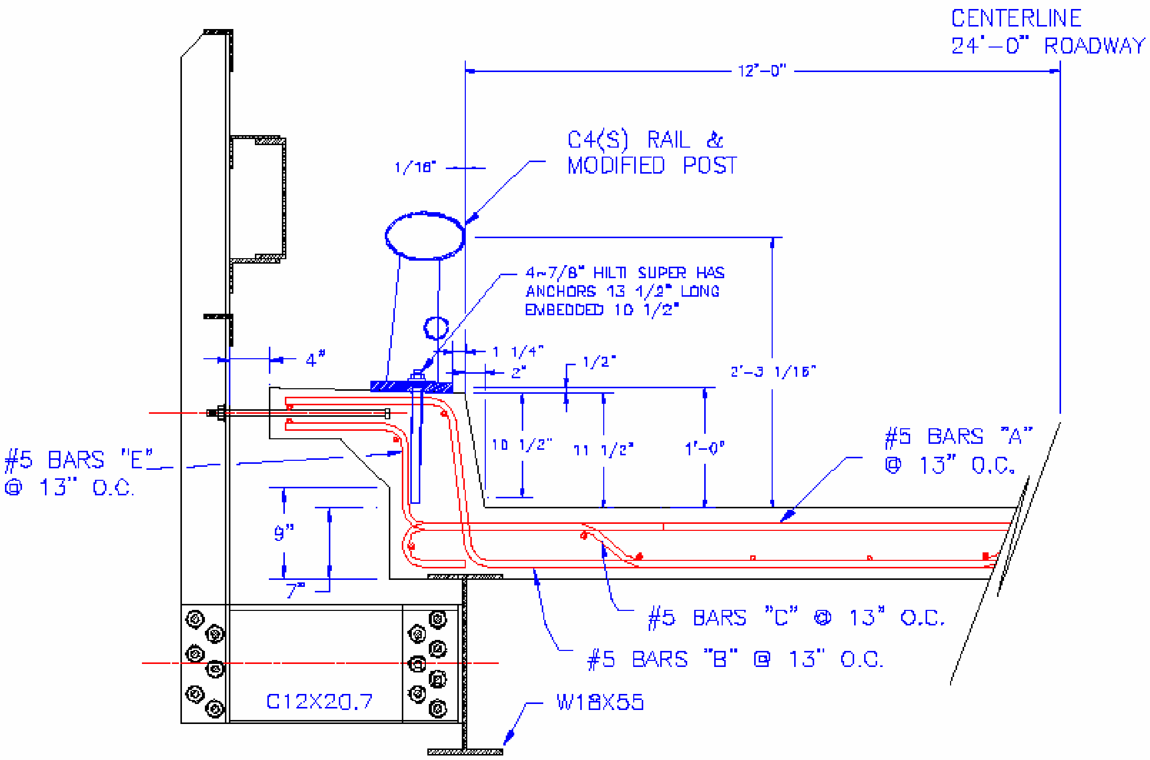
Texas Transportation Institute

PAGE 3 of 05
JOB NO. 444193
Date: 01-28-03

SUBJECT Retrofit Railing for U.S. 281
Truss Bridge over Brazos River

BY: W. Williams
CKD: ____________

CLIENT TxDOT

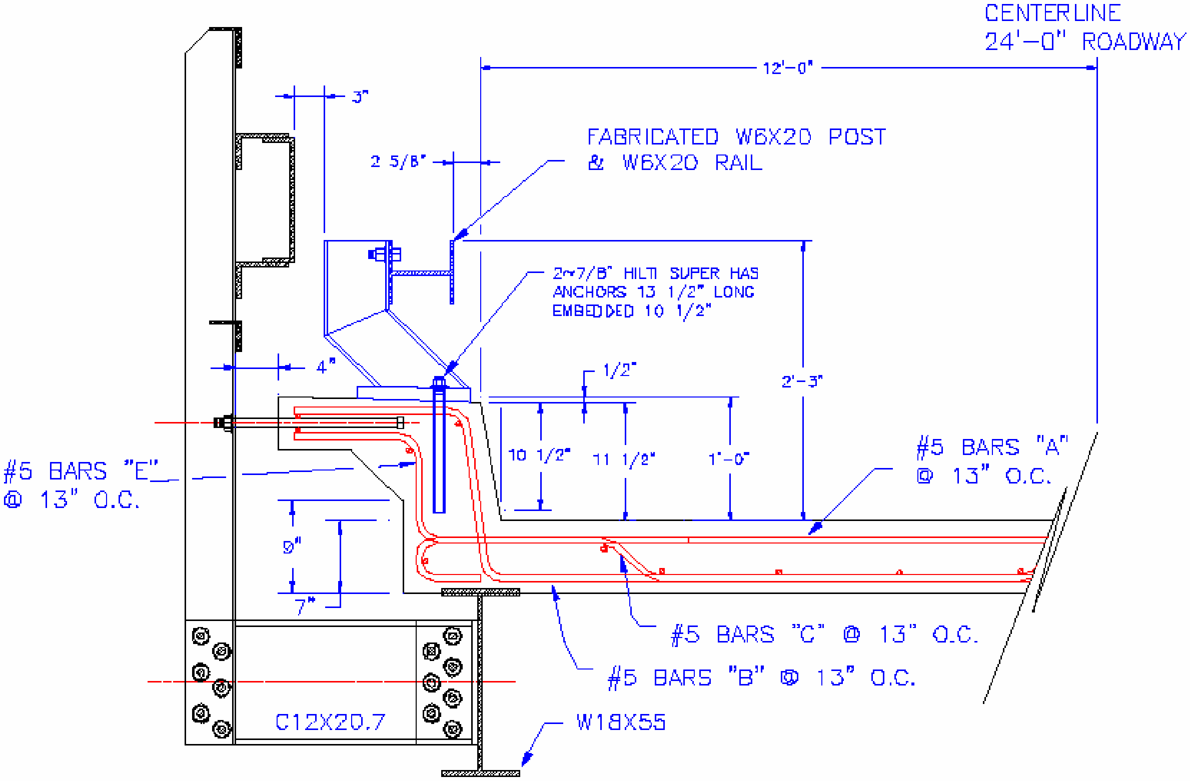


RETROFIT OPTION #3

Texas Transportation Institute

PAGE 4 of 05
JOB NO. 444193
Date: 01-28-03

SUBJECT Retrofit Railing for U.S. 281
Truss Bridge over Brazos River
BY: W. Williams
CKD: ____________
CLIENT TxDOT

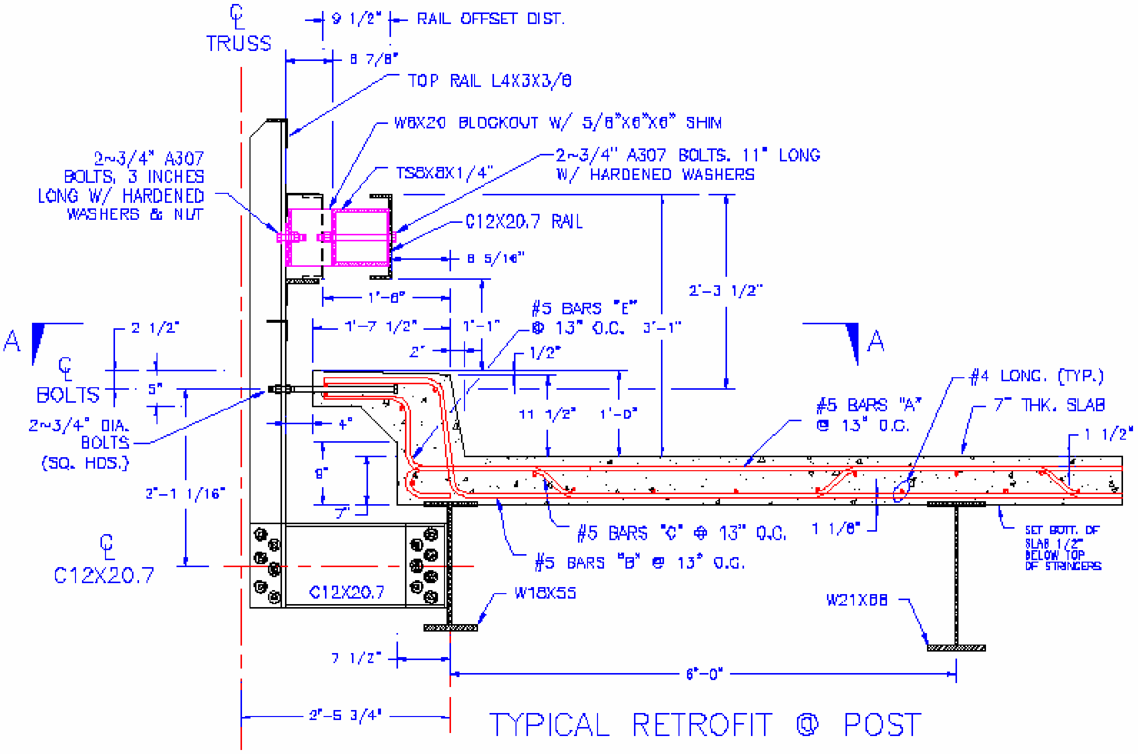


RETROFIT OPTION #4

Texas Transportation Institute

PAGE 05 of 05
JOB NO. 444193
Date: 02-19-03

SUBJECT Retrofit Railing for U.S. 281
Truss Bridge over Brazos River
BY: W. Williams
CKD: ____________
CLIENT TxDOT



TYPICAL RETROFIT @ POST

RETROFIT OPTION #5

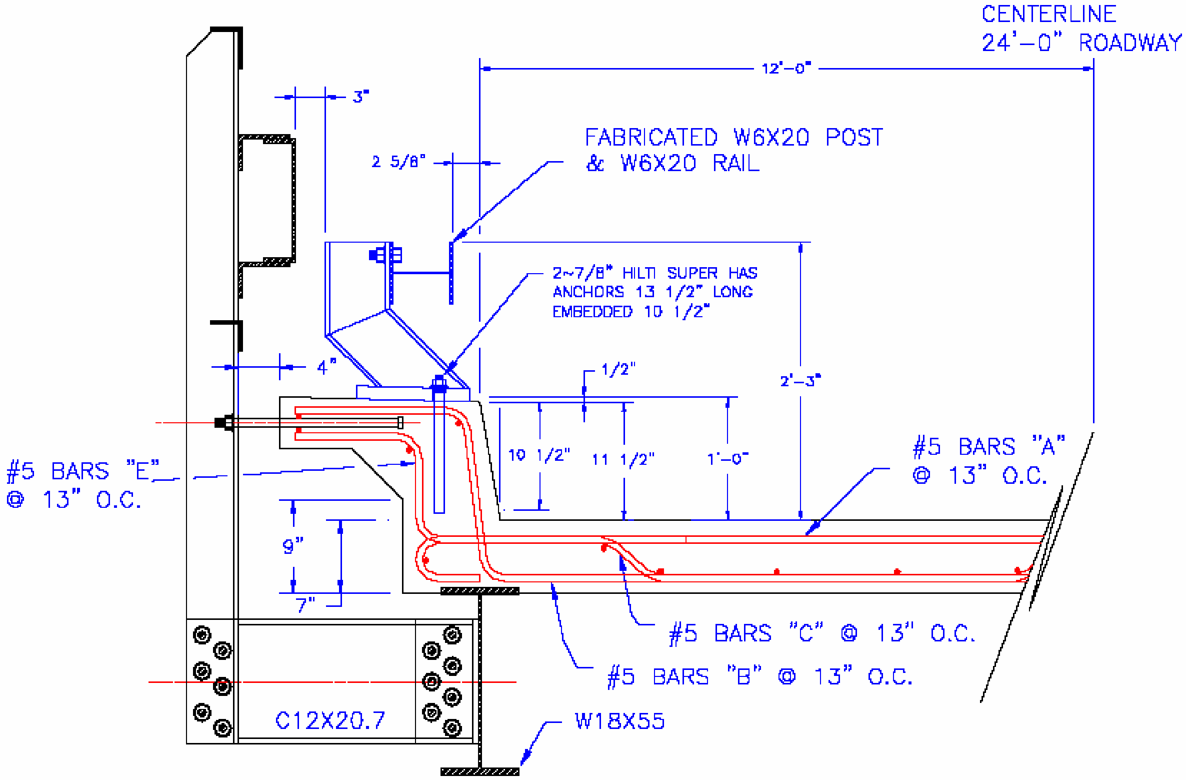
Texas Transportation Institute

PAGE 1 of 19
JOB NO. 444193
Date: 04-09-03

SUBJECT Retrofit Railing Design for U.S. 281 Truss Bridge over Brazos River
BY: W. Williams
CKD: ____________
CLIENT TxDOT

1.) Given Details of Retrofit Design:

1.) U.S. 281 Bridge over Brazos River, Palo Pinto, Co.
2.) AutoCad Details: T:\2002-2003\444192\US281Bridge1.dwg.



RETROFIT OPTION #4

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Institute

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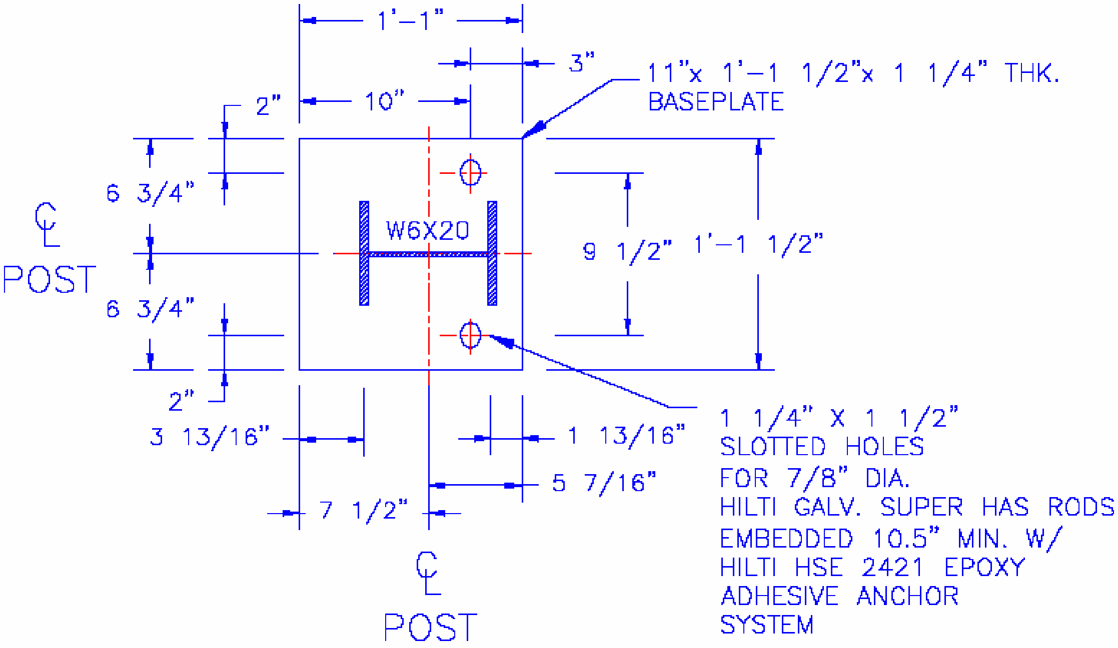
JOB NO. 444193
Date: 04-09-03

SUBJECT Retrofit Railing Design for U.S. 281
Truss Bridge over Brazos River
BY: W. Williams
CKD:
CLIENT TxDOT

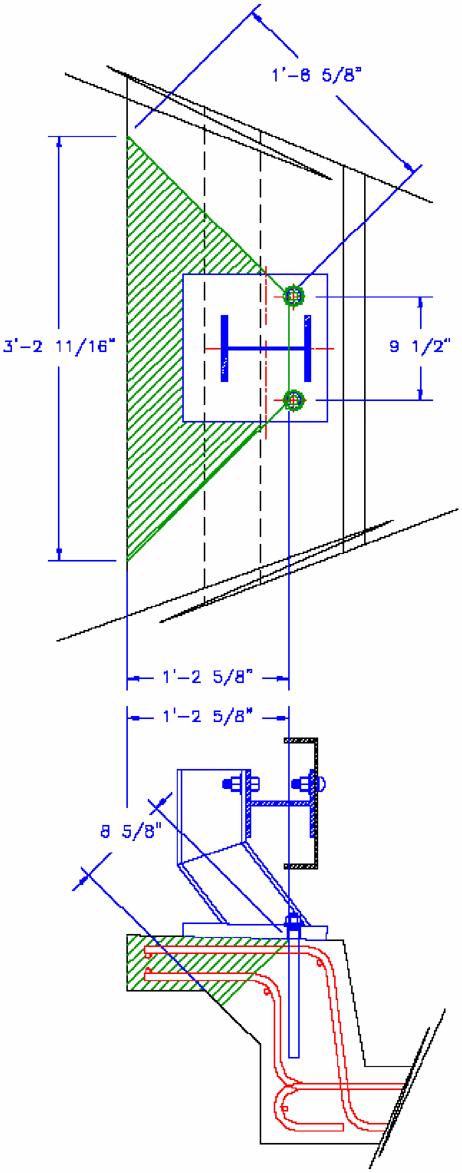


W6X20 POST BASEPLATE DETAILS

Texas
Transportation
Institute

PAGE 3 of 19
JOB NO. 444193
Date: 04-09-03

SUBJECT Retrofit Railing Design for U.S. 281
Truss Bridge over Brazos River
BY: W. Williams
CKD: ____________
CLIENT TxDOT



Texas Transportation Institute

PAGE 4 of 19
JOB NO. 444193
Date: 04-09-03

SUBJECT Retrofit Railing Design for U.S. 281
Truss Bridge over Brazos River
BY: W. Williams
CKD:
CLIENT TxDOT

2.) Given Design Information:

************************* **Bridge Rail Information** ******************************

$f_{ysteel} := 36ksi$ Yield Strength of New Structural Steel

$Ht := 30in$ Top of Rail Height from Pavement Surface

$L := 6ft$ ..... L = Post Spacing (ft) Used for Design

$Z_{yrail} := 14.9in^3$ ..... Plastic Modulus of W6x20 Rail behind C12x20.7 Rail
(USE ONLY W6 STRENGTH)

$Z_{ypost} := 14.9in^3$ Plastic Modulus of Post

$Bplate_{thk} := 1.25in$ Thickness of Baseplate (in)

$H_w := 11.5in$ Height of Concrete Curb (in)

$Rail_{ht} := Ht - H_w$ $\quad Rail_{ht} = 18.5\,in$

$Post_{ht} := Rail_{ht} - 6in - Bplate_{thk}$ $\quad Post_{ht} = 11.25\,in$ Height of Post Used for Post Bending (ft)

************************* **Concrete & Reinforcing Steel Information** ***************************

$f'_c := 3000psi$ Compressive Strength of Concrete

$f_y := 40ksi$ Yield Strength of 1938 Rebar

$E_c := 3400000psi$ Modulus of Elasticity of Concrete, (psi)

$\phi := 1.0$ Strength Reduction Factor (no reduction)

***************************** **AASHTO LRFD Information** ************************************

$F_t := 54kips$ Transverse Force Specified in LRFD Table A13.2-1, TL-3 Conditions.

$L_t := 4.0ft$ Longitudinal Length Distribution of Imapct Force (ft) for TL-3 Conditions.

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******************** Anchor Bolt Design Information ***************************

Use 7/8" Dia. Hilti Super HAS Galvanized Anchors, 13-1/2" long, embedded 10-1/2"

$F_{uA193} := 125\text{ksi}$ Tensile strength of A193 Rod/Bolt Material (ksi), see page 53, 2001 Edition, Technical Guide.

$F_{yA193} := 105\text{ksi}$

$Dia_{bolt} := \frac{7}{8}\text{in}$ ... Diameter of Anchor Bolt (in) $Area_{bolt} := \pi \times Dia_{bolt}^{\,2} \times 0.25$

3.) Calculate the Plastic Strength of the Existing Rail (M_p): $Area_{bolt} = 0.6\,\text{in}^2$

$M_{prail} := Z_{yrail} \times f_{ysteel}$ $Z_{yrail} = 14.9\,\text{in}^3$ $f_{ysteel} = 36\,\text{ksi}$ $M_{prail} = 44.7\,\text{kip} \times \text{ft}$

4.) Calculate the Plastic Strength of the Posts (P_{Post1}):

$M_{post} := Z_{ypost} \times f_y$ $M_{post} = 49.67\,\text{kip} \times \text{ft}$ $Post_{ht} = 11.25\,\text{in}$ $Rail_{ht} = 18.5\,\text{in}$

$P_{Post1} := \frac{M_{post}}{Rail_{ht}}$ $P_{Post1} = 32.22\,\text{kip}$ Load @ Top of Rail to Fail Post

5.) Calculate Spring Information for RISA-3D Finite Element Model:

$L_{spring} := 7.5\text{in}$ Assumed effective depth/length of spring (concrete thickness beneath field flange

$E_c = 3.4 \times 10^6\,\text{psi}$ Modulus of Concrete (psi)

.. Nodes on Edges (Row1) of BP Use 0.5" x 0.844" (weigthed average)

$Area_{r1} := 0.5\text{in} \times 0.844\text{in}$ $K_1 := Area_{r1} \times E_c \times L_{spring}^{\,-1}$ $K_1 = 191.31\,\frac{\text{kips}}{\text{in}}$

$Area_{r2} := 1\text{in} \times 0.844\text{in}$ $K_2 := Area_{r2} \times E_c \times L_{spring}^{\,-1}$ $K_2 = 382.61\,\frac{\text{kips}}{\text{in}}$

$Area_{r3} := Area_{r2}$ $K_3 := Area_{r3} \times E_c \times L_{spring}^{\,-1}$ $K_3 = 382.61\,\frac{\text{kips}}{\text{in}}$

$\mathrm{Area}_{r4} := 0.8943\mathrm{in} \times 0.844\mathrm{in}$ $\quad K_4 := \mathrm{Area}_{r4} \times E_c \times L_{spring}^{-1}$ $\quad K_4 = 342.17\,\frac{\mathrm{kips}}{\mathrm{in}}$

$\mathrm{Area}_{r5} := 1.143\mathrm{in} \times 0.844\mathrm{in}$ $\quad K_5 := \mathrm{Area}_{r5} \times E_c \times L_{spring}^{-1}$ $\quad K_5 = 437.33\,\frac{\mathrm{kips}}{\mathrm{in}}$

$\mathrm{Area}_{r6} := 1.4875\mathrm{in} \times 0.844\mathrm{in}$ $\quad K_6 := \mathrm{Area}_{r6} \times E_c \times L_{spring}^{-1}$ $\quad K_6 = 569.14\,\frac{\mathrm{kips}}{\mathrm{in}}$

$\mathrm{Area}_{r7} := 1.475\mathrm{in} \times 0.844\mathrm{in}$ $\quad K_7 := \mathrm{Area}_{r7} \times E_c \times L_{spring}^{-1}$ $\quad K_7 = 564.35\,\frac{\mathrm{kips}}{\mathrm{in}}$

$\mathrm{Area}_{r8} := 1.6188\mathrm{in} \times 0.844\mathrm{in}$ $\quad K_8 := \mathrm{Area}_{r8} \times E_c \times L_{spring}^{-1}$ $\quad K_8 = 619.37\,\frac{\mathrm{kips}}{\mathrm{in}}$

$\mathrm{Area}_{r9} := 1.5143\mathrm{in} \times 0.844\mathrm{in}$ $\quad K_9 := \mathrm{Area}_{r9} \times E_c \times L_{spring}^{-1}$ $\quad K_9 = 579.39\,\frac{\mathrm{kips}}{\mathrm{in}}$

$\mathrm{Area}_{r10} := 1.5\mathrm{in} \times 0.844\mathrm{in}$ $\quad K_{10} := \mathrm{Area}_{r10} \times E_c \times L_{spring}^{-1}$ $\quad K_{10} = 573.92\,\frac{\mathrm{kips}}{\mathrm{in}}$

$\mathrm{Area}_{r11} := 0.867\mathrm{in} \times 0.844\mathrm{in}$ $\quad K_{11} := \mathrm{Area}_{r11} \times E_c \times L_{spring}^{-1}$ $\quad K_{11} = 331.73\,\frac{\mathrm{kips}}{\mathrm{in}}$

6.) Calculate the Strength of the Post to Fail Hilti HSE Epoxy Anchors in Tension Considering Hilti Spacing & Cover Reductions (P_{Post2}):

Use Hilti Hight Strength Super HAS, 7/8" Dia. Rod, Embedment Depth 10.5 inches min.

1.) As per page 93, 2001 Hilti Product Technical Guide,

$HSE_{ultimatetensile} := 62825 lbf$ Ultimate Bond Strength (see pg 93 Hilti 2001 Tech. Guide)

$HSE_{ultimateshear} := 34955 lbf$ Ulitmate shear strength of anchor (see pg 93)

Check Load Adjustment Factors for Anchor Spacings & Clearance:

From page 60, Hilti 2001 Technical Guide

$s_{actual} := 9.5 in$ Actual anchor spacing (inches)

$c_{actual} := 8.0 in$ Edge distance (inches)

$h_{nom} := 7.625 in$ Standard embedment depth (inches)

$h_{ef} := 10.5 in$ Actual embedment depth (inches)

$$f_A := 0.3 \times \left(\frac{s_{actual}}{h_{ef}}\right) + 0.55 \qquad f_A = 0.82$$

Load Adjustment Factor for Spacing

$$f_{RN} := 0.4 \times \frac{c_{actual}}{h_{ef}} + 0.40 \qquad f_{RN} = 0.7$$

Load Adjustment Factor for Edge Distance in Tension

$$f_{RV} := 0.75 \times \left(\frac{c_{actual}}{h_{ef}}\right) - 0.125 \qquad f_{RV} = 0.45$$

Load Adjustment Factor for Edge Distance in Shear

Calculate Factored Tension Strength Due to Spacing & Clearance:

$$\mathbf{HiltiUltimate_{Tension}} := \mathbf{HSE_{ultimatetensile}} \times \mathbf{f_A} \times \mathbf{f_{RN}}$$

$\mathbf{HiltiUltimate_{Tension}} = 36.37\,\mathbf{kips}$ Factored Tension < Ultimate Applied

$$\mathbf{HiltiUltimate_{Shear}} := \mathbf{HSE_{ultimateshear}} \times \mathbf{f_A} \times \mathbf{f_{RV}}$$

$\mathbf{HiltiUltimate_{Shear}} = 12.82\,\mathbf{kips}$ Factored Shear > Ultimate Applied .. O.K.

Run RISA-3D Model to Determine Actual Force in Bolts for Given Post Load:

Try 34 kips in Post $\mathbf{Post_{load2}} := 34\mathbf{kips}$

$\mathbf{N_{d2}} := 36.8\mathbf{kips}$ Actual bolt tension for loading @ post

$\mathbf{V_{d2}} := 17.5\mathbf{kips}$ Actual bolt shear for loading @ post

Check Combined Loading:

$$\mathbf{Hilti_{SR}} := \left(\frac{\mathbf{N_{d2}}}{\mathbf{HiltiUltimate_{Tension}}}\right)^{\frac{5}{3}} + \left(\frac{\mathbf{V_{d2}}}{\mathbf{HiltiUltimate_{Shear}}}\right)^{\frac{5}{3}}$$

$\mathbf{Hilti_{SR}} = 2.7$ Check Post Strength Based on Limited Hilti Bolt Strength

Calculate Post Strength (P_{Post2}) Based on Limited Hilti Bolt Strength:

6a.) Determine Bolt "Moment Arm" From RISA Analysis

$$Risa_{bolttension} := N_{d2} \times 2$$

$$Post_{load2} = 34\,kips$$

$$Rail_{ht} = 18.5\,in \qquad Post_{ht} = 11.25\,in$$

$$Load_{ht} := Rail_{ht} - 6in \qquad Load_{ht} = 12.5\,in$$

$$BP_{arm2} := \frac{Post_{load2} \times Load_{ht}}{Risa_{bolttension}}$$

$$BP_{arm2} = 5.77\,in$$

$$P_{Post2} := \frac{BP_{arm2} \times HiltiUltimate_{Tension} \times 2}{Rail_{ht}}$$

$P_{Post2} = 22.7\,kips$ Ultimate Strength of Post using 2~A193 HSE 2421 Anchors

7.) Calculate the Strength of the Post to Fail Hiliti Bolts in Combined Shear & Tension (P_{Post3}):

$$Post_{load3} := 40kips$$

From RISA-3D Modeling Analysis:

$N_{d3} := 46.24kips$ Actual bolt tension for loading @ post

$V_{d3} := 20.0kips$ Actual bolt shear for loading @ post

$R_{nt} := F_{uA193} \times \left(0.75 \times Area_{bolt}\right) \quad R_{nt} = 56.37\,kips$ Nominal Strength in Tension

$R_{nv} := Area_{bolt} \times \left(0.45 \times F_{uA193}\right) \quad R_{nv} = 33.82\,kips$ Nominal Strength in Shear w/ Threads in Shear Plane

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Calculate Design Strengths of Bolt in Tension & Shear

$\phi_t := 1.00$ $\phi_v := 1.00$

(0.75) (0.65)

$R_{ut} := N_{d3}$ R_{ut} is equal to the actual tension force on each bolt

$R_{uv} := V_{d3}$ R_{uv} is equal to actual shear on each bolt

Calculate LRFD Stress Ratio of Bolts in Tension & Shear:

$$Bolt_{S.R.} := \left(\frac{R_{ut}}{\phi_t \times R_{nt}}\right)^2 + \left(\frac{R_{uv}}{\phi_v \times R_{nv}}\right)^2$$

Equation 4.14.1, pg 173 Salmon & Johnson, 3rd Edition

$Bolt_{S.R.} = 1.022$ .... Stress Ratio .... O.K.

Therefore...... calculate equalivalent load @ 30-inch height:

$Risa_{bolttension2} := N_{d3} \times 2$

$Post_{load3} = 40\,kips$

$Rail_{ht} = 18.5\,in$ $Post_{ht} = 11.25\,in$

$Load_{ht2} := Rail_{ht} - 6in$ $Load_{ht} = 12.5\,in$

$$BP_{arm3} := \frac{Post_{load3} \times Load_{ht2}}{Risa_{bolttension2}}$$

$BP_{arm3} = 5.41\,in$

$$P_{Post3} := \frac{BP_{arm3} \times HiltiUltimate_{Tension} \times 2}{Rail_{ht}}$$

$P_{Post3} = 21.26\,kips$ Ultimate Strength of Post using 2~A193 HSE 2421 Anchors.

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8.) Calculate the Strength of the Post to Fail Concrete in Baseplate Bearing (P_{Post4}):

$$Post_{load4} := 40kips$$

$$Force_{N77} := 2.513kips$$

$$Area_{N77} := \left(\frac{0.7885in + 1.499in}{2}\right) \times 0.75in \qquad Area_{N77} = 0.86\,in^2$$

$$\sigma_{conbearing} := \frac{Force_{N77}}{Area_{N77}} \qquad \sigma_{conbearing} = 2929.54\,psi \qquad Bplate_{thk} = 1.25\,in$$

Determine Equivalent Load @ 30 inches:

$$P_{Post4} := \frac{Post_{load4} \times Load_{ht}}{Rail_{ht}}$$

$$P_{Post4} = 27.03\,kips$$

Load @ top of rail (30 inches) to cause excessive concrete bearing stresses at or above 3000 psi

9.) Calculate the Strength of the Post to Fail Concrete in Punching Failure (P_{Post5}) (see failure diagram above):

$Area_1 := 9.5in \times 8.5in$ failure area between bolts @ 45 deg angle downward from top of concrete

$Area_2 := [(1ft + 8.625in) \times 5in] \times 2$ .... there are two failure surfaces here

$$Area_{tension} := \left(Area_1 + Area_2\right) \qquad Area_{tension} = 287\,in^2$$

$$V_{cr} := 4 \times \sqrt{f'_c \times psi} \times 0.85 \times Area_{tension}$$

$$V_{cr} = 53.45\,kips$$

$$P_{Post5} := \frac{H_w \times \left(V_{cr}\right)}{Ht}$$

$$P_{Post5} = 20.49\,kips$$

Post strength based on concrete tensile failure

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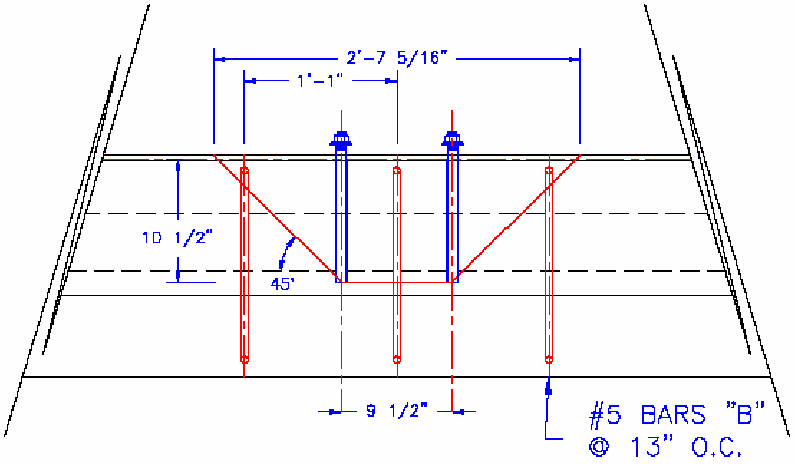
10.) Calculate the Strength of the Post to Fail Post due to Bending Failure in Curb (P_{Post6}):



TRAFFIC SIDE VIEW

Approach: Add moment capacity of vertical reinforcement (M_1) & longitudinal reinforcement in top of curb (M_2) and compare this capacity to P_{Post6} X ht (30 inches).

Calculate moment capacity (M1) of vertical curb reinforcement #5 "B" bars @ 13" O.C.:

$f_y = 40\,\text{ksi}$	Yield strength of rebar
$f'_c := 3000\text{psi}$	Compressive strength of concrete
$A_{s1} := 0.31\text{in}^2 \times 2$	Use 2 vertical #5 bars "B" @ 13" O.C. over failure area
$b_1 := 30\text{in}$	Use 2 ft 6 inches for length resisting shear force

$$a_1 := \frac{A_{s1} \times f_y}{0.85 \times f'_c \times b_1} \qquad a_1 = 0.32\,\text{in}$$

$d_1 := 7.375\text{in}$ This distance measured in AutoCad

$d_1 = 7.38\,\text{in}$

Calculate M_1:

with:

$d_1 = 7.38\,\text{in}$ $\phi = 1$

$A_{s1} = 0.62\,\text{in}^2$

$f_y = 40\,\text{ksi}$

$a_1 = 0.32\,\text{in}$

Therefore:

$$M_1 := (\phi \times A_{s1} \times f_y) \times \left(d_1 - \frac{a_1}{2}\right)$$

$M_1 = 14.91\,\text{kip} \times \text{ft}$ Flexural resistance of 12 inch curb using only 2~#5 "B" bars

Now calculate M_2 .... contribution from top of curb (beam Action):

$A_{s2} := 0.2\text{in}^2 \times 3$ ... 3 Longitudinal #4 tension bars in beam

$f_y = 40\,\text{ksi}$

$b_2 := 8\text{in}$ ..Use 8 inch width of compression side of curb

$$a_2 := \frac{A_{s2} \times f_y}{0.85 \times f'_c \times b_2}$$

$a_2 = 1.18\,\text{in}$

$$d_2 := \frac{(2 \times 18.5625\text{in}) + (1 \times 7.875\text{in})}{3} \qquad d_2 = 15\,\text{in}$$

..distance to 3 tension bars measured in AutoCad

Calculate M_2:

with:

$d_2 = 15\,in$ $H_W = 11.5\,in$

$A_{s2} = 0.6\,in^2$ $\phi = 1$

$f_y = 40\,ksi$

$a_2 = 1.18\,in$

Therefore:

$$M_2 := \left[\phi \times A_{s2} \times f_y \times \left(d_2 - \frac{a_2}{2}\right)\right]$$

$M_2 = 28.82\,kip \times ft$ Flexural resistance of beam in top of curb in (kip-ft)

Calculate P_{Post6}:

$$P_{Post6} := \frac{M_1 + M_2}{Ht}$$

$P_{Post6} = 17.49\,kip$ Force at top of rail to cause flexural failure in curb

11.) Summarize Post Strengths and Select Worst Case:

$P_{Post1} = 32.22\,kips$ Strength Based on Plastic Failure of Post

$P_{Post2} = 22.7\,kips$ Strength Based on Failure of Bolts in Tension

$P_{Post3} = 21.26\,kips$ Strength Based on Failure of Bolts in Combined Tension & Shear

$P_{Post4} = 27.03\,kips$ Strength Based on Excessive Concrete Bearing Stresses at or above 3000 psi

$P_{Post5} = 20.49\,kips$ Strength Based on Lateral Punching Shear Failure on Concrete behind Anchor Bolts

$P_{Post6} = 17.49\,kips$ Strength Based on Bending Failure in Curb & Beam

Therefore ..... the worst case is ......

$P_{Post} := P_{Post6}$ Failure Mode of Post

$M_p := M_{prail}$

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12.) Determine Total Rail Resistance of Existing Rail for One Span:

$P_{Post} = 17.49\,\text{kips}$ Post Spacing

$M_p = 44.7\,\text{kips} \times \text{ft}$ $L_t := 4.0\text{ft}$

$N := 1$ ..... Single Span Check

$$R_{1span} := \frac{16 \times M_p + (N-1) \times (N+1) \times P_{Post} \times L}{2 \times N \times L - L_t}$$

$R_{1span} = 89.4\,\text{kips}$

13.) Determine Total Rail Resistance of Existing Rail for Two Span:

$N := 2$ ..... Double Span w/ Load applied @ Post

$M_p = 44.7\,\text{kips} \times \text{ft}$ $P_{Post} = 17.49\,\text{kips}$ $L = 6\ \text{ft}$

$$R_{2span} := \frac{16 \times M_p + N^2 \times P_{Post} \times L}{2 \times N \times L - L_t}$$

$R_{2span} = 56.75\,\text{kips}$

14.) Determine Total Rail Resistance of Existing Rail for Three Span:

$P_{Post} = 17.49\,\text{kips}$ $M_p = 44.7\,\text{kips} \times \text{ft}$ $N := 3$ ..... Three Span Check

$$R_{3span} := \frac{16 \times M_p + (N-1) \times (N+1) \times P_{Post} \times L}{2 \times N \times L - L_t}$$

$R_{3span} = 48.59\,\text{kips}$

15.) Determine Total Rail Resistance of Existing Rail for Four Span:

$N := 4$ ..... 4 Spans: $M_p = 44.7\,\text{kips} \times \text{ft}$ $P_{Post} = 17.49\,\text{kips}$ $L = 6\ \text{ft}$

$$R_{4span} := \frac{16 \times M_p + N^2 \times P_{Post} \times L}{2 \times N \times L - L_t}$$

$$R_{4span} = 54.42\,\text{kips}$$

$$R_{Bar} := R_{3span} \qquad Y_{Bar} := 30\text{in}$$

******* Resultant @ 24-inch Height *********

$$Y_{27} := 27\text{in}$$

$$R_{bar27} := \frac{R_{Bar} \times Y_{Bar}}{Y_{27}}$$

$R_{bar27} = 53.99\,\text{kips}$ @ 27 inches

***** Resultant @ 32-inch Height ***********

$$Y_{32} := 32\text{in}$$

$$R_{bar32} := \frac{R_{Bar} \times Y_{Bar}}{Y_{32}}$$

$R_{bar32} = 45.55\,\text{kips}$ @ 32 inches

In summary: The new retrofit rail meets the requirements for TL-3 loading conditions (F_t = 54 kips @ 27 inches).

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16.) Check Strength of Post in Reverse Loading Conditions:

From RISA-3D Modeling Analysis:

$\mathrm{Post}_{\mathrm{load7}} := 10\mathrm{kips}$ (34 kips in the impact direction)

(15 kips for HSE bolt strength)

Check max bearing on edge of baseplate

$$\mathrm{Force}_{\mathrm{N184}} := 2.048\mathrm{kips}$$

$$\mathrm{Area}_{\mathrm{N184}} := \frac{1.7339\mathrm{in}}{2} \times 0.875\mathrm{in}$$

$$\mathrm{Area}_{\mathrm{N184}} = 0.76\,\mathrm{in}^2$$

$$\sigma_{\mathrm{conbearingREV}} := \frac{\mathrm{Force}_{\mathrm{N184}}}{\mathrm{Area}_{\mathrm{N184}}}$$

$\sigma_{\mathrm{conbearingREV}} = 2699.78\,\mathrm{psi}$ .... probably upper limit on bearing

$\mathrm{Load}_{\mathrm{ht}} = 12.5\,\mathrm{in}$ Height from bottom of BP to center of load on post

$\mathrm{Rail}_{\mathrm{ht}} = 18.5\,\mathrm{in}$ Height from bottom of BP to top of rail

$$\mathrm{P}_{\mathrm{Post7}} := \frac{\mathrm{Post}_{\mathrm{load7}} \times \mathrm{Load}_{\mathrm{ht}}}{\mathrm{Rail}_{\mathrm{ht}}}$$

$\mathrm{P}_{\mathrm{Post7}} = 6.76\,\mathrm{kips}$ Reverse load @ top of rail to produce hign baseplate bearing

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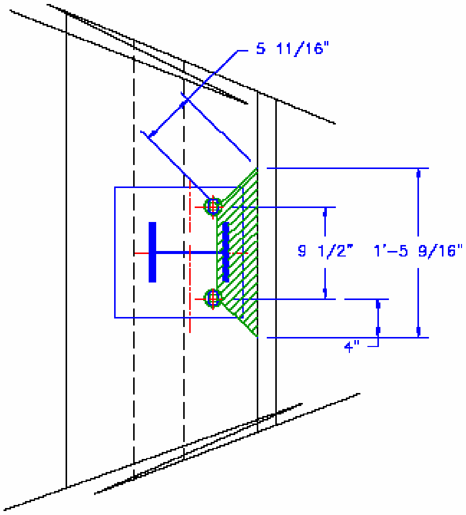
17.) Check Reverse Post Lateral Shear Punch-out Capacity:



$\text{Area}_1 := 9.5\text{in} \times 6.75\text{in}$ failure area between bolts @ 45 degree angle downward from top of concrete

$\text{Area}_2 := (5.69\text{in} \times 6.75\text{in}) \times 2$

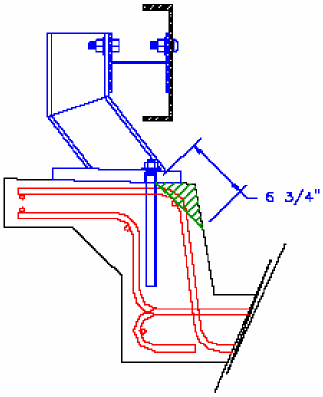
.... there are two failure surfaces here

$\text{Area}_{\text{tension}} := (\text{Area}_1 + \text{Area}_2)$

$\text{Area}_{\text{tension}} = 140.94\,\text{in}^2$

$V_{cr} := 4 \times \sqrt{f'_c \times psi} \times 0.85 \times \text{Area}_{\text{tension}}$

$V_{cr} = 26.25\,\text{kips}$



$$P_{Post8} := \frac{H_W \times (V_{cr})}{Ht}$$

$P_{Post8} = 10.06\,\text{kips}$

Reverse strength @ 30-inch height

Post has about 1/3 strength in the reverse direction

APPENDIX H. ANALYSES OF BRIDGE RAIL DESIGN FOR NEW TRUSS BRIDGES

Results of the analyses performed to develop a bridge rail design for new truss bridges are presented in this appendix.

A cross section and other details of the proposed railing are shown on pages 1 of 16 and 2 of 16 of the following calculations. The railing design provides two tubular steel rail elements mounted on crushable blockouts made from 10-inch (254 mm) diameter schedule 80 pipe. Total height of the railing above the top of the deck is 2 ft 6 inches (0.8 m), and the traffic face presents suitable geometry.

Finite element modeling was performed on several sizes of crushable pipe blockouts using the computer modeling program LS-DYNA. The blocks were loaded with diametrically opposing plates. The crushable pipe blockouts ranged in size from 6-inch (152 mm) diameter schedule 40 pipe to 10-inch (254 mm) diameter schedule 80 pipe. Five of the seven blockouts were 6 inches (152 mm) in length and the remaining two were 8 inches (203 mm) in length. Tabulations of force versus crush data for the various blockouts are presented on page 3 of 16 through 6 of 16 and are plotted on page 7 of 16.

A 10-inch (254 mm) diameter schedule 80 blockout was selected for the new railing design, and the plot for that blockout is shown on page 8 of 16. For further calculations of the behavior of the proposed railing, the crush characteristics of the blockout were idealized as two straight lines having slopes of 12 kips/inch (2.1 kN/mm) and 0.8 kips/inch (0.14 kN/mm).

Structural analyses of several different railing designs using the results obtained from the crushable pipe blockouts were performed using STAAD.Pro. Considering TL-3 conditions and using a two-rail bridge rail system, the analyses performed for each design consisted of 27 kips (120 kN) distributed over a length of 4 ft (1.2 m) at different locations along the rail. Analyses were performed on several different combinations of rail and crushable pipe blockout types using five continuous spans with span lengths ranging from 10 to 20 ft (3.0 to 6.1 m). Analyses were performed on each rail/crushable pipe combination with the 27 kips (120 kN) distributed over 4 ft (1.2 m) located at the following:

- mid-span,
- centered over a crushable pipe support located within the length of the rail (vertical truss member support), and
- over a crushable pipe support located at the end of the rail element.

Results of these analyses are presented on pages 9 of 16 through 16 of 16 of the following calculations.

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Job #: **444193**

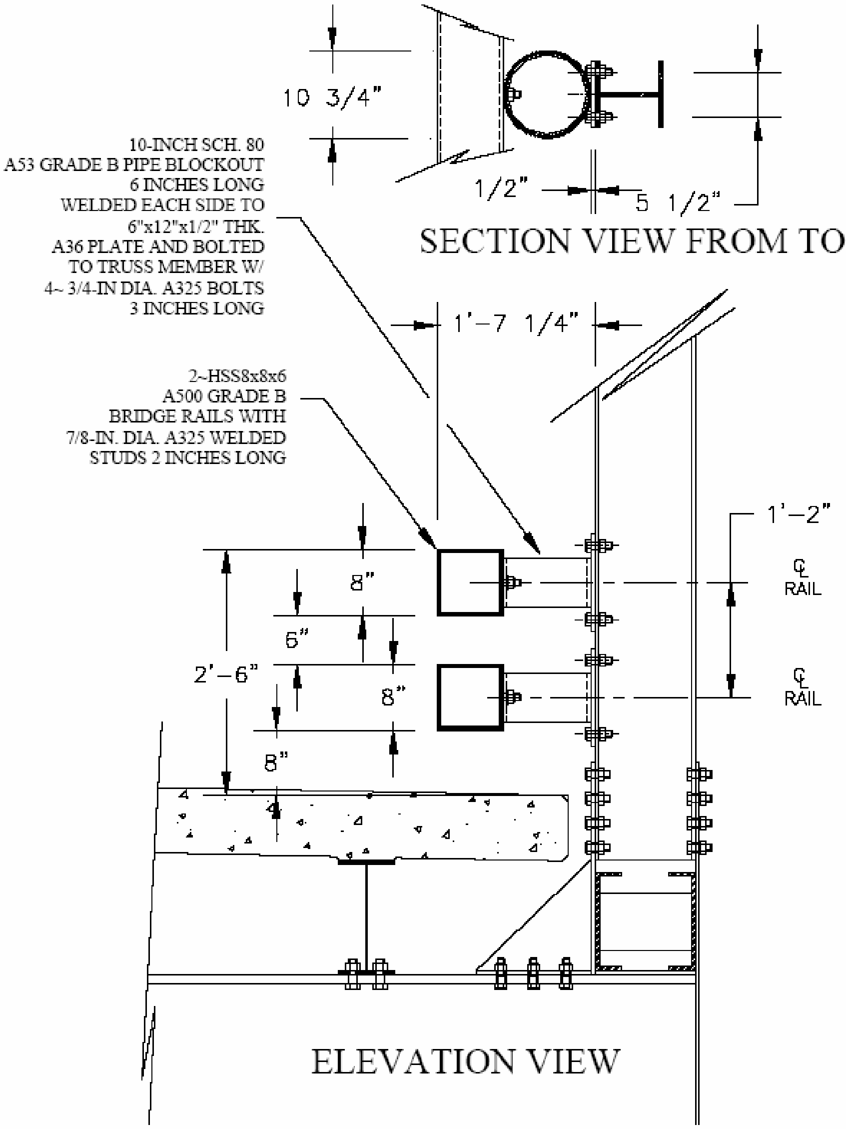
By: **William Williams**

Checked: ______

Subject: **New Bridge Rail Design**

2-Rail Tubular Design with Crushable Pipe Blockouts

Client: **Texas Department of Transportation**



10-INCH SCH. 80
A53 GRADE B PIPE BLOCKOUT
6 INCHES LONG
WELDED EACH SIDE TO
6"x12"x1/2" THK.
A36 PLATE AND BOLTED
TO TRUSS MEMBER W/
4~ 3/4-IN DIA. A325 BOLTS
3 INCHES LONG

SECTION VIEW FROM TOP

2~HSS8x8x6
A500 GRADE B
BRIDGE RAILS WITH
7/8-IN. DIA. A325 WELDED
STUDS 2 INCHES LONG

ELEVATION VIEW

New 2-Rail Design Concept

Texas Transportation Institute

Page: 2 of 16

Job #: 444193

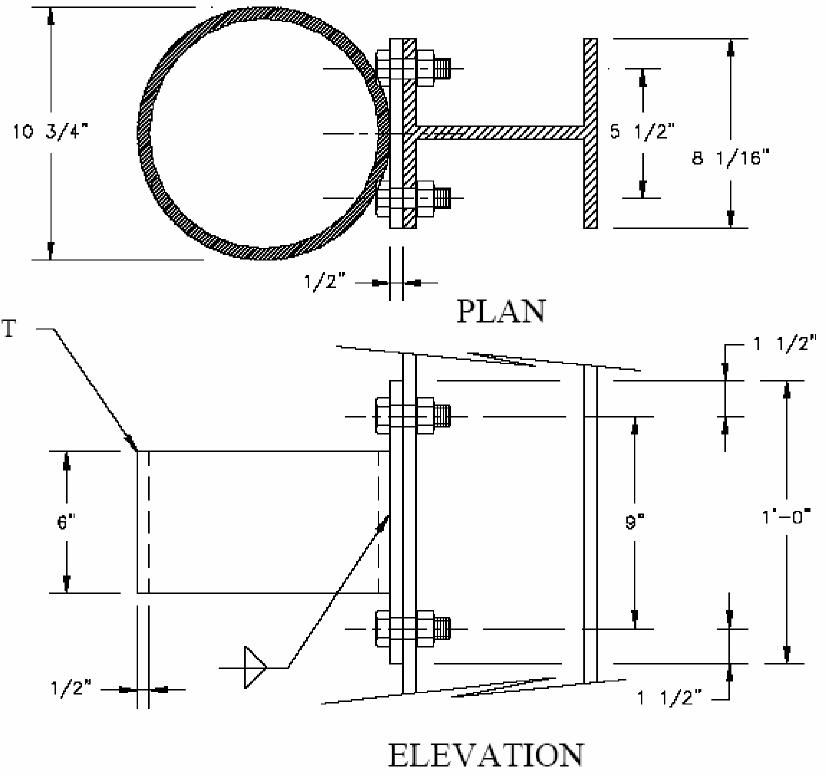
By: **William Williams**

Checked: ______

Subject: **New Bridge Rail Design**

2-Rail Tubular Design with Crushable Pipe Blockouts

Client: **Texas Department of Transportation**



10-INCH SCHEDULE 80
A53 GRADE B PIPE BLOCKOUT
6 INCHES LONG
WELDED EACH SIDE TO
6"x12"x1/2" THICK
A36 PLATE AND BOLTED
TO TRUSS MEMBER W/
4~3/4-INCH DIA. A325 BOLTS
3 INCHES LONG

10-inch Schedule 80 Crushable Pipe Blockout for
New 2-Rail Bridge Rail

Texas Transportation Institute

Page: **3 of 16**

Job #: **444193**

By: **William Williams**

Checked: ______

Subject: **Pipe Crush Data for Different Size Pipe Blockouts for New Bridge Rail Design**

Client: **Texas Department of Transportation**

$$\text{Data6inSch40}_6 := \begin{array}{c} \begin{array}{cc} \text{Force (lbs)} & \text{Crush (inches)} \end{array} \\ \begin{pmatrix} 0 & 0 \\ 2000 & 0.140913386 \\ 4000 & 0.392027559 \\ 6000 & 1.294885827 \\ 8000 & 3.930688976 \\ 10000 & 5.747448819 \end{pmatrix} \end{array}$$

$$\text{Data6inSch40}_8 := \begin{array}{c} \begin{array}{cc} \text{Force (lbs)} & \text{Crush (inches)} \end{array} \\ \begin{pmatrix} 0 & 0 \\ 2000 & 0.07057874 \\ 4000 & 0.236811024 \\ 6000 & 0.502334646 \\ 8000 & 1.193807087 \\ 10000 & 3.038102362 \\ 12000 & 4.819767717 \\ 14000 & 5.922043307 \end{pmatrix} \end{array}$$

$$\text{Data8inSch40}_6 := \begin{array}{c} \begin{array}{cc} \text{Force (lbs)} & \text{Crush (inches)} \end{array} \\ \begin{pmatrix} 0 & 0 \\ 2000 & 0.059102362 \\ 4000 & 0.123783465 \\ 6000 & 0.28230315 \\ 8000 & 4.142566929 \\ 10000 & 7.453043307 \\ 12000 & 8.143645669 \\ 14000 & 8.143645669 \\ 16000 & 8.143645669 \\ 18000 & 8.143645669 \\ 20000 & 8.143645669 \\ 22000 & 8.143645669 \\ 24000 & 8.143645669 \end{pmatrix} \end{array}$$

Texas Transportation Institute

Page: **4 of 16**

Job #: **444193**

By: **William Williams**

Checked: ______

Subject: **Pipe Crush Data**

for Different Size Pipe Blockouts for New Bridge Rail Design

Client: **Texas Department of Transportation**

$$\mathrm{Data6inSch80}_8 := \left(\begin{array}{cc} \text{Force (lbs)} & \text{Crush (inches)} \\ 0 & 0 \\ 2000 & 0.017822835 \\ 4000 & 0.062874016 \\ 6000 & 0.078992126 \\ 8000 & 0.121948819 \\ 10000 & 0.198744094 \\ 12000 & 0.279090551 \\ 14000 & 0.369185039 \\ 16000 & 0.45346063 \\ 18000 & 0.655952756 \\ 20000 & 1.018673228 \\ 22000 & 1.435393701 \\ 24000 & 2.127169291 \\ 26000 & 2.964208661 \\ 28000 & 3.66223622 \\ 30000 & 4.268208661 \\ 32000 & 4.764094488 \\ 34000 & 5.19730315 \\ 36000 & 5.568318898 \\ 38000 & 5.899047244 \end{array}\right)$$

$$\mathrm{Data8inSch80}_6 := \left(\begin{array}{cc} \text{Force (lbs)} & \text{Crush (inches)} \\ 0 & 0 \\ 2000 & 0.009708661 \\ 4000 & 0.017429134 \\ 6000 & 0.022291339 \\ 8000 & 0.033047244 \\ 10000 & 0.060795276 \\ 12000 & 0.095870079 \\ 14000 & 0.136255906 \\ 16000 & 0.199669291 \\ 18000 & 0.683074803 \\ 20000 & 2.893992126 \\ 22000 & 5.18373622 \\ 24000 & 6.329173228 \\ 26000 & 7.094594488 \\ 28000 & 7.674011811 \end{array}\right)$$

Texas Transportation Institute

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Job #: **444193**

By: **William Williams**

Checked: ______

Subject: **Pipe Crush Data**
for Different Size Pipe Blockouts for New Bridge Rail Design

Client: **Texas Department of Transportation**

$$\mathrm{Data10inSch80}_6 := \begin{array}{c} \begin{array}{cc} \text{Force (lbs)} & \text{Crush (inches)} \end{array} \\ \begin{pmatrix} 0 & 0 \\ 2000 & 0.019685 \\ 4000 & 0.035433 \\ 6000 & 0.07874 \\ 8000 & 0.192913 \\ 10000 & 0.397638 \\ 12000 & 0.988189 \\ 14000 & 4.055118 \\ 16000 & 6.780315 \\ 18000 & 8.449606 \\ 20000 & 9.731102 \end{pmatrix} \end{array}$$

$$\mathrm{Data6inSch80}_6 := \begin{array}{c} \begin{array}{cc} \text{Force (lbs)} & \text{Crush (inches)} \end{array} \\ \begin{pmatrix} 0 & 0 \\ 2000 & 0.040688976 \\ 4000 & 0.101011811 \\ 6000 & 0.164385827 \\ 8000 & 0.238846457 \\ 10000 & 0.350767717 \\ 12000 & 0.518098425 \\ 14000 & 0.895244094 \\ 16000 & 1.440015748 \\ 18000 & 2.453862205 \\ 20000 & 3.441066929 \\ 22000 & 4.248090551 \\ 24000 & 4.880496063 \\ 26000 & 5.411582677 \\ 28000 & 5.849248031 \end{pmatrix} \end{array}$$

$$F6inSch80_6 := \frac{Data6inSch80_6^{\langle 1 \rangle}}{1000} \cdot kips \qquad \Delta 6Sh80_6 := Data6inSch80_6^{\langle 2 \rangle} \cdot in$$

$$F6inSch40_6 := \frac{Data6inSch40_6^{\langle 1 \rangle}}{1000} \cdot kips \qquad \Delta 6Sh40_6 := Data6inSch40_6^{\langle 2 \rangle} \cdot in$$

$$F6inSch40_8 := \frac{Data6inSch40_8^{\langle 1 \rangle}}{1000} \cdot kips \qquad \Delta 6Sh40_8 := Data6inSch40_8^{\langle 2 \rangle} \cdot in$$

$$F8inSch40_6 := \frac{Data8inSch40_6^{\langle 1 \rangle}}{1000} \cdot kips \qquad \Delta 8Sh40_6 := Data8inSch40_6^{\langle 2 \rangle} \cdot in$$

$$F6inSch80_8 := \frac{Data6inSch80_8^{\langle 1 \rangle}}{1000} \cdot kips \qquad \Delta 6Sh80_8 := Data6inSch80_8^{\langle 2 \rangle} \cdot in$$

$$F8inSch80_6 := \frac{Data8inSch80_6^{\langle 1 \rangle}}{1000} \cdot kips \qquad \Delta 8Sh80_6 := Data8inSch80_6^{\langle 2 \rangle} \cdot in$$

$$F10inSch80_6 := \frac{Data10inSch80_6^{\langle 1 \rangle}}{1000} \cdot kips \qquad \Delta 10Sh80_6 := Data10inSch80_6^{\langle 2 \rangle} \cdot in$$

Texas Transportation Institute

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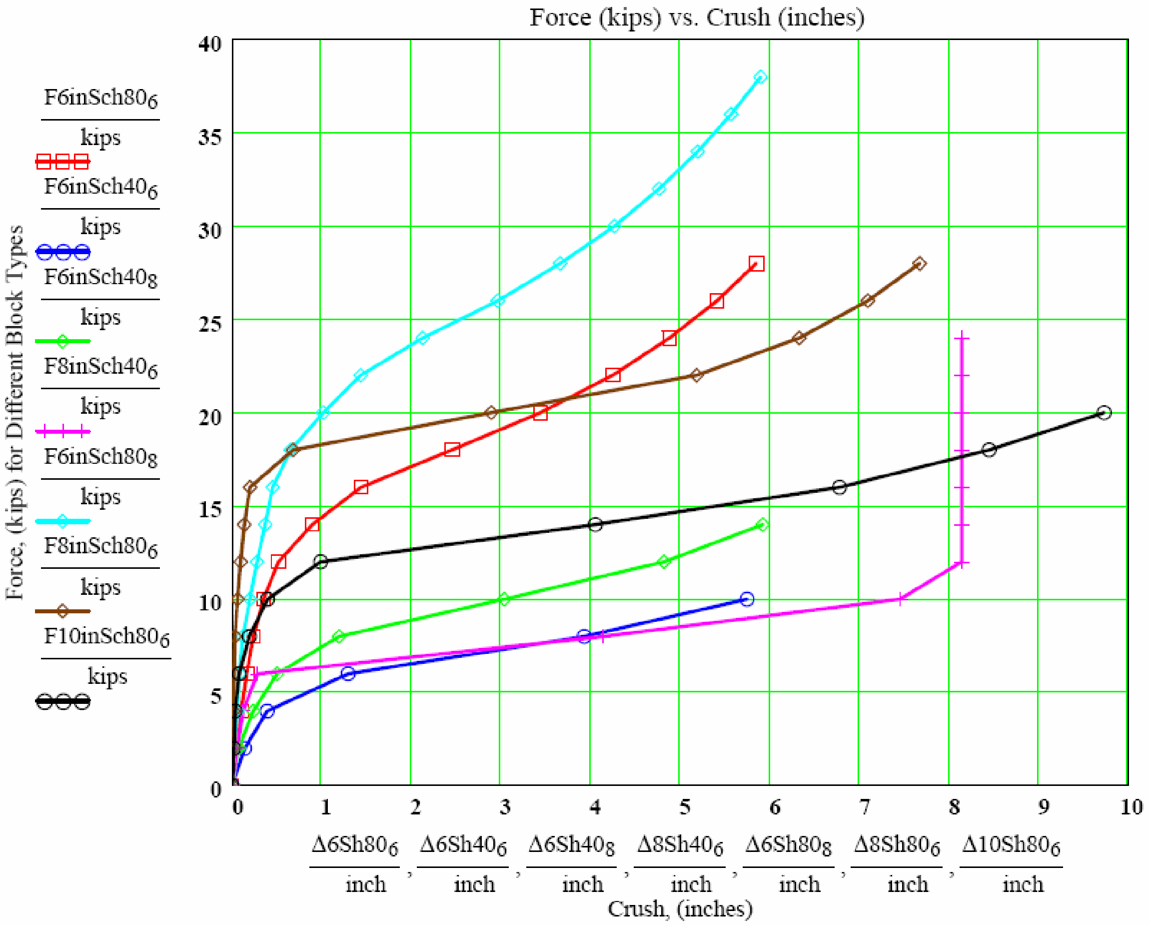
Job #: 444193

By: William Williams

Checked: ______

Subject: **Pipe Crush Data for Different Pipe Blockout Types**

Client: **Texas Department of Transportation**



Texas
Transportation
Institute

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Job #: 444193

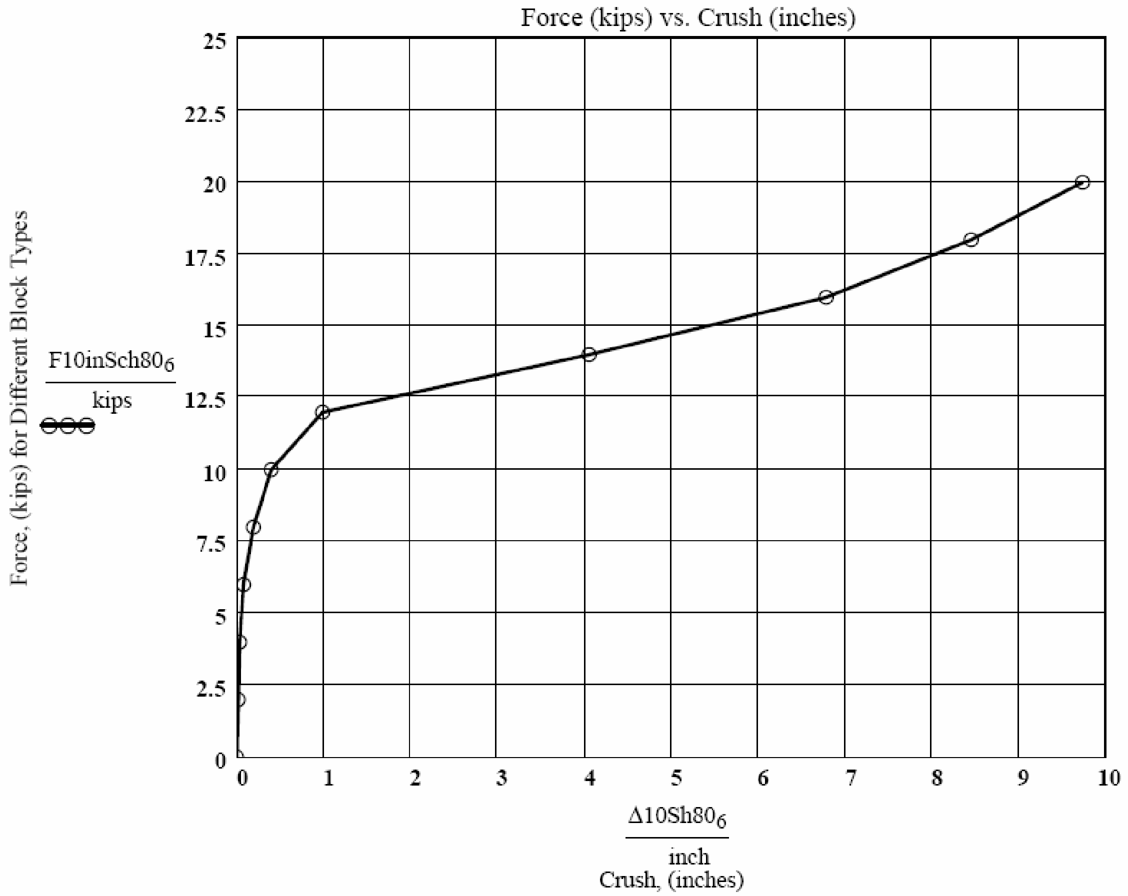
By: William Williams

Checked: ______

Subject: **Pipe Crush Data**

for 10-inch Schedule 80 Pipe Blockout ~ 6 inches Long

Client: **Texas Department of Transportation**



$F10inSch80_6$ = 10-inch Diameter Schedule 80, A53 Grade B Pipe, 6 inches in Length

Texas Transportation Institute

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Job #: 444193

By: William Williams

Checked: ______

Subject: **HSS8x6x6 Rail w/8-inch Sch 40 Pipe Blockouts ~ 6 inches Long**

STAAD Analysis Data, Load at: 1.) Mid-span; 2.) At support; 3.) End of rail

Client: **Texas Department of Transportation**

This data is for STAAD analyses on HSS8x6x6 tube continuous over 5 spans at the span lengths given using 8-inch Schedule 40 pipe blocks ~ 6 inches long, with 27 kips distributed over 4 ft at: 1.) mid-span of middle span (3rd); 2.) centered over 3rd support; and 3.) at the end of the rail

$$\mathrm{Design1}_{MID} := \begin{array}{c} \begin{array}{cccccccc} \text{Span (ft)} & F_{zsupport}\ \text{(kips)} & \text{Crush (in.)} & M_{xsupport}\ \text{(k-in)} & M_{xmidspan}\ \text{(k-in)} & \Delta_{supp.}\ \text{(in)} & \Delta_{mid}\ \text{(in)} & \text{S.R.} \end{array} \\ \begin{pmatrix} 20 & 10.17 & -10.72 & -151.91 & -1609.91 & -10.717 & -14.52 & 2.26 \\ 18 & 10.15 & -7.93 & -141.56 & -1437.56 & -7.93 & -10.70 & 2.02 \\ 16 & 9.57 & -6.2 & -181.82 & -1315.82 & -6.24 & -8.29 & 1.85 \\ 14 & 9.10 & -4.88 & -209.37 & -1181.37 & -4.88 & -6.32 & 1.66 \\ 12 & 8.83 & -4.10 & -236.12 & -1046.12 & -4.10 & -5.06 & 1.47 \\ 10 & 8.53 & -3.23 & -262.03 & -910.03 & -3.23 & -3.83 & 1.28 \end{pmatrix} \end{array}$$

Load at Mid-Span

$$\mathrm{Design1}_{SUP} := \begin{array}{c} \begin{array}{cccccccc} \text{Span (ft)} & F_{zsupp.}\ \text{(kips)} & \text{Crush (in)} & M_{xsupp}\ \text{(k-in)} & M_{xAdj.supp.}\ \text{(k-in)} & \Delta_{supp.}\ \text{(in)} & \Delta_{Adj.}\ \text{Supp. (in)} & \text{S.R.} \end{array} \\ \begin{pmatrix} 20 & 10.17 & -17.9 & -1347.24 & 555.19 & -17.92 & -6.89 & 1.9 \\ 18 & 10.17 & -14.44 & -1236.96 & 472.88 & -14.44 & -5.86 & 1.74 \\ 16 & 10.17 & -11.28 & -1127.81 & 395.33 & -11.28 & -4.81 & 1.59 \\ 14 & 10.17 & -8.39 & -1013.34 & 322.18 & -8.39 & -3.74 & 1.42 \\ 12 & 9.59 & -6.30 & -924.55 & 252.22 & -6.30 & -2.97 & 1.31 \\ 10 & 8.92 & -4.37 & -838.34 & 182.45 & -4.37 & -2.19 & 1.18 \end{pmatrix} \end{array}$$

Load at Support (Centered)

$$\mathrm{Design1}_{END} := \begin{array}{c} \begin{array}{ccccccc} \text{Span (ft)} & F_{zsup}\ \text{(kips)} & \text{Crush (in)} & F_{zupp}\ \text{(adj.) (k-in)} & \Delta_{Adj.Sup.}\ \text{(in)} & M_{xadj.}\ \text{supp. (k-in)} & \text{S.R.} \end{array} \\ \begin{pmatrix} 20 & 21.75 & -10.78 & 7.98 & -1.63 & -611.34 & 0.86 \\ 18 & 21.30 & -10.67 & 8.22 & -2.33 & -583.73 & 0.83 \\ 16 & 20.73 & -10.54 & 8.42 & -2.90 & -566.20 & 0.79 \\ 14 & 19.97 & -10.35 & 8.57 & -3.34 & -532.77 & 0.75 \\ 12 & 18.90 & -10.10 & 8.67 & -3.65 & -518.91 & 0.73 \\ 10 & 17.26 & -9.70 & 8.74 & -3.84 & -521.14 & 0.74 \end{pmatrix} \end{array}$$

Load at End of Rail

Subject: **HSS8x6x6 Rail w/8-inch Sch 80 Pipe Blockouts ~ 6 inches Long**

By: **William Williams**

STAAD Analysis Data, Load at: 1.) Mid-span; 2.) At support; 3.) End of rail

Checked: ______

Client: **Texas Department of Transportation**

This data is for STAAD analyses on HSS8x6x6 tube continuous over 5 spans at the span lengths given using 8-inch Schedule 80 pipe blocks ~ 6 inches long, with 27 kips distributed over 4 ft at: 1.) mid-span of middle span (3rd); 2.) centered over 3rd support; and 3.) at the end of the rail

$$\mathrm{Design2}_{MID} := \begin{array}{cccccccc} \text{Span (ft)} & F_{zsupport}\ \text{(kips)} & \text{Crush (in)} & M_{xsupport}\ \text{(k-in)} & M_{xmidspan}\ \text{(k-in)} & \Delta_{supp.}\ \text{(in)} & \Delta_{mid}\ \text{(in)} & \text{S.R.} \\ 20 & 15.95 & -0.23 & 486.63 & -971.37 & -0.23 & -2.03 & 1.37 \\ 18 & 15.88 & -0.23 & 430.79 & -865.21 & -0.23 & -1.55 & 1.22 \\ 16 & 15.78 & -0.23 & 373.38 & -760.62 & -0.23 & -1.16 & 1.07 \\ 14 & 15.62 & -0.22 & 313.69 & -658.31 & -0.22 & -0.86 & 0.95 \\ 12 & 15.35 & -0.22 & 250.59 & -559.41 & -0.22 & -0.63 & 0.79 \\ 10 & 14.90 & -0.21 & 182.48 & -465.52 & -0.21 & -0.46 & 0.65 \end{array}$$

Load at Mid-Span

$$\mathrm{Design2}_{SUP} := \begin{array}{cccccccc} \text{Span (ft)} & F_{zsupp.}\ \text{(kips)} & \text{Crush (in)} & M_{xsupp}\ \text{(k-in)} & M_{xAdj.supp.}\ \text{(k-in)} & \Delta_{supp.}\ \text{(in)} & \Delta_{Adj.}\ \text{Supp. (in)} & \text{S.R.} \\ 20 & 20.00 & -3.06 & -453.78 & 320.72 & -3.060 & -0.07 & 0.64 \\ 18 & 19.44 & -2.43 & -435.72 & 309.16 & -2.434 & -0.08 & 0.61 \\ 16 & 18.93 & -1.85 & -407.92 & 290.05 & -1.857 & -0.08 & 0.57 \\ 14 & 18.48 & -1.35 & -370.71 & 263.07 & -1.349 & -0.09 & 0.52 \\ 12 & 18.10 & -0.93 & -325.34 & 228.31 & -0.929 & -0.09 & 0.45 \\ 10 & 17.81 & -0.60 & -273.99 & 186.29 & -0.603 & -0.09 & 0.38 \end{array}$$

Load at Support (Centered)

$$\mathrm{Design2}_{END} := \begin{array}{ccccccc} \text{Span (ft)} & F_{zsup}\ \text{(kips)} & \text{Crush (in)} & F_{zupp}\ \text{(adj.) (k-in)} & \Delta_{Adj.Sup.}\ \text{(in)} & M_{xadj.}\ \text{supp. (k-in)} & \text{S.R.} \\ 20 & 22.21 & -5.54 & 7.38 & -0.11 & -502.54 & 0.71 \\ 18 & 21.59 & -4.84 & 8.37 & -0.12 & -521.31 & 0.74 \\ 16 & 20.87 & -4.04 & 9.47 & -0.14 & -528.80 & 0.75 \\ 14 & 20.07 & -3.14 & 10.57 & -0.15 & -515.44 & 0.73 \\ 12 & 19.24 & -2.21 & 11.48 & -0.16 & -468.80 & 0.66 \\ 10 & 18.46 & -1.33 & 11.89 & -0.17 & -376.52 & 0.54 \end{array}$$

Load at End of Rail

Texas Transportation Institute

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Job #: 444193

Subject: **HSS8x8x6 Rail w/10-inch Sch 80 Pipe Blockouts ~ 6 inches Long**

By: **William Williams**

STAAD Analysis Data, Load at: 1.) Mid-span; 2.) At support; 3.) End of rail

Checked: ______

Client: **Texas Department of Transportation**

This data is for STAAD analyses on HSS8x8x6 tube continuous over 5 spans at the span lengths given using 10-in Schedule 80 pipe blocks ~ 6 inches long, with 27 kips distributed over 4 ft at: 1.) mid-span of middle span (3rd); 2.) centered over 3rd support; and 3.) at the end of the rail

$$\text{Design3}_{\text{MID}} := \begin{array}{cccccccc}
\text{Span (ft)} & F_{zsupport}\text{ (kips)} & \text{Crush (in)} & M_{xsupport}\text{ (k-in)} & M_{xmidspan}\text{ (k-in)} & \Delta_{supp.}\text{ (in)} & \Delta_{mid}\text{ (in)} & \text{S.R.} \\
20 & 13.82 & -3.27 & -249.90 & 1208.11 & -3.27 & -5.29 & 1.35 \\
18 & 13.46 & -2.83 & -186.80 & 1109.20 & -2.83 & -4.37 & 1.24 \\
16 & 13.09 & -2.37 & -128.50 & 1005.53 & -2.37 & -3.50 & 1.12 \\
14 & 12.73 & -1.91 & -76.18 & 895.84 & -1.91 & -2.70 & 1.00 \\
12 & 12.38 & -1.48 & -30.27 & 779.73 & -1.48 & -2.00 & 0.87 \\
10 & 12.08 & -1.10 & 10.50 & 658.50 & -1.10 & -1.42 & 0.74
\end{array}$$

Load at Mid-Span

$$\text{Design3}_{\text{SUP}} := \begin{array}{ccccccccc}
\text{Span (ft)} & F_{zsupp.}\text{ (kips)} & \text{Crush (in)} & M_{xsupp}\text{ (k-in)} & M_{xAdj.supp.}\text{ (k-in)} & \Delta_{supp.}\text{ (in)} & \Delta_{Adj.}\text{ Supp. (in)} & \text{S.R.} & F_{xAdj.}\text{ supp.(kips)} \\
20 & 15.23 & -5.03 & 754.90 & -498.40 & -5.03 & -0.68 & 0.89 & 8.15 \\
18 & 14.53 & -4.16 & 764.30 & -460.68 & -4.16 & -0.70 & 0.85 & 8.43 \\
16 & 13.90 & -3.37 & 722.26 & -412.24 & -3.37 & -0.72 & 0.81 & 8.70 \\
14 & 13.35 & -2.69 & 669.93 & -354.13 & -2.69 & -0.72 & 0.75 & 8.91 \\
12 & 12.88 & -2.10 & 609.60 & -287.83 & -2.10 & -0.70 & 0.68 & 8.92 \\
10 & 12.49 & -1.62 & 544.31 & -213.94 & -1.62 & -0.68 & 0.61 & 8.61
\end{array}$$

Load at Support (Centered)

$$\text{Design3}_{\text{END}} := \begin{array}{ccccccc}
\text{Span (ft)} & F_{zsup}\text{ (kips)} & \text{Crush (in)} & F_{zupp}\text{ (adj.) (k-in)} & \Delta_{Adj.Sup.}\text{ (in)} & M_{xadj.}\text{ supp. (k-in)} & \text{S.R.} \\
20 & 20.91 & -10.43 & 9.83 & -0.82 & -813.19 & 0.91 \\
18 & 19.84 & -10.16 & 11.58 & -0.96 & -897.81 & 1.00 \\
16 & 18.99 & -9.74 & 12.42 & -1.53 & -889.08 & 1.00 \\
14 & 18.22 & -8.78 & 12.76 & -1.95 & -826.29 & 0.93 \\
12 & 17.31 & -7.64 & 12.94 & -2.17 & -746.76 & 0.84 \\
10 & 16.22 & -6.28 & 12.94 & -2.18 & -645.29 & 0.73
\end{array}$$

Load at End of Rail

Texas Transportation Institute

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Job #: 444193

Subject: **HSS8x8x5 Rail w/10-inch Sch 80 Pipe Blockouts ~ 6 inches Long** By: **William Williams**

STAAD Analysis Data, Load at: 1.) Mid-span; 2.) At support; 3.) End of rail Checked: ______

Client: **Texas Department of Transportation**

This data is for STAAD analyses on HSS8x8x5 tube continuous over 5 spans at the spanlengths given using 10-inch Schedule 80 pipe blocks ~ 6 inches long, with 27 kips distributed over 4 ft at: 1.) mid-span of middle span (3rd); 2.) centered over 3rd support; and 3.) at the end of the rail

$$\mathrm{Design4}_{MID} := \begin{array}{cccccccc} \begin{matrix}\text{Span}\\(\text{ft})\end{matrix} & \begin{matrix}F_{zsupport}\\(\text{kips})\end{matrix} & \begin{matrix}\text{Crush}\\(\text{in})\end{matrix} & \begin{matrix}M_{xsupport}\\(\text{k-in})\end{matrix} & \begin{matrix}M_{xmidspan}\\(\text{k-in})\end{matrix} & \begin{matrix}\Delta_{supp.}\\(\text{in})\end{matrix} & \begin{matrix}\Delta_{mid}\\(\text{in.})\end{matrix} & \text{S.R.} \\ 20 & 13.92 & -3.40 & -261.70 & 1196.30 & -3.40 & -5.58 & 1.38 \\ 18 & 13.56 & -2.96 & -197.54 & 1098.46 & -2.96 & -4.62 & 1.27 \\ 16 & 13.19 & -2.48 & -137.79 & 996.21 & -2.48 & -3.71 & 1.15 \\ 14 & 12.80 & -2.00 & -83.98 & 888.02 & -2.00 & -2.87 & 1.03 \\ 12 & 12.44 & -1.55 & -36.83 & 773.19 & -1.55 & -2.12 & 0.89 \\ 10 & 12.12 & -1.15 & 4.69 & 652.69 & -1.15 & -1.50 & 0.75 \end{array}$$

Load at Mid-Span

$$\mathrm{Design4}_{SUP} := \begin{array}{cccccccc} \begin{matrix}\text{Span}\\(\text{ft})\end{matrix} & \begin{matrix}F_{zsupp.}\\(\text{kips})\end{matrix} & \begin{matrix}\text{Crush}\\(\text{in})\end{matrix} & \begin{matrix}M_{xsupp}\\(\text{k-in})\end{matrix} & \begin{matrix}M_{xAdj.supp.}\\(\text{k-in})\end{matrix} & \begin{matrix}\Delta_{supp.}\\(\text{in})\end{matrix} & \begin{matrix}\Delta_{Adj.}\\\text{Supp.}\\(\text{in})\end{matrix} & \text{S.R.} \\ 20 & 15.45 & -5.32 & 776.20 & -492.41 & -5.32 & -0.67 & 0.90 \\ 18 & 14.72 & -4.40 & 749.01 & -457.86 & -4.40 & -0.70 & 0.87 \\ 16 & 14.05 & -3.56 & 709.43 & -412.15 & -3.56 & -0.71 & 0.82 \\ 14 & 13.46 & -2.83 & 658.81 & -356.11 & -2.83 & -0.72 & 0.76 \\ 12 & 12.96 & -2.20 & 599.46 & -291.31 & -2.20 & -0.71 & 0.69 \\ 10 & 12.55 & -1.69 & 534.43 & -218.74 & -1.69 & -0.69 & 0.62 \end{array}$$

Load at Support (Centered)

$$\mathrm{Design4}_{END} := \begin{array}{ccccccc} \begin{matrix}\text{Span}\\(\text{ft})\end{matrix} & \begin{matrix}F_{zsup}\\(\text{kips})\end{matrix} & \begin{matrix}\text{Crush}\\(\text{in})\end{matrix} & \begin{matrix}F_{zupp}\\(\text{adj.})\\(\text{k-in})\end{matrix} & \begin{matrix}\Delta_{Adj.Sup.}\\(\text{in})\end{matrix} & \begin{matrix}M_{xadj.}\\\text{supp.}\\(\text{k-in})\end{matrix} & \text{S.R.} \\ 20 & 21.29 & -9.74 & 9.05 & -0.75 & -721.18 & 0.83 \\ 18 & 20.32 & -9.49 & 10.63 & -0.89 & -794.64 & 0.92 \\ 16 & 19.23 & -9.21 & 12.17 & -1.20 & -844.16 & 0.98 \\ 14 & 18.34 & -8.92 & 12.70 & -1.86 & -807.36 & 0.93 \\ 12 & 17.43 & -7.79 & 12.91 & -2.14 & -730.15 & 0.84 \\ 10 & 16.34 & -6.42 & 12.94 & -2.18 & -631.47 & 0.73 \end{array}$$

Load at End of Rail

Subject: **HSS8x8x4 Rail w/10-inch Sch 80 Pipe Blockouts ~ 6 inches Long** By: **William Williams**

STAAD Analysis Data, Load at: 1.) Mid-span; 2.) At support; 3.) End of rail Checked: ______

Client: **Texas Department of Transportation**

This data is for STAAD analyses on HSS8x8x4 tube continuous over 5 spans at the span lengths given using 10-inch Schedule 80 pipe blocks ~ 6 inches long, with 27 kips distributed over 4 ft at: 1.) mid-span of middle span (3rd); 2.) centered over 3rd support; and 3.) at the end of the rail

$$\begin{array}{c|cccccccc}
 & \begin{matrix}\text{Span}\\ \text{(ft)}\end{matrix} & \begin{matrix}F_{zsupport}\\ \text{(kips)}\end{matrix} & \begin{matrix}\text{Crush}\\ \text{(in)}\end{matrix} & \begin{matrix}M_{xsupport}\\ \text{(k-in)}\end{matrix} & \begin{matrix}M_{xmidspan}\\ \text{(k-in)}\end{matrix} & \begin{matrix}\Delta_{supp.}\\ \text{(in)}\end{matrix} & \begin{matrix}\Delta_{mid}\\ \text{(in)}\end{matrix} & \text{S.R.} \\
\text{Design5}_{MID} := & 20 & 14.13 & -3.66 & -285.97 & 1172.03 & -3.66 & -6.22 & 1.65 \\
 & 18 & 13.78 & -3.21 & -219.91 & 1076.09 & -3.22 & -5.17 & 1.52 \\
 & 16 & 13.38 & -2.72 & -157.47 & 976.53 & -2.73 & -4.17 & 1.38 \\
 & 14 & 12.98 & -2.22 & -100.55 & 871.45 & -2.22 & -3.24 & 1.23 \\
 & 12 & 12.58 & -1.72 & -50.50 & 759.50 & -1.72 & -2.39 & 1.07 \\
 & 10 & 12.22 & -1.27 & -7.11 & 640.88 & -1.27 & -1.68 & 0.90
\end{array}$$

Load at
Mid-Span

$$\begin{array}{c|cccccccc}
 & \begin{matrix}\text{Span}\\ \text{(ft)}\end{matrix} & \begin{matrix}F_{zsupp.}\\ \text{(kips)}\end{matrix} & \begin{matrix}\text{Crush}\\ \text{(in)}\end{matrix} & \begin{matrix}M_{xsupp}\\ \text{(k-in)}\end{matrix} & \begin{matrix}M_{xAdj.supp.}\\ \text{(k-in)}\end{matrix} & \begin{matrix}\Delta_{supp.}\\ \text{(in)}\end{matrix} & \begin{matrix}\Delta_{Adj.}\\ \text{Supp.}\\ \text{(in)}\end{matrix} & \text{S.R.} \\
\text{Design5}_{SUP} := & 20 & 15.96 & -5.95 & 736.67 & -477.22 & -5.95 & -0.65 & 1.04 \\
 & 18 & 15.14 & -4.93 & 716.12 & -449.59 & -4.93 & -0.68 & 1.01 \\
 & 16 & 14.39 & -3.98 & 682.14 & -409.99 & -3.98 & -0.71 & 0.96 \\
 & 14 & 13.72 & -3.15 & 635.67 & -358.81 & -3.15 & -0.72 & 0.90 \\
 & 12 & 13.14 & -2.43 & 578.83 & -297.47 & -2.43 & -0.71 & 0.82 \\
 & 10 & 12.67 & -1.84 & 514.75 & -227.62 & -1.84 & -0.70 & 0.73
\end{array}$$

Load at
Support
(Centered)

$$\begin{array}{c|ccccccc}
 & \begin{matrix}\text{Span}\\ \text{(ft)}\end{matrix} & \begin{matrix}F_{zsup}\\ \text{(kips)}\end{matrix} & \begin{matrix}\text{Crush}\\ \text{(in)}\end{matrix} & \begin{matrix}F_{zupp}\\ \text{(adj.)}\\ \text{(k-in)}\end{matrix} & \begin{matrix}\Delta_{Adj.Sup.}\\ \text{(in)}\end{matrix} & \begin{matrix}M_{xadj.}\\ \text{supp.}\\ \text{(k-in)}\end{matrix} & \text{S.R.} \\
\text{Design5}_{END} := & 20 & 21.63 & -9.82 & 8.39 & -0.70 & -640.08 & 0.90 \\
 & 18 & 20.73 & -9.60 & 9.86 & -0.82 & -706.62 & 1.00 \\
 & 16 & 19.56 & -9.30 & 11.71 & -0.98 & -781.26 & 1.10 \\
 & 14 & 18.60 & -9.05 & 12.53 & -1.66 & -763.43 & 1.08 \\
 & 12 & 17.66 & -8.08 & 12.85 & -2.06 & -696.45 & 0.98 \\
 & 10 & 16.57 & -6.72 & 12.94 & -2.18 & -603.21 & 0.85
\end{array}$$

Load at
End of Rail

Texas Transportation Institute

Page: **14 of 16**

Job #: **444193**

Subject: **New Rail Design for New Truss Bridges**

Matrix Data Summary

By: **William Williams**

Client: **Texas Department of Transportation**

Checked: ______

Data for plotting and graphing from the matrices above

$\mathrm{Span}_1 := \mathrm{Design1}_{\mathrm{SUP}}^{\langle 1\rangle} \cdot \mathrm{ft}$

$\mathrm{Span}_2 := \mathrm{Design2}_{\mathrm{SUP}}^{\langle 1\rangle} \cdot \mathrm{ft}$ $\quad$ $\mathrm{Crush}_{\mathrm{SUPP2}} := \mathrm{Design2}_{\mathrm{SUP}}^{\langle 3\rangle}\ \mathrm{in} \cdot -1$ $\quad$ $\mathrm{Crush}_{\mathrm{END2}} := \mathrm{Design2}_{\mathrm{END}}^{\langle 3\rangle}\ \mathrm{in} \cdot -1$

$\mathrm{Span}_3 := \mathrm{Design3}_{\mathrm{SUP}}^{\langle 1\rangle} \cdot \mathrm{ft}$ $\quad$ $\mathrm{Crush}_{\mathrm{SUPP3}} := \mathrm{Design3}_{\mathrm{SUP}}^{\langle 3\rangle} \cdot \mathrm{in} \cdot -1$ $\quad$ $\mathrm{Crush}_{\mathrm{END3}} := \mathrm{Design3}_{\mathrm{END}}^{\langle 3\rangle}\ \mathrm{in} \cdot -1$

$\mathrm{Span}_4 := \mathrm{Design4}_{\mathrm{SUP}}^{\langle 1\rangle} \cdot \mathrm{ft}$ $\quad$ $\mathrm{Crush}_{\mathrm{SUPP4}} := \mathrm{Design4}_{\mathrm{SUP}}^{\langle 3\rangle} \cdot \mathrm{in} \cdot -1$ $\quad$ $\mathrm{Crush}_{\mathrm{END4}} := \mathrm{Design4}_{\mathrm{END}}^{\langle 3\rangle}\ \mathrm{in} \cdot -1$

$\mathrm{Span}_5 := \mathrm{Design5}_{\mathrm{SUP}}^{\langle 1\rangle} \cdot \mathrm{ft}$ $\quad$ $\mathrm{Crush}_{\mathrm{SUPP5}} := \mathrm{Design5}_{\mathrm{SUP}}^{\langle 3\rangle} \cdot \mathrm{in} \cdot -1$ $\quad$ $\mathrm{Crush}_{\mathrm{END5}} := \mathrm{Design5}_{\mathrm{END}}^{\langle 3\rangle}\ \mathrm{in} \cdot -1$

$F_{\mathrm{SUPP2}} := \mathrm{Design2}_{\mathrm{SUP}}^{\langle 2\rangle} \cdot \mathrm{kips}$ $\quad$ $F_{\mathrm{END2}} := \mathrm{Design2}_{\mathrm{END}}^{\langle 2\rangle} \cdot \mathrm{kips}$

$F_{\mathrm{SUPP3}} := \mathrm{Design3}_{\mathrm{SUP}}^{\langle 2\rangle} \cdot \mathrm{kips}$ $\quad$ $F_{\mathrm{END3}} := \mathrm{Design3}_{\mathrm{END}}^{\langle 2\rangle} \cdot \mathrm{kips}$

$F_{\mathrm{SUPP4}} := \mathrm{Design4}_{\mathrm{SUP}}^{\langle 2\rangle} \cdot \mathrm{kips}$ $\quad$ $F_{\mathrm{END4}} := \mathrm{Design4}_{\mathrm{END}}^{\langle 2\rangle} \cdot \mathrm{kips}$

$F_{\mathrm{SUPP5}} := \mathrm{Design5}_{\mathrm{SUP}}^{\langle 2\rangle} \cdot \mathrm{kips}$ $\quad$ $F_{\mathrm{END5}} := \mathrm{Design5}_{\mathrm{END}}^{\langle 2\rangle} \cdot \mathrm{kips}$

Texas Transportation Institute

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Job #: 444193

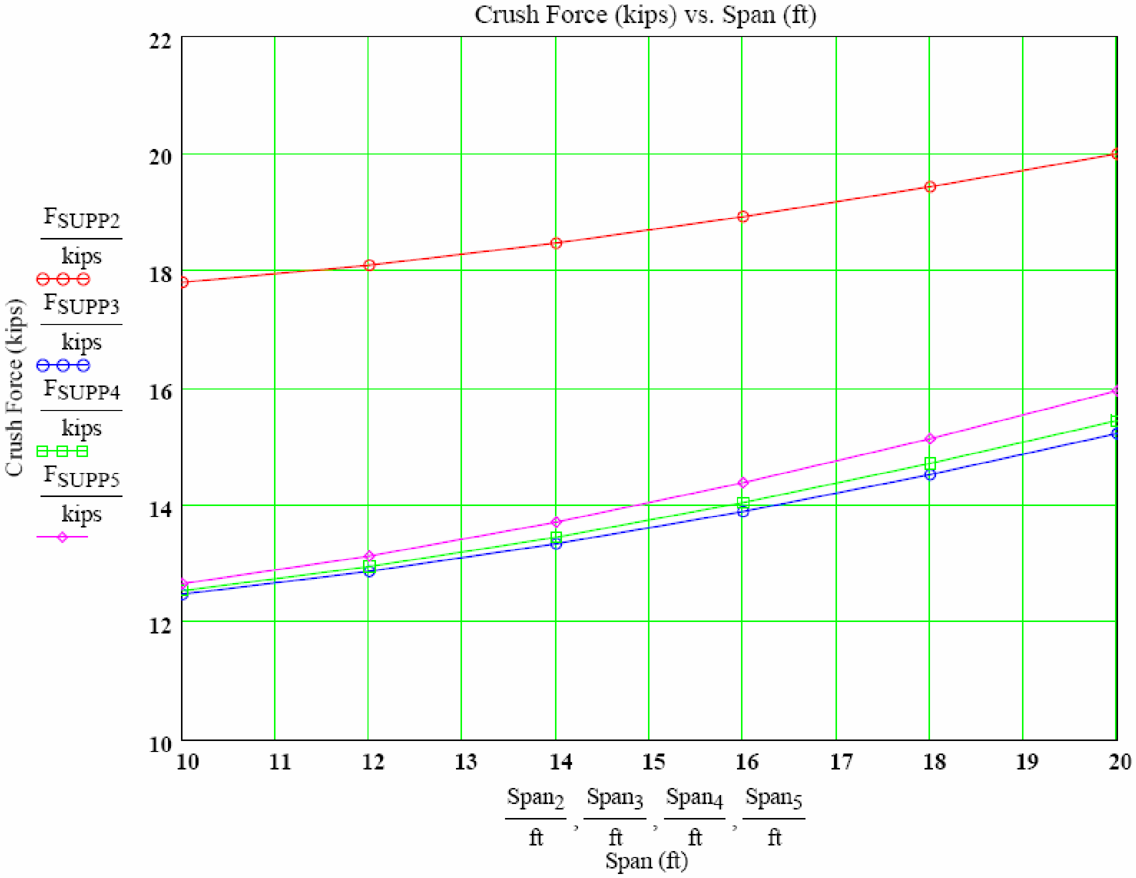
Subject: New Rail Design for New Truss Bridges

Plot of Force vs. Span for Load @ support case

By: William Williams

Checked: ______

Client: Texas Department of Transportation



Design #2: HSS 8x6x6 w/ 8-inch Schedule 80 Pipe Blocks, 6 inches long

Design #3: HSS8x8x6 w/ 10-inch Schedule 80 Pipe Blocks, 6 inches long

Design #4: HSS8x8x5 w/ 10-inch Schedule 80 Pipe Blocks, 6 inches long

Design #5: HSS8x8x4 w/ 10-inch Schedule 80 Pipe Blocks, 6 inches long

Texas Transportation Institute

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Job #: 444193

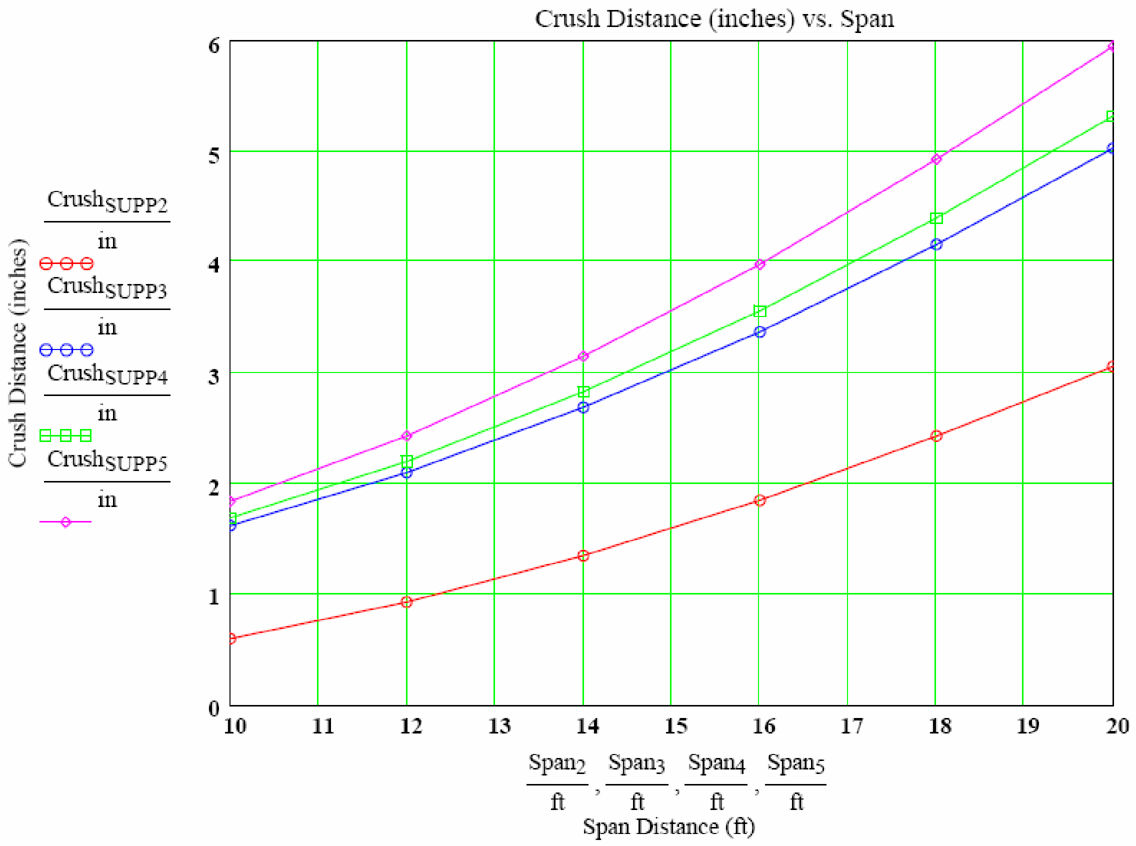
Subject: New Rail Design for New Truss Bridges

By: William Williams

Plot of Crush vs. Span for Load @ support case

Checked: ______

Client: Texas Department of Transportation



Design #2: HSS 8x6x6 with 8-inch Schedule 80 Pipe Blocks, 6 inches long

Design #3: HSS8x8x6 with 10-inch Schedule 80 Pipe Blocks, 6 inches long

Design #4: HSS8x8x5 with 10-inch Schedule 80 Pipe Blocks, 6 inches long

Design #5: HSS8x8x4 with 10-inch Schedule 80 Pipe Blocks, 6 inches long