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16. Abstract

The American Association of State Highway and Transportation Officials (AASHTO) Load and Resistance Factor Design (*LRFD*) *Bridge Design Specifications* require that "abutments and piers located within a distance of 30.0 ft of the edge of the roadway, or within a distance of 50.0 ft to the centerline of a railway track, shall be designed for an equivalent static force of 400 kip…" Magnitude of the design force (400 kip) was established from data available at the time the *LRFD* specification was prepared. Supporting documentation for this design requirement, both its applicability and magnitude of the design force, was not extensive. Further detailed guidance for the design engineer is not available.

The objective of this research effort is to address the following questions:

1. What risks warrant application of this requirement?
2. Is the magnitude of design force (400 kip) appropriate?

This is a report of work performed under Phase 1 of a multi-state pooled funds project entitled "Guidelines for Designing Bridge Piers and Abutments for Vehicle Collisions."

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ANALYSIS OF LARGE TRUCK COLLISIONS WITH BRIDGE PIERS: PHASE 1. REPORT OF GUIDELINES FOR DESIGNING BRIDGE PIERS AND ABUTMENTS FOR VEHICLE COLLISIONS

by

C. Eugene Buth, P.E.
Research Fellow
Texas Transportation Institute

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

Michael S. Brackin
Graduate Assistant Research
Texas Transportation Institute

Dominque Lord
Assistant Professor
Texas A&M University

Srinivas R. Geedipally
Engineering Research Associate
Texas Transportation Institute

and

Akram Y. Abu-Odeh
Associate Research Scientist
Texas Transportation Institute

DISCLAIMER

This research was performed in cooperation with the Texas Department of Transportation (TxDOT) and the Federal Highway Administration (FHWA). The contents of this report reflect the views of the authors, who are responsible for the facts and the accuracy of the data presented herein. The contents do not necessarily reflect the official view or policies of the FHWA or TxDOT. 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 United States Government and the State of Texas do not endorse products or manufacturers. Trade or manufacturers' names appear herein solely because they are considered essential to the object of this report. The engineer in charge of the project was C. Eugene Buth, P.E. (Texas, #27579).

ACKNOWLEDGMENTS

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CHAPTER 1. INTRODUCTION

INTRODUCTION

This is a report of work performed under Phase I of a multi-state pooled funds project entitled "Guidelines for Designing Bridge Piers and Abutments for Vehicle Collisions."

BACKGROUND

The American Association of State Highway and Transportation Officials (AASHTO) *Load and Resistance Factor Design* (*LRFD*) *Bridge Design Specifications* require that "abutments and piers located within a distance of 30.0 ft of the edge of the roadway, or within a distance of 50.0 ft to the centerline of a railway track, shall be designed for an equivalent static force of 400 kip…" *(1)*. Supporting documentation for this design requirement, both its applicability and magnitude of the design force, is not extensive. Further detailed guidance for the design engineer is not available.

Magnitude of the design force (400 kip) was established from data available at the time the *LRFD* specification was prepared. Additional data/information are now available and more are needed to address whether the magnitude of the 400 kip design force should be changed. Recent tests with single-unit trucks colliding with fixed bollards and concrete walls have yielded data that will be applicable. More information for heavily loaded articulated vehicles is still needed. Some helpful information might be obtained from reconstruction of recent collisions of such vehicles that have occurred in the field.

OBJECTIVES/SCOPE OF RESEARCH

The objective of this research effort is to address the following questions:

1. What risks warrant application of this requirement?
2. Is the magnitude of design force (400 kip) appropriate?

Phase 1 included the following tasks:

1a. Literature review.
1b. Computer simulations of vehicle/bridge column and abutment collisions.
1c. Accident survey and analysis study.
1d. Development of a risk analysis methodology for vehicle/bridge column and abutment collisions (analogous to AASHTO *LRFD* vessel impact requirements).
1e. Detailed justification and work plan for research (if any) to be conducted under Phase 2 of the project.

1f.   Provide facilities and host a meeting to present Phase 1 results to project sponsors, including pooled fund project participants from other state Departments of Transportation (DOTs).

Phase 2 may include the following tasks:

2a.   Crash testing with a single-unit truck to verify loading from Phase 1 literature survey and computer simulations.

2b.   Crash testing of a 5-axle tractor-trailer rig to verify loading from Phase 1 literature survey and computer simulations.

CHAPTER 2. ACCIDENT INVESTIGATIONS/DATA

Several accidents involving large truck-tractor-trailer collisions with bridge piers were investigated as part of this project. Information such as vehicle speed, weight, and bridge pier details were gathered. In many of the investigations, interviews were conducted with law enforcement personnel who were familiar with the accidents. Information obtained from the accidents investigated for this project is provided as follows.

ACCIDENT #1 – FM 2110 BRIDGE OVER IH-30, TEXARKANA, TEXAS

On August 8, 1994, at approximately 3:00 a.m., a truck-tractor-trailer loaded with two large coils of steel crashed into a bridge pier on Interstate Highway (IH)-30 about 10 miles west of Texarkana, Texas. This bridge is located on Farm-to-Market (FM) 2110 (West 22nd Street) approximately ¼-mile north of the Red River Army Depot. This vehicle impacted the easternmost pier of the center 2-pier bent located in the median of IH-30. The collision with the pier caused two spans of the bridge to collapse. The collision killed the driver of the truck and a passenger. The truck was hauling several large coils of steel. On July 2, 2007, Officer Kevin Lorance was interviewed at the scene of the accident to gather additional information about the accident. Officer Lorance was the Texas Department of Public Safety (DPS) Official who was present on the scene immediately after the accident. Officer Lorance indicated that the weight of the vehicle was near 80,000 lb, and the impact speed was approximately 60 mph. The truck, traveling westbound on IH-30, made a sharp turn off the interstate and entered the median 280 ft from the bridge. The truck went behind the center guardrail and struck the eastern column of the center bent. There were no signs of any braking. The column impacted was a 30-inch diameter pier with eight #9 size rebars in the longitudinal direction of the pier. Transverse reinforcement in the column consisted of #2 spiral stirrup reinforcement with a 6-inch pitch. Figure 2.1 shows a photo of the accident.

ACCIDENT #2 – CHATFIELD ROAD BRIDGE OVER IH-35, NAVARRO COUNTY, TEXAS

On May 30, 2007, at approximately 4:15 a.m., a truck-tractor-trailer loaded with home building products crashed into a bridge pier on IH-45 about 3 miles east of Corsicana, Texas. This bridge is located on Roane Road and carries traffic over IH-45. This vehicle impacted the northernmost 30-inch diameter pier of the center 2-pier bent located in the median of IH-45. The collision with the pier caused severe cracking in the 30-inch diameter pier. The bridge did not collapse as a result of impact. The collision did not kill the driver. On February 21, 2008, Officer Casey Croker was interviewed at the scene of the accident to gather additional information about the accident. Officer Croker was the Texas DPS Official who was present on the scene immediately after the accident. Officer Croker indicated that the weight of the vehicle and payload was approximately at 80,000 lb, and the impact speed was approximately 60 mph. Officer Croker indicated that the driver fell asleep while driving the vehicle. The truck, traveling southbound on IH-45, drifted off the roadway and impacted a cable median barrier. The truck

entered the median approximately 300 ft from the bridge. The truck impacted the northern column of the center bent. There were no signs of any braking. The impacted column was a 30-inch diameter pier with eight #9 size rebars in the longitudinal direction of the pier. Transverse reinforcement in the column consisted of #2 spiral stirrup reinforcement with a 6-inch pitch. A photo of the accident is shown in Figure 2.2.

Figure 2.1. Truck Accident FM 2110 Bridge over IH-30, Texarkana, Texas.

Figure 2.2. Truck Accident Chatfield Road Bridge, IH-45, Navarro County, Texas.

ACCIDENT #3 – TANCAHUA STREET BRIDGE OVER IH-37, CORPUS CHRISTI, TEXAS

On May 14, 2004, at approximately 9:00 a.m., a truck-tractor with tanker loaded with a flammable compressed gas crashed into a bridge pier on IH-37 in downtown Corpus Christi, Texas. This bridge is located on Tancahua Street and carries traffic over IH-37. This vehicle impacted the easternmost 30-inch diameter pier of the center 3-pier bent located in the median of IH-37. The collision with the pier caused failure in the 30-inch diameter pier. The bridge did not collapse as a result of impact. The collision killed the driver. On February 19, 2007, Officer M. Staff was interviewed at the scene of the accident to gather additional information about the accident. Officer Staff was the Texas DPS Official who was present on scene immediately after the accident. Officer Staff indicated that the approximate speed of the vehicle was near 55 mph and overturned off the ramp curve onto IH-37. The vehicle load was approximately 72,000 lb. The column impacted was a 30-inch diameter pier with eight #9 size rebars in the longitudinal direction of the pier. Transverse reinforcement in the column consisted of #2 spiral stirrup reinforcement with a 6-inch pitch. Figure 2.3 shows a photo of the accident.

Figure 2.3. Truck Accident Tancahua Street Bridge over IH-37, Corpus Christi, Texas.

ACCIDENT #4 – BRIDGE AT IH-35 AND US-77, RED OAK, TEXAS

On July 7, 2005, a truck-tractor-trailer loaded with an unknown load crashed into a bridge pier on IH-35 in Red Oak, Texas. This bridge is located on US-77 and carries traffic over IH-35. This vehicle impacted the northernmost 30-inch diameter pier of the center 3-pier bent located in the median of IH-35. The collision with the pier caused failure in the 30-inch diameter pier. The bridge did not collapse as a result of impact. The collision killed the driver. A phone interview was conducted with Corporal Josh Newman. Corporal Newman was the Texas DPS Official who was present on scene immediately after the accident. Corporal Newman indicated that the vehicle was speeding in excess of 60 mph when it struck the bridge pier. Based on the photos taken after the accident, the vehicle appeared to be empty. The column impacted was a 30-inch diameter pier with eight #9 size rebars in the longitudinal direction of the pier. Transverse reinforcement in the column consisted of #2 spiral stirrup reinforcement with a 6-inch pitch. Photos of the accident are shown in Figure 2.4.

Figure 2.4. Truck Accident at Bridge at IH-35 and US-77, Red Oak, Texas.

ACCIDENT #5 – FM 2207 BRIDGE OVER IH-20, TYLER, TEXAS

Several years ago, a truck-tractor-trailer loaded with structural steel crashed into a bridge pier on IH-20 near Tyler, Texas. This bridge is located on FM 2207 and carries traffic over IH-20. This vehicle impacted the easternmost 30-inch diameter pier of the 2-pier bent located on the shoulder of the westbound lanes of IH-20. The collision with the pier caused failure in the 30-inch diameter pier. The bridge did not collapse as a result of impact. Reinforcement in the pier consisted of eight #9 size longitudinal bars equally spaced. Transverse reinforcement in the column consisted of #2 spiral stirrup reinforcement with a 6-inch pitch. Figure 2.5 shows photo of the accident.

Figure 2.5. Truck Accident – FM 2207 Bridge over IH-20, Tyler, Texas.

ACCIDENT #6 – BRIDGE OVER IH-45, DALLAS COUNTY, TEXAS

In May 1965, a truck-tractor-trailer with an unknown load crashed into a bridge pier on IH-45 in Dallas County, Texas. This vehicle impacted a 30-inch diameter pier of the 2-pier bent located in the median of IH-45. The collision with the pier caused failure in the 30-inch diameter pier. The bridge collapsed as a result of the impact. Reinforcement in the pier is unknown. This accident was one of the first collisions to cause catastrophic failure/collapse of a bridge in Texas from a vehicular impact.

ACCIDENT #7 – PYKE ROAD BRIDGE OVER IH-10, SEALY, TEXAS

On January 28, 2004, a truck-tractor-trailer loaded with structural steel sheet piling crashed into a bridge pier on IH-10 near Sealy, Texas. This bridge is located on Pyke Road and carries traffic over IH-10. This vehicle impacted the westernmost 30-inch diameter pier of the center 2-pier bent located on the shoulder of the westbound lanes of IH-10. The collision with the pier caused failure in the 30-inch diameter pier. The bridge did not collapse as a result of impact. The collision killed the driver. Lieutenant Reese with the Sealy Police Department was interviewed at the accident site on February 14, 2008. Lieutenant Reese was present on the scene immediately after the accident. Lieutenant Reese indicated that the vehicle was traveling approximately 50 mph with a vehicle weight near 80,000 lb when it struck the pier. The column impacted was a 30-inch diameter pier with eight #9 size rebars in the longitudinal direction of the pier. Transverse reinforcement in the pier consisted of #2 spiral stirrup reinforcement with a 6-inch pitch. Photos of the accident are shown in Figure 2.6.

ACCIDENT #8 – SH 14 BRIDGE OVER IH-45, CORSICANA, TEXAS

On September 8, 2002, a truck-tractor-trailer loaded with paper crashed into a bridge pier on IH-45 near Corsicana, Texas. This bridge is located on State Highway (SH) 14 and carries traffic over and onto IH-45. This vehicle impacted the southernmost 30-inch diameter pier of the center 2-pier bent located on the shoulder of the southbound lanes of IH-45. The collision with the pier caused failure in the 30-inch diameter pier. The bridge collapsed as a result of impact. The collision killed the driver. State Trooper J. Authier with Texas DPS was interviewed at the accident site on March 4, 2008. Officer Authier was present on the scene immediately after the accident. State Trooper Authier indicated that the vehicle was traveling at a high rate of speed with a vehicle weight near 80,000 lb when it struck the pier. The column impacted was a 30-inch diameter pier with eight #9 size rebars in the longitudinal direction of the pier. Transverse reinforcement in the pier consisted of #2 spiral stirrup reinforcement with a 6-inch pitch. A photo of the accident is shown in Figure 2.7.

Figure 2.6. Truck Accident – Pyke Road Bridge over IH-10, Sealy, Texas.

Figure 2.7. Truck Accident – SH 14 Bridge over IH-45, Corsicana, Texas.

ACCIDENT #9 – BRIDGE ON 26½ ROAD OVER IH-70, GRAND JUNCTION, COLORADO

On August 15, 2007, a truck-tractor-trailer loaded with 55-gallon barrels of sodium hypochlorite (flammable liquid) crashed into a bridge pier located on the shoulder of the westbound lanes of IH-70 in Grand Junction, Colorado. This bridge is located on 26½ Road over IH-70. A phone interview was conducted with Colorado State Trooper John Ferguson. The vehicle impacted the bridge pier at a high rate of speed. Structural details for the bridge were not obtained. Figure 2.8 shows a photo of the accident.

ACCIDENT #10 – IH-20 OVER RABBIT CREEK, LONGVIEW, TEXAS

On September 6, 2007, a truck-tractor-trailer loaded with an unknown load crashed into a bridge pier on IH-20 near Longview, Texas. This bridge is supported by numerous 2- and 3-column bents. The vehicle impacted an exterior 24-inch diameter pier of an interior 3-pier bent located over Rabbit Creek of the eastbound lanes of IH-20. The collision with the pier caused failure in the 24-inch diameter pier. A phone interview was conducted with Officer Chris Brock of Texas DPS. Officer Brock indicated that the vehicle weight was estimated to be near 80,000 lb. The speed of the truck as it left the roadway was estimated to be between 70 and 75 mph. The bridge did not collapse as a result of the impact. The pier impacted was a 24-inch diameter pier with eight #7 size rebars in the longitudinal direction of the pier. Transverse reinforcement in the pier consisted of #2 spiral stirrup reinforcement with a 6-inch pitch. A photo of the accident is shown as Figure 2.9.

Figure 2.8. Truck Accident – Bridge on 26½ Road over IH-70, Grand Junction, Colorado.

Figure 2.9. Truck Accident – IH-20 Bridge over Rabbit Creek, Longview, Texas.

ACCIDENT #11 – IH-240 OVER IH-40, SHELBY COUNTY, TENNESSEE

On September 28, 2007, a truck-tractor-trailer loaded with produce struck an exterior pier of a bridge carrying IH-240 over IH-40, Memphis, Tennessee. The vehicle speed and weight are unknown. The 30-inch diameter pier suffered minimal damage. Structural details for the bridge pier are not known at the time of this writing.

ACCIDENT #12 – IH-275 NORTH RAMP BRIDGE AT IH-40 EAST, KNOXVILLE, TENNESSEE

On December 5, 2003, a truck-tractor-trailer overturned on IH-275 North ramp at IH-40 East in Knoxville, Tennessee. The vehicle overturned and fell to the roadway below and impacted a large bent supporting the elevated ramp. The large bent was slightly damaged. A police report was obtained. Structural details have not been obtained on the bridge pier impacted by the vehicle.

ACCIDENT #13 – AUTUMN AVENUE OVER IH-40 RAMP AND IH-240, SHELBY COUNTY, TENNESSEE

In December 1988, a propane tanker impacted near a bridge pier on the IH-40 ramp near the Autumn Avenue Bridge in Shelby County, Tennessee. The curving ramp had a posted speed limit of 25 mph. The truck caused minimal damage to the 3-ft diameter bridge pier. The speed and weight of the vehicle are not known. The propane tanker exploded during the accident and caused severe damage to the bridge.

ACCIDENT #14 – IH-580/IH-880 COLLAPSE BY TANKER TRUCK FIRE, OAKLAND, CALIFORNIA

On April 29, 2007, a tanker-truck fire on the IH-580 overpass in Oakland, California, caused severe damage to a bridge which resulted in collapse of the bridge due to the intense heat from the fire.

ACCIDENT #15 – EXIT 111 BRIDGE OVER IH-24, MANCHESTER, TENNESSEE

On March 17, 2008, a truck-tractor-trailer loaded with pies impacted a large bridge pier on the Exit 111 bridge over IH-24 in Manchester, Tennessee. Damage to the pier was minor. The speed and the weight of the truck are not known.

ACCIDENT #16 – MURPHY HOLLOW ROAD OVER IH-24, MARION COUNTY, TENNESSEE

In 1989, a westbound truck with a box-type trailer impacted a 2-pier bent in the median of IH-24 in Marion County, Tennessee. The weight of the truck and trailer along with the impact speed are not known. The collision with the pier caused failure in the 24-inch square pier. Longitudinal reinforcement in the pier consisted of eight #10 bars equally spaced. Transverse reinforcement consisted of #4 closed stirrups spaced at 12 inches on centers. The bridge did not collapse as a result of the impact.

ACCIDENT #17 – IH-90 BRIDGE, #53812, MINNESOTA

On June 3, 2003, a large single-unit truck impacted a bridge pier located along IH-90 in near Worthington, Minnesota. The collision with the pier caused failure in the 32-inch diameter pier. The bridge did not collapse as a result of impact. Reinforcement in the pier consisted of nine #9 longitudinal bars equally spaced. Transverse reinforcement in the column consisted of #4 spiral stirrup reinforcement with a 6-inch pitch. A photo of the accident is shown as Figure 2.10.

Figure 2.10. Truck Accident – IH-90 Bridge, #53812, Minnesota.

ACCIDENT #18 – FM 1401 BRIDGE OVER IH-30, MOUNT PLEASANT, TEXAS

On May 29, 2008, a truck-tractor-trailer loaded with car parts crashed into a bridge pier on IH-30 near Mount Pleasant, Texas. This bridge is located on FM 1401 and carries traffic over IH-30. The vehicle impacted the westernmost 30-inch diameter pier of the 3-pier bent located on the shoulder of the eastbound lanes of IH-30. The collision with the pier caused failure in the 30-inch diameter pier. The bridge did not collapse as a result of impact. The collision killed the driver. State Trooper Daniel Crooks with Texas DPS was interviewed. State Trooper Crooks was present on the scene immediately after the accident. State Trooper Crooks indicated that the vehicle was traveling at a high rate of speed. The approximate weight of the vehicle was 80,000 lb when it struck the pier. The column impacted was a 30-inch diameter pier with eight #9 size rebars in the longitudinal direction of the pier. Transverse reinforcement in the pier consisted of #3 spiral stirrup reinforcement with a 6-inch pitch. Figure 2.11 shows a photo of the accident.

Figure 2.11. Truck Accident – FM 1401 Bridge over IH-30, Mount Pleasant, Texas.

ACCIDENT #19 – MILE POST 519 BRIDGE OVER IH-20, CANTON, TEXAS

On August 18, 2008, a truck-tractor-trailer, unloaded, crashed into a bridge pier on IH-20 near Canton, Texas. This bridge is located on Turner-Hayden Road and carries traffic over IH-20. The vehicle impacted the westernmost 30-inch diameter pier of the 2-pier bent located on

the shoulder of the eastbound lanes of IH-20. The collision with the pier caused failure in the 30-inch diameter pier. The bridge did not collapse as a result of impact. State Trooper Odie Phillips with Texas DPS was interviewed at the accident scene. State Trooper Phillips was present on the scene immediately after the accident. State Trooper Phillips indicated that the vehicle was traveling at a high rate of speed. The weight of the vehicle was not known. The column impacted was a 30-inch diameter pier with eight #9 size rebars in the longitudinal direction of the pier. Transverse reinforcement in the pier consisted of #3 spiral stirrup reinforcement with a 6-inch pitch. Figure 2.12 shows a photo of the accident.

Figure 2.12. Truck Accident – Mile Post 519 Bridge over IH-20, Canton, Texas.

CONCLUSIONS

Accident data collected for this project involve large truck collisions with bridge piers. The impacting speed and the weight of the vehicle at the time of impact with the pier were not precisely known. In most cases, this information was approximated based on the information from the police reports and personal interviews with law enforcement officials. In nearly every accident case, the damage to the impacted bridge pier was catastrophic, resulting in reconstruction of the pier. In four of the 19 cases listed above, collapse of the bridge structure occurred as a result of the large truck collisions with the piers.

CHAPTER 3. STRENGTH OF BRIDGE PIERS

SUMMARY OF SHEAR CAPACITIES OF CIRCULAR PIERS FROM ACCIDENT INVESTIGATIONS

Bridge piers impacted by large trucks are typically subjected to large shear and bending forces. These forces can cause catastrophic structural failure in the piers. As part of this project, several accidents involving large trucks were investigated. In most of the cases investigated, structural failure in the bridge column occurred as a result of the impact. From the piers investigated for this project, a typical failure mechanism from a large truck collision is shown in Figure 3.1.

Figure 3.1. Typical Failure Mechanism in Bridge Pier from Large Truck Collision.

Typically, the truck collision force is relatively close to the ground surface as shown in Figure 3.1. Although a large bending force is applied to the pier, the high shear force from the truck collision exceeds the shear capacity of the pier, thus resulting in a shear failure mechanism in the pier. Shear capacity analyses were performed on the piers investigated for this project to determine the shear resistance of the piers.

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Structural analyses were performed on several piers impacted from the accident investigations. Structural details for each specific pier were obtained from the state bridge engineer of the associated state where the accident occurred. The nominal shear strength of each pier was calculated in accordance with AASHTO LRFD Bridge Design Specifications, Fourth Edition, 2007. These capacities are based on two failure planes resisting the force. These two shear failure planes radiate at approximately 45 degrees from the applied impact force as shown in Figure 3.2.
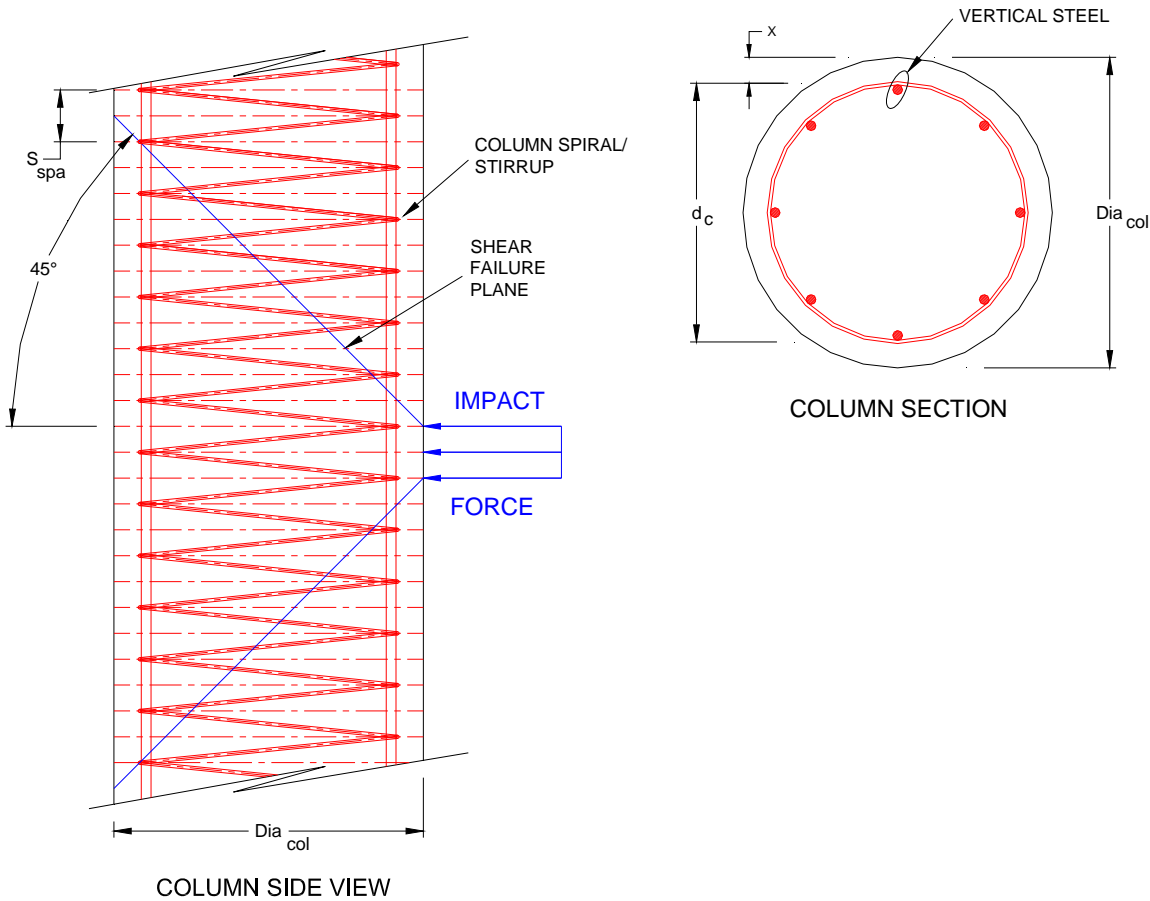
Figure 3.2. Observed Failure Mechanism from Impact Force on Bridge Pier.

The failure mechanism shown in Figure 3.2 was observed in many piers which were impacted by large trucks. In all cases, the design compressive strength of the concrete as provided by the structural drawings was used. In addition, the nominal shear strength of each pier was calculated using a higher estimated strength that could exist due to years of concrete age. Please refer to Table 3.1 for a summary of the calculated shear capacities for the piers investigated for this project. Please refer to the calculations in Appendix A for additional information.

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Table 3.1. Shear Capacities of Circular Piers from Accident Investigations.

Accident No.	Pier Diameter (inches)	Design (Estimated) Concrete Compressive Strength (psi)	Shear Reinforment Size	Vertical Reinforcment	Calculated Shear Capicty (kips)*
1	30	3050 (4000)	#2 - 6" Pitch	8 - #9's Eq. Spa.	80 (88)
2	30	3050 (4000)	#2 - 6" Pitch	8 - #9's Eq. Spa.	80 (88)
3	30	3050 (4000)	#2 - 6" Pitch	8 - #9's Eq. Spa.	80 (88)
4	30	3050 (4000)	#2 - 6" Pitch	8 - #9's Eq. Spa.	80 (88)
5	30	3050 (4000)	#2 - 6" Pitch	8 - #9's Eq. Spa.	80 (88)
7	30	3050 (4000)	#2 - 6" Pitch	8 - #9's Eq. Spa.	80 (88)
8	30	3050 (4000)	#3 - 6" Pitch	8 - #9's Eq. Spa.	140 (148)
10	24	3050 (4000)	#2 - 6" Pitch	8 - #7's Eq. Spa.	56 (62)
17	32	4300 (5500)	#4 - 6" Pitch	9 - #9's Eq. Spa.	310 (330)
18	30	3600 (4000)	#3 - 6" Pitch	8 - #9's Eq. Spa.	215 (222)
19	30	3050 (4000)	#2 - 6" Pitch	8 - #9's Eq. Spa.	80 (88)

* - Design (Estimated) Concrete Compressive Strength

Shear capacities were also calculated for various pier sizes. Please refer to Table 3.2 for a summary of the calculated shear capacities of various pier sizes investigated for this project.

Table 3.2. Shear Capacities for Different Pier Diameters.

Pier Diameter (inches)	Design Concrete Compressive Strength (psi)	Shear Reinforcement Size**	Calculated Shear Capacity (kips)
24	3600	#3 - 6 " Pitch	148
30	3600	#3 - 6 " Pitch	215
36	3600	#3 - 6 " Pitch	292
42	3600	#4 - 6 " Pitch	474
48	3600	#4 - 6 " Pitch	589
54	3600	#4 - 6 " Pitch	714
60	3600	#4 - 6 " Pitch	851
66	3600	#4 - 6 " Pitch	1000
72	3600	#5 - 6 " Pitch	1366

** = 60 KSI Material

CONCLUSIONS

The calculated strength capacities listed in Tables 3.1 and 3.2 are the nominal (ultimate) unfactored shear strengths of the piers considering the compressive strengths of the concrete, transverse (spiral) reinforcements, and two shear planes radiating at 45-degree angles from the direction of impact. In many of the actual piers investigated for this project, which were involved in large truck collisions, the mode of failure in the piers were similar to the failure mechanism previously described. In nearly all the piers investigated and analyzed for this project, the calculated nominal shear capacity was less than 400 kips.

CHAPTER 4. SIMULATION ANALYSIS OF VEHICULAR IMPACTS ON BRIDGE PIERS

BACKGROUND

LRFD Bridge Design Specifications require that an equivalent static force of 400 kip be used for the design of piers and abutments to withstand vehicle collisions. As data and information have become available, it is desired to reevaluate the 400-kip design force requirement. Recent advances in computer hardware and finite element methodologies allow researchers to investigate vehicle impact problems with more fidelity and to obtain the overall dynamic load-time history of the impact event. Availability of public domain models of the heavy trucks of interest are being developed albeit not encompassing all desired features of a heavy truck model.

OBJECTIVE

The objective of this portion of the research effort is to perform finite element analyses of heavy vehicle impacts on rigid piers and to quantify the force imparted during the impact.

MODELING AND SIMULATION: OVERVIEW

Modeling Methodology

Each case (simulation run) consisted of the pier and heavy truck vehicle model. The pier was modeled using rigid material model with fixed boundary conditions (top and bottom) so the maximum possible impact force can be calculated. A contact was defined between the truck and the pier to define the impact interface. The heavy truck models were comprised of mostly elastic-plastic material representation. The cargo (modeled vas a single ballast) was assumed to have either one of the two stiffness properties ("rigid" or "deformable").

TTI researchers updated a beta version of the tractor-trailer model developed by the National Crash Analysis Center (NCAC) to incorporate realistic trailer mode, as well as enhanced the tractor model to increase its fidelity of simulating 90-degree impacts with piers. The research team morphed the tractor model into a single unit truck (SUT) representing a 65,000 lb dump truck vehicle. Details of the development of both models are shown in Appendix B of this report.

The rigid pier top and lower ends constraints were instrumented to measure force due to the impact event.

Heavy Truck Vehicle Models

Two heavy truck models were used:

1) Single Unit Truck (SUT) (65,000 lb) with
    a. Rigid Cargo
    b. Deformable Cargo
2) Tractor-Trailer (80,000 lb) with
    a. Rigid Cargo
    b. Deformable Cargo

Simulation Methodology

The first stage of the numerical analyses was a parametric study to quantify the sensitivity of the impact force to the diameter of the pier. This study indicated that there was not significant effect of the pier diameter on the magnitude of the impact force as shown in the first three simulation cases in Table 4.1. Hence a pier of diameter 36 inches was selected for all further numerical analyses. The SUT and tractor-trailer finite element models were used to simulate collisions with the rigid 36-inch diameter pier.

The simulations indicated that the collision event consists of basically two major impacts, the engine block impact with the pier and the rigid (or deformable) cargo (ballast) impact with the pier (through the crushed cab structure). Different impact velocities were simulated as shown in the Table 4.1.

Table 4.1. Simulation Matrix and Summary for SUT.

	Pier Diameter	Vehicle (Weight)	Cargo/Ballast	Impact Speed	Peak-50 ms Avg. Force (kip)	
					Engine Block	Ballast
Matrix I	24 inches	SUT (65 K-lb)	Rigid	50	560	2490
	36 inches	SUT (65 K-lb)	Rigid	50	570	2430
	48 inches	SUT (65 K-lb)	Rigid	50	560	2160
Ballast Test Matrix	36 inches	SUT (65 K-lb)	Rigid	40	500	1470
	36 inches	SUT (65 K-lb)	Rigid	50	570	2430
	36 inches	SUT (19 K-lb)	Rigid	50	550	None
Matrix II	36 inches	SUT (65 K-lb)	Rigid	40	500	1470
	36 inches	SUT (65 K-lb)	Rigid	50	570	2430
	36 inches	SUT (65 K-lb)	Rigid	60	N.A.	N.A.
Matrix III	36 inches	SUT (65 K-lb)	Deformable	40	480	480
	36 inches	SUT (65 K-lb)	Deformable	50	540	800
	36 inches	SUT (65 K-lb)	Deformable	60	580	1000

As the impact velocity increased in Matrix II, the numerical simulation became unstable in the 60 mph case since two rigid bodies (pier and ballast) are colliding, which gives a theoretical infinite impact force. Therefore, the research team opted for incorporating elastic-plastic material behavior for the ballast (and the SUT bed structure). The development of a deformable container and ballast lead to a more stable analysis, as well as accounting for movement and compliance of the ballast. SUTs can carry a variety of cargo ranging from very stiff, rigidly attached to deformable, loosely attached.

SIMULATION: SUT IMPACT ANALYSIS

Matrix I

The objective of Matrix I was to determine the effect of pier diameter on the peak impact force from an SUT. Three commonly used pier diameters, 24, 36, and 48 inches, were used in the simulations. A collision by a 65,000 lb SUT with rigid ballast and traveling at 50 mph was simulated for each pier size. Results of the simulations are shown in Figures 4.1 through 4.9.

Dynamic forces, averaged over 50-millisecond (ms) time intervals for the duration of the collisions, are shown in Figure 4.1. Results show that the effect of pier diameter on collision force is not significant.
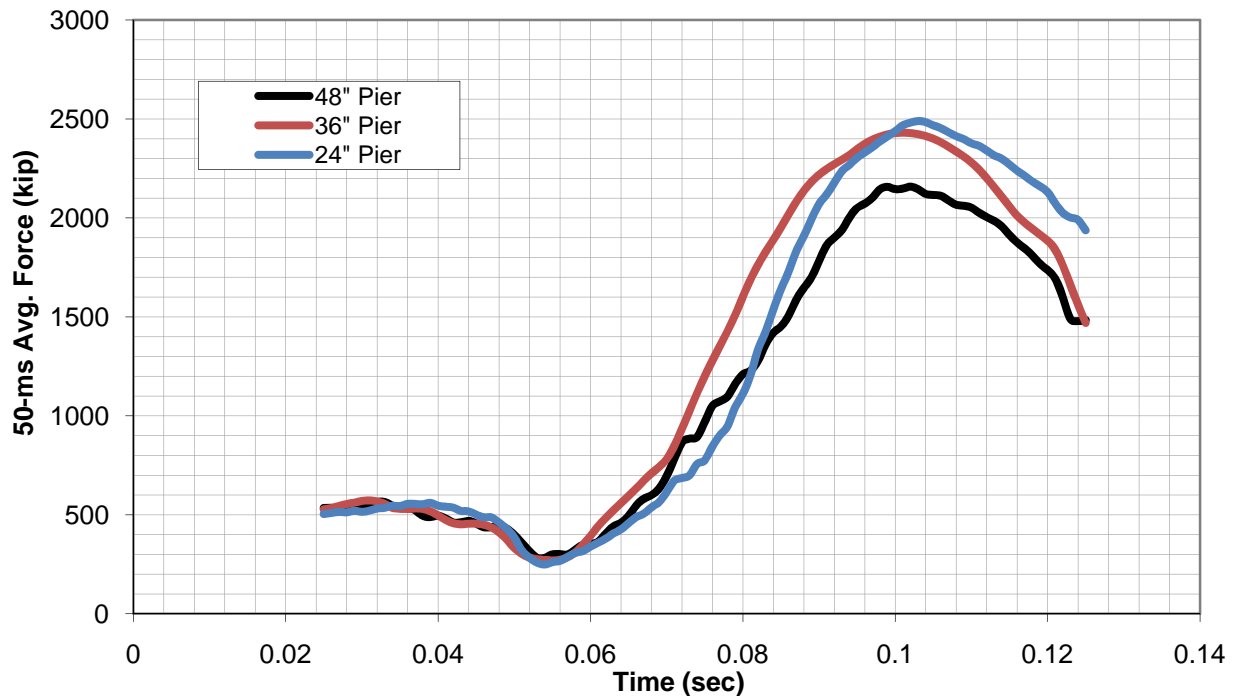
Figure 4.1. Matrix I, SUT, Rigid Ballast, 50 mph, 50-ms Average Resultant Reactions.

Force versus displacement of the truck and displacement versus time are shown in Figures 4.2 and 4.3, respectively. Force values in Figure 4.2 are unfiltered raw output from the simulations computed at 1000 samples per second (sec). Displacements are for the original center-of-gravity of the undeformed truck. These results also show the effect of pier diameter to be insignificant. Similarities in deformations of the truck for different pier diameters are

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illustrated in Figures 4.4 through 4.9. As indicated earlier, a 36-inch diameter pier was selected for use in subsequent parametric simulations.
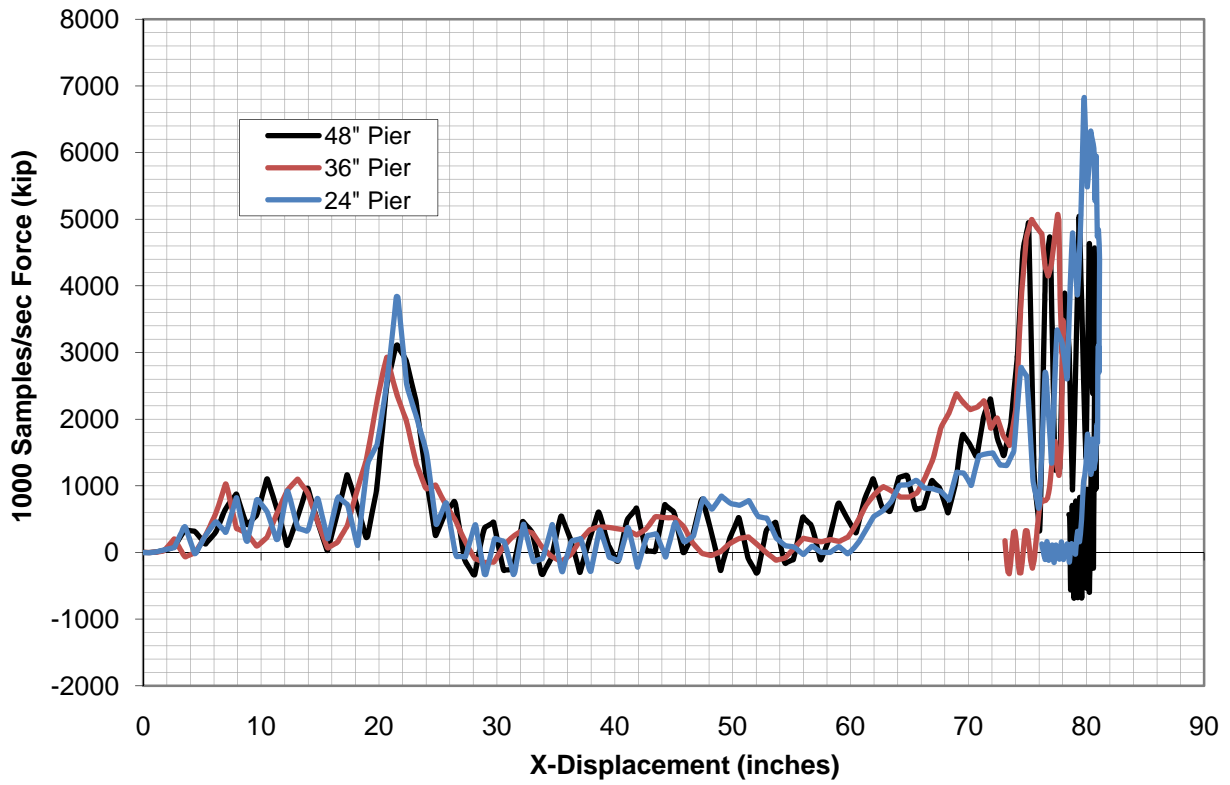
Figure 4.2. Matrix I, SUT, Rigid Ballast, 50 mph, Force versus X-Displacement.
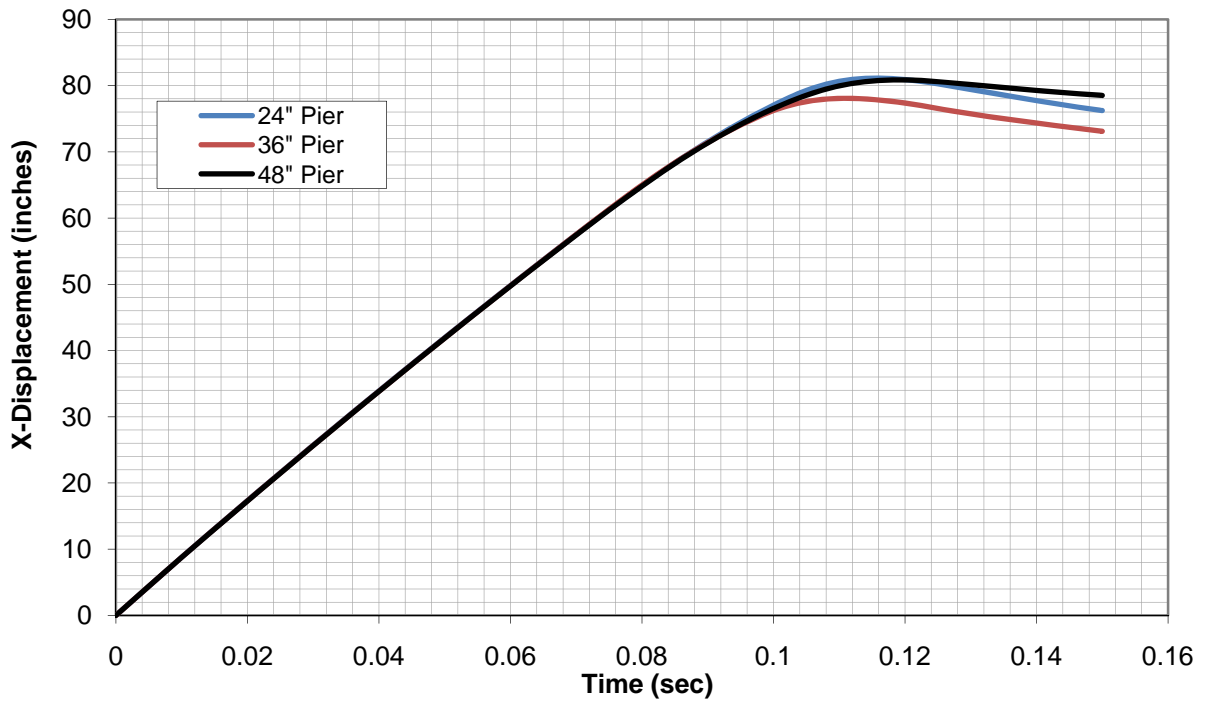
Figure 4.3. Matrix I, SUT, Rigid Ballast, 50 mph, X-Displacement versus Time.

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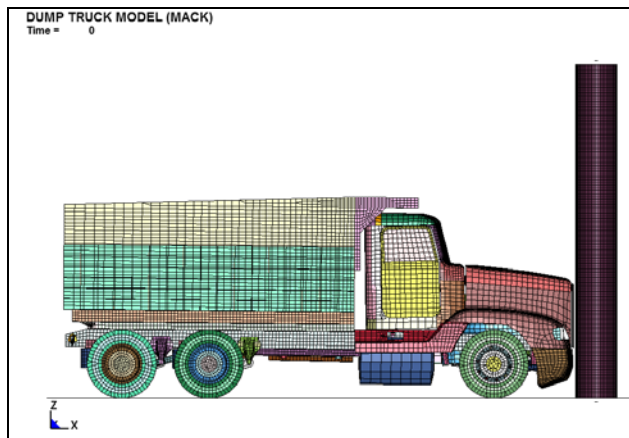
Figure 4.4. Matrix I, SUT, Rigid Ballast, 50 mph, 24-inch Pier Before.
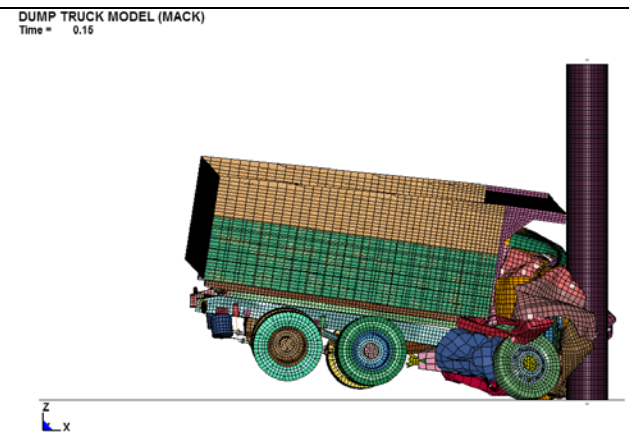
Figure 4.5. Matrix I, SUT, Rigid Ballast, 50 mph, 24-inch Pier After.
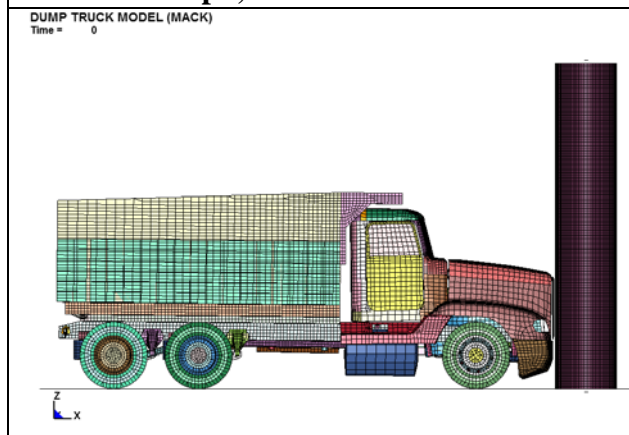
Figure 4.6. Matrix I, SUT, Rigid Ballast, 50 mph, 36-inch Pier Before.
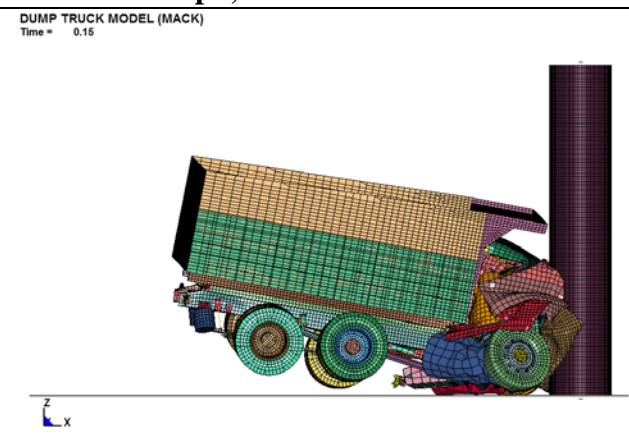
Figure 4.7. Matrix I, SUT, Rigid Ballast, 50 mph, 36-inch Pier After.
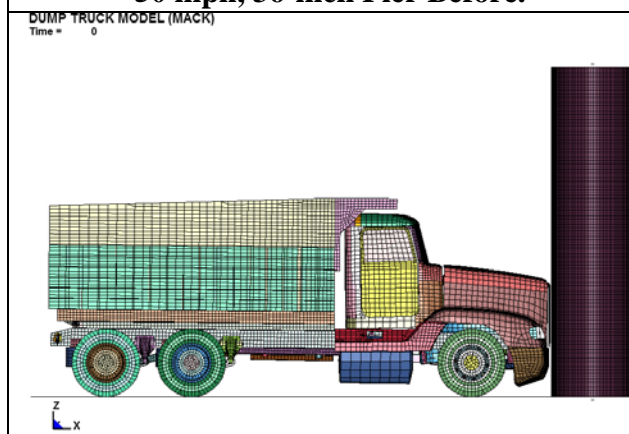
Figure 4.8. Matrix I, SUT, Rigid Ballast, 50 mph, 48-inch Pier Before.
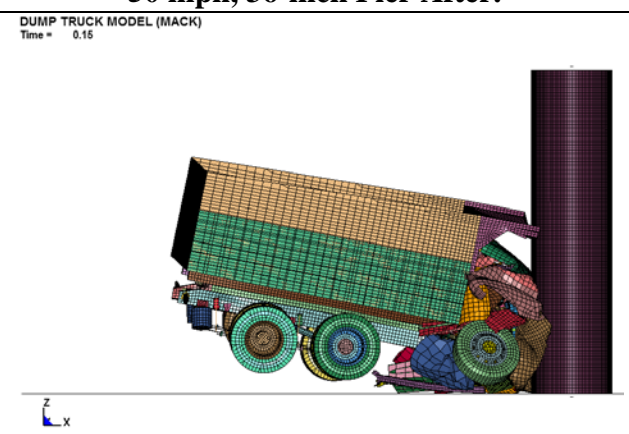
Figure 4.9. Matrix I, SUT, Rigid Ballast, 50 mph, 48-inch Pier After.

SUT Ballast Interaction Matrix

The ballast test matrix simulations were performed to quantify the effect of presence of ballast on impact force. This matrix also yielded information about the effect of impact speed on collision force.

Figures 4.10 and 4.11 show the relationship between the ballast and impact force on the pier. A direct relationship between the ballast mass and force on the pier can be concluded. With an increase in mass of the ballast, an increase in peak force on the pier occurs. A correlation between the velocities can also be made. A decrease in velocity leads to a decrease in the peak pier impact force.
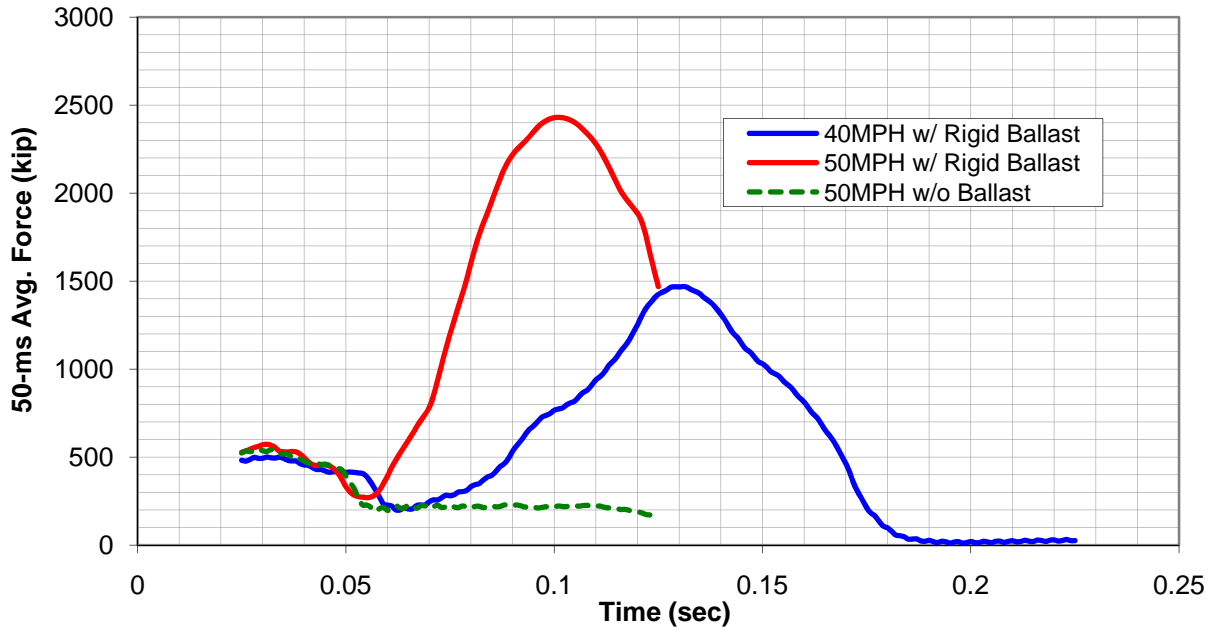
Figure 4.10. Ballast Interaction Matrix, SUT, 36-inch Pier, 50-ms Average Resultant Reactions.
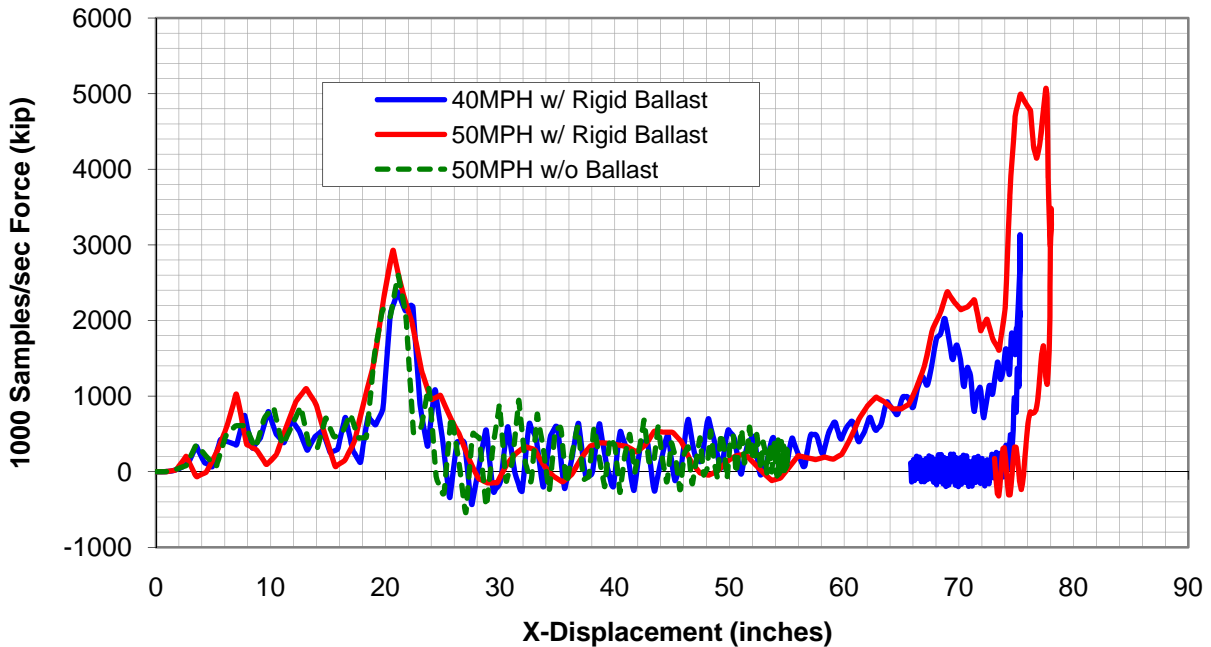
Figure 4.11. Ballast Interaction Matrix, SUT, 36-inch Pier, Force versus X-Displacement.

Figure 4.12 describes the deformation related to the mass of the ballast and velocities. An increase in mass and velocity leads to higher deformations in a shorter time frame. The non-ballasted vehicle deformed less than the ballasted vehicle. Deformations of the truck are illustrated in Figures 4.13 through 4.18.
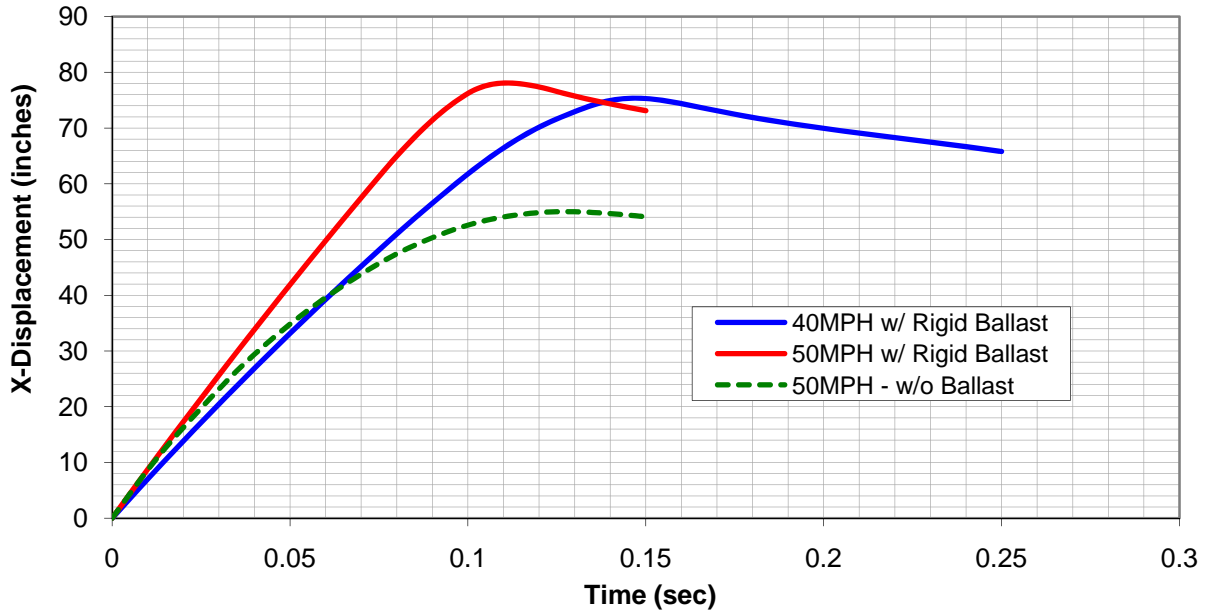
Figure 4.12. Ballast Analysis X-Displacement versus Time.

Peak Force Analysis

The peak force analysis was performed to determine the factors influencing and/or causing the peak forces on the reaction forces shown in Figure 4.19. Force values are from the simulation for a SUT with rigid ballast impacting at 50 mph. The charts are unfiltered raw data computed at 1000 samples per sec.

Figure 4.20 was used to determine the displacement at which each peak in the x-displacement versus force curve occurs. These data were then cross referenced using Figure 4.21 to resolve the time in which each peak occurs. From this, the components of the SUT causing the peak forces during the impact were determined. The results are depicted in Figure 4.22 through Figure 4.25.
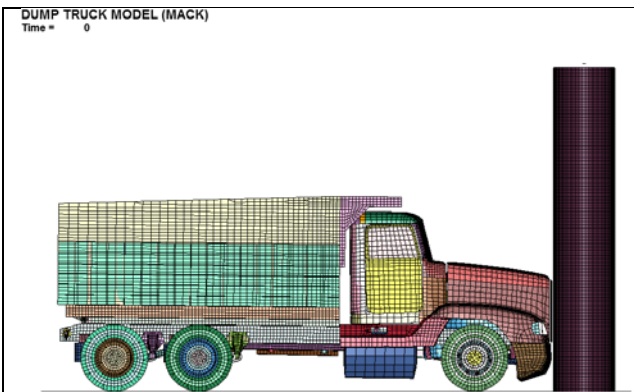
Figure 4.13. Ballast Interaction Matrix, SUT, 40 mph, Rigid Ballast (65,000-lb) Before.
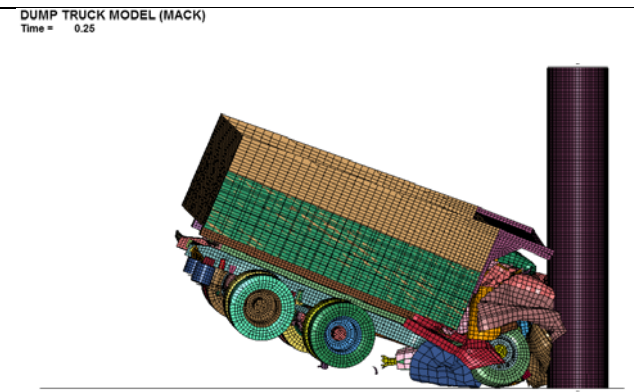
Figure 4.14. Ballast Interaction Matrix, SUT, 40 mph, Rigid Ballast (65,000-lb) After.
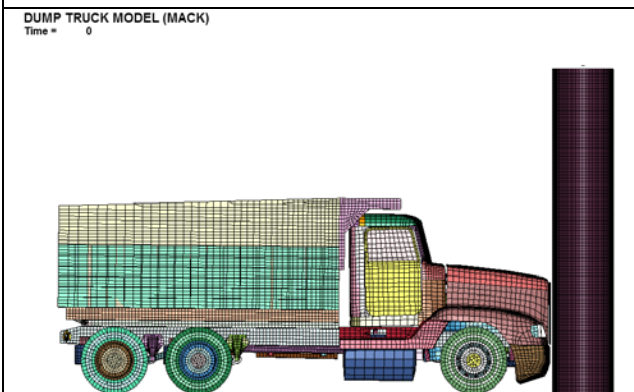
Figure 4.15. Ballast Interaction Matrix, SUT, 50 mph, Rigid Ballast (65,000-lb) Before.
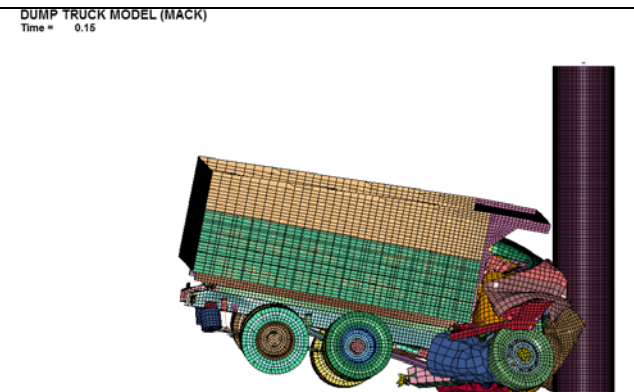
Figure 4.16. Ballast Interaction Matrix, SUT, 50 mph, Rigid Ballast (65,000-lb) After.
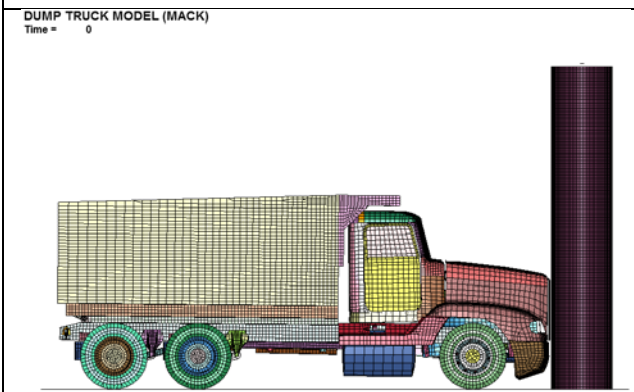
Figure 4.17. Ballast Interaction Matrix, SUT, 50 mph, No Ballast (19,000-lb) Before.
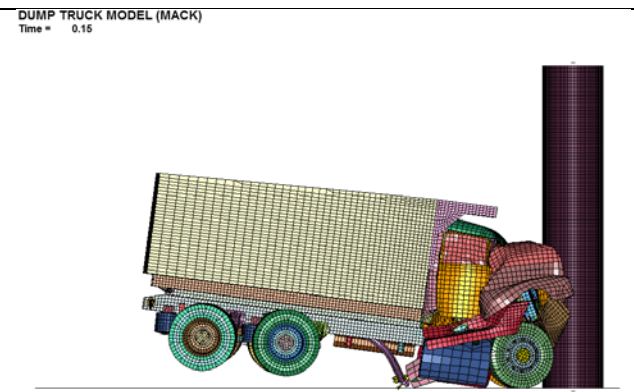
Figure 4.18. Ballast Interaction Matrix, SUT, 50 mph, No Ballast (19,000-lb) After.
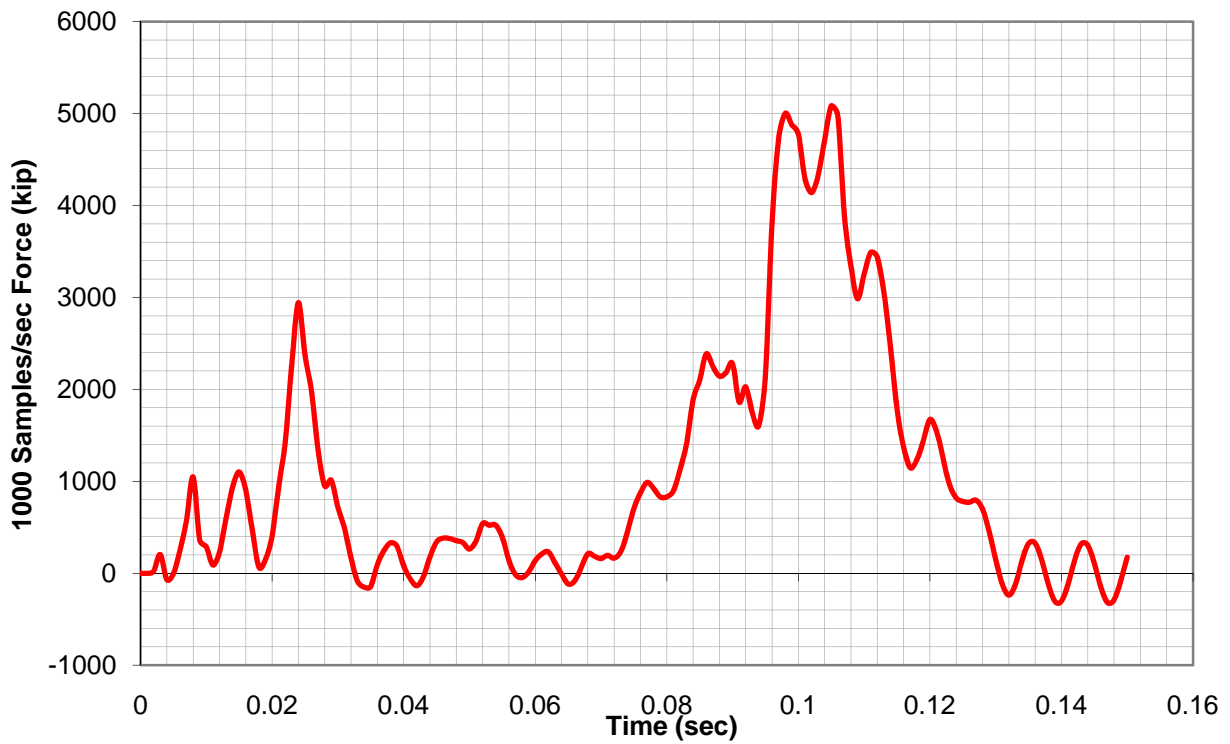
Figure 4.19. Matrix I, SUT, Rigid Ballast, 36-inch Pier, 50 mph, Resultant Reaction Force.
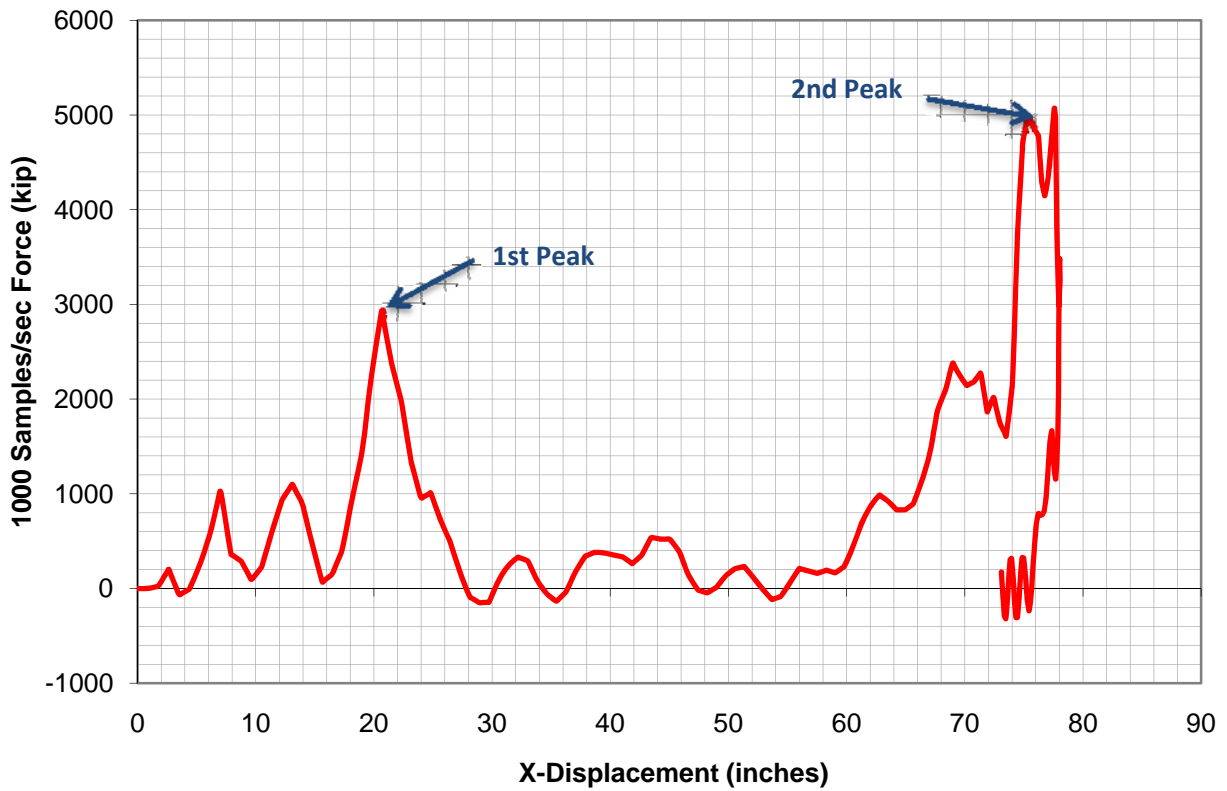
Figure 4.20. Matrix I, SUT, Rigid Ballast, 36-inch Pier, 50 mph, Force versus X-Displacement.
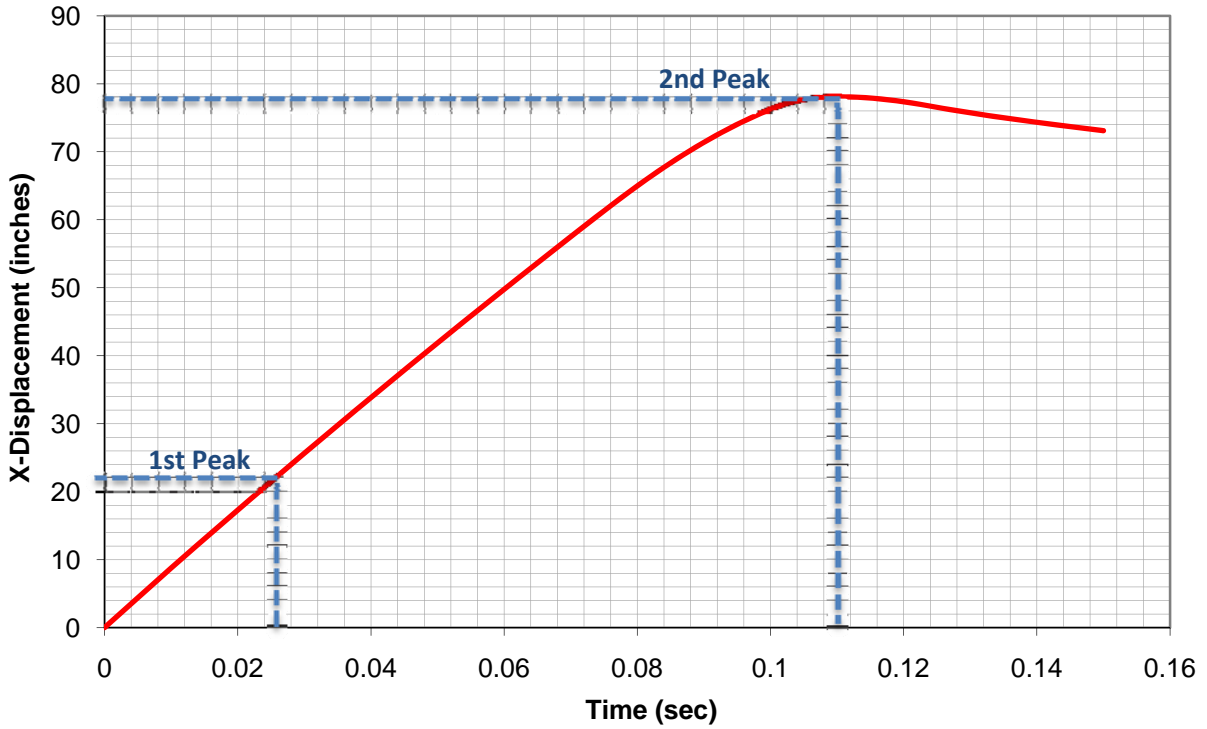
Figure 4.21. Matrix I, SUT, Rigid Ballast, 36-inch Pier, 50 mph, X-Displacement versus Time.
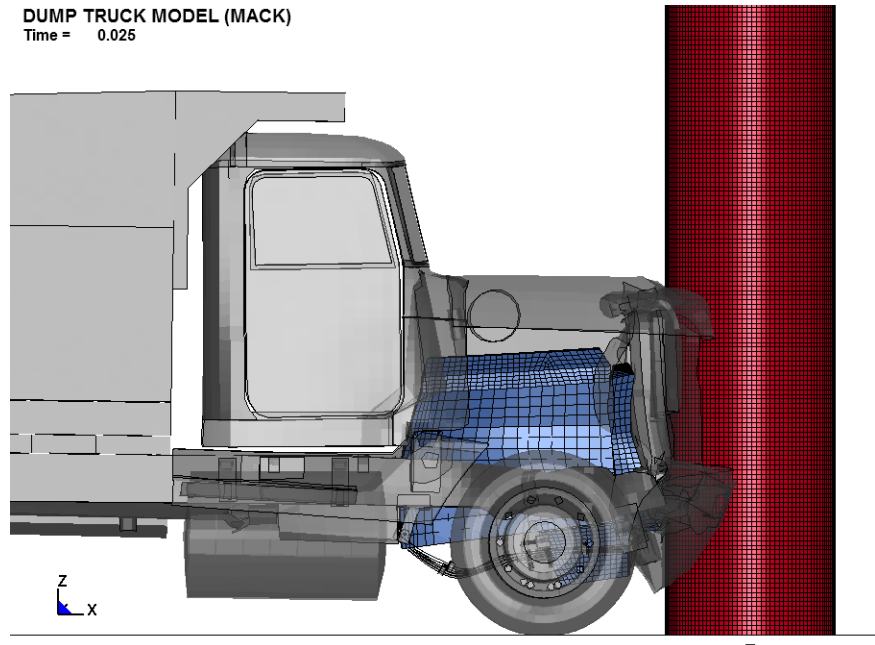
Figure 4.22. SUT Deformation, Engine-Pier Impact Right View.

Figure 4.23. SUT Deformation, Engine-Pier Impact Top View.

Figure 4.24. SUT Deformation, Ballast-Engine Impact Right View.

DUMP TRUCK MODEL (MACK)
Time =   0.11

Y
X
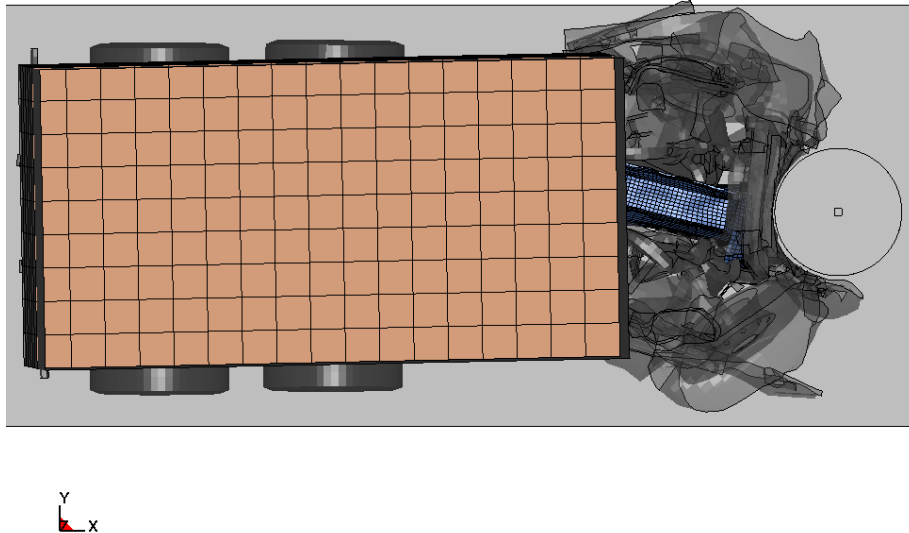
Figure 4.25. SUT Deformation, Ballast-Engine Impact Top View.

The initial peak force on the pier during impact is a result of the engine block impacting the pier. This can be seen in both Figure 4.22 and Figure 4.23 at 0.025 seconds**.** The second and largest peak in the impact force plot is the result of the ballast striking the engine block at 0.11 seconds. As the engine block has effectively no crush, the force is transmitted into the pier. This process can be seen in Figure 4.24 and Figure 4.25.

Matrix II

The objective of Matrix II was to analyze the effects of velocity and its corresponding force an SUT imposes during impact into a pier. Velocities of 40, 50, and 60 mph were evaluated.

Rigid ballast with a total vehicle weight of 65,000 lb was intended for each simulation in Matrix II. However, the rigid ballast at 60 mph yielded unreliable simulation results. For the 60 mph simulation, the deformable bed and ballasted SUT model were used. The weight of the SUT remained constant at 65,000 lb.

Figure 4.26 depicts force versus time as the SUT impacts the pier. Increases in velocity of the SUT result in increases of the peak force on the pier. This peak force is greatly reduced as seen in the 60 mph run with the deformable ballast as the force is less than both the 40 and 50 mph with rigid ballast. As discussed previously, the initial peak in Figure 4.27 occurs as the engine impacts the pier, and the second peak is impact forces from the ballast. It is worth noting, the deformable ballast yields lower forces at higher velocities of 60 mph than does the rigid ballast at 40 and 50 mph.

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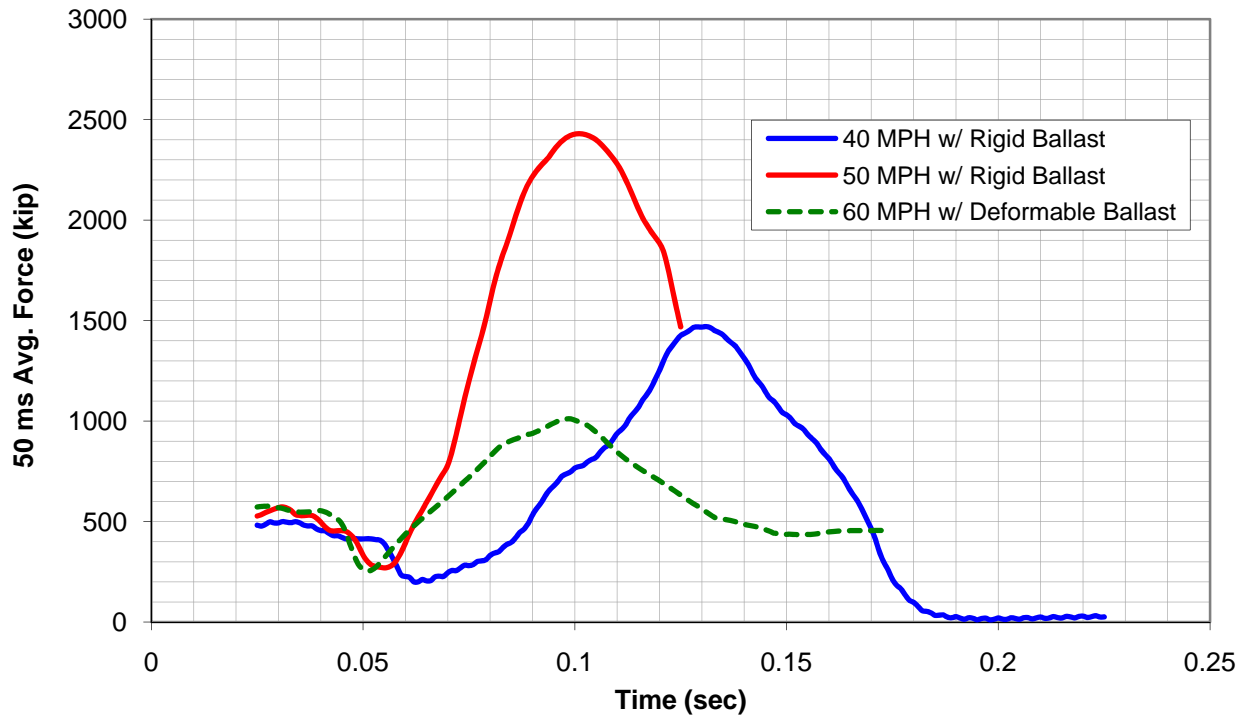
Figure 4.26. Matrix II, SUT, 36-inch Pier, 50-ms Average Resultant Reactions.
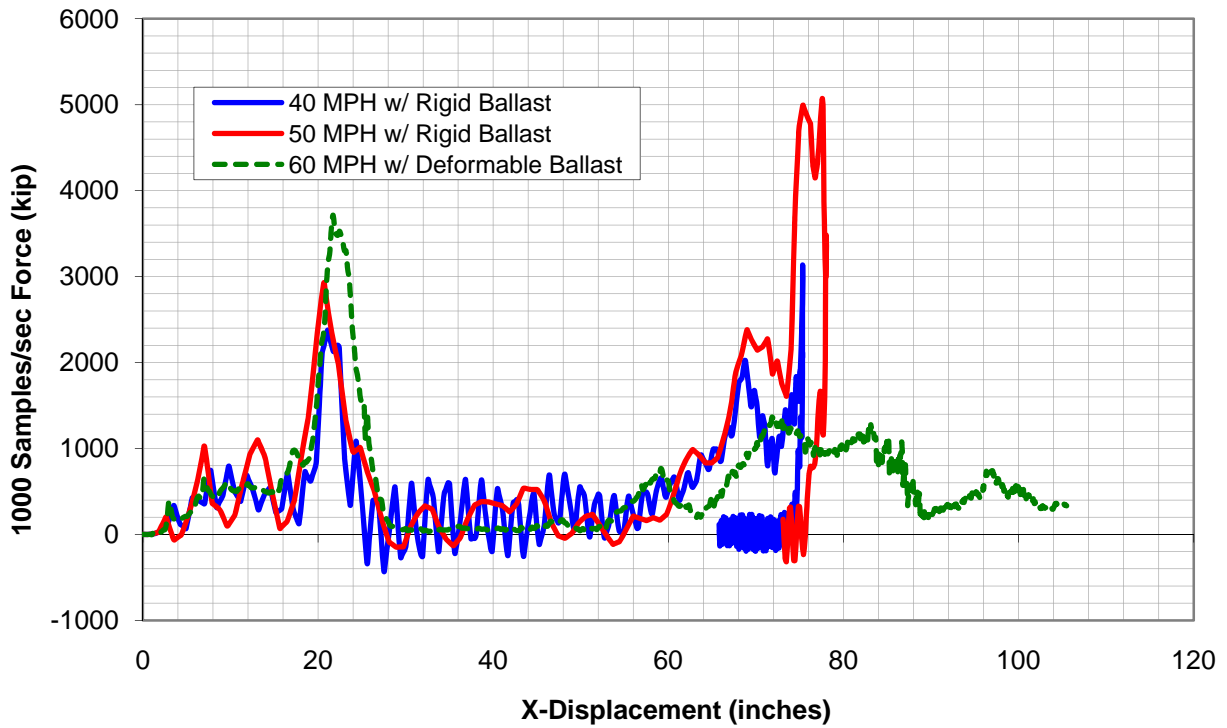
Figure 4.27. Matrix II, SUT, 36-inch Pier, Force versus X-Displacement.

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Figure 4.28 shows the relationships between impact velocity and the crush of the vehicle. As the velocity of the truck increases, the crush of the truck is increased. The truck also crushes at a higher rate with increased velocities.
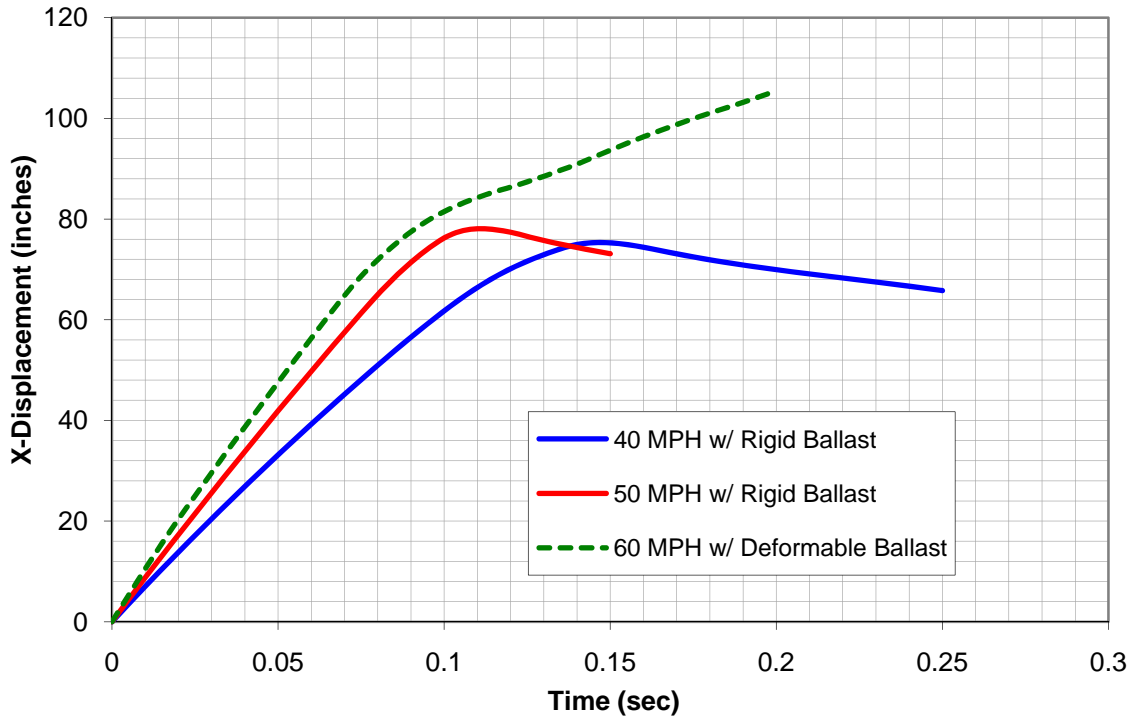
Figure 4.28. Matrix II, SUT, 36-inch Pier, X-Displacement versus Time.

Figures 4.29 and 4.30 show crush of the truck with rigid ballast impacting at 40 mph, while Figures 4.31 and 4.32 show crush for the same vehicle impacting at 50 mph. Figures 4.33 and 4.34 show crush of the SUT with deformable ballasting impacting at 60 mph.
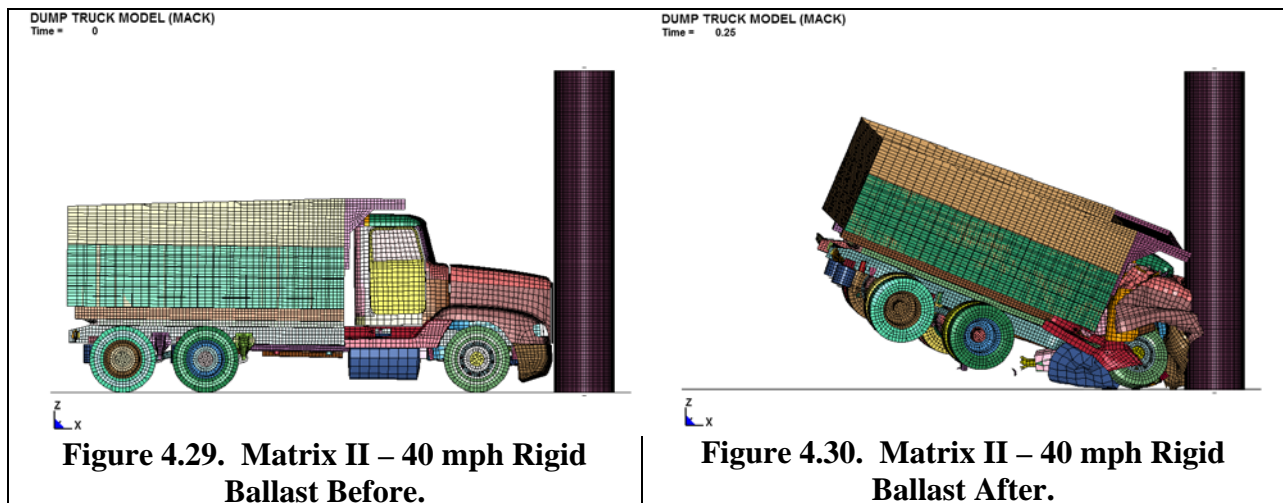
| **Figure 4.29. Matrix II – 40 mph Rigid Ballast Before.** | **Figure 4.30. Matrix II – 40 mph Rigid Ballast After.** |

Figure 4.31. Matrix II – 50 mph Rigid Ballast Before.

Figure 4.32. Matrix II – 50 mph Rigid Ballast After.

Figure 4.33. Matrix II – 60 mph Deformable Ballast Before.

Figure 4.34. Matrix II – 60 mph Deformable Ballast After.
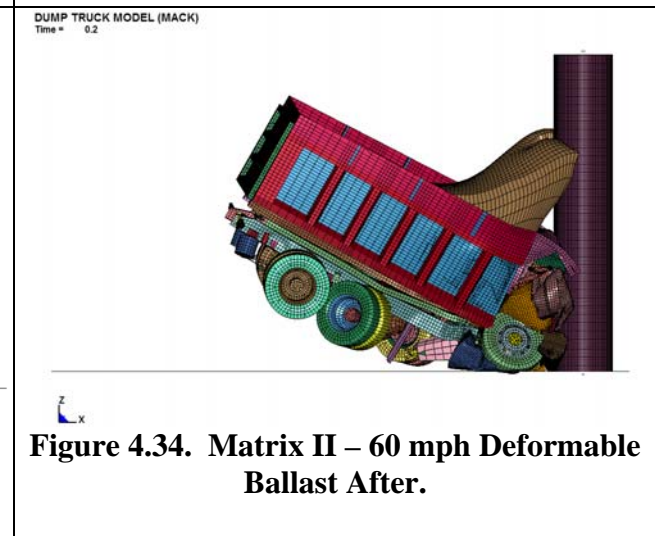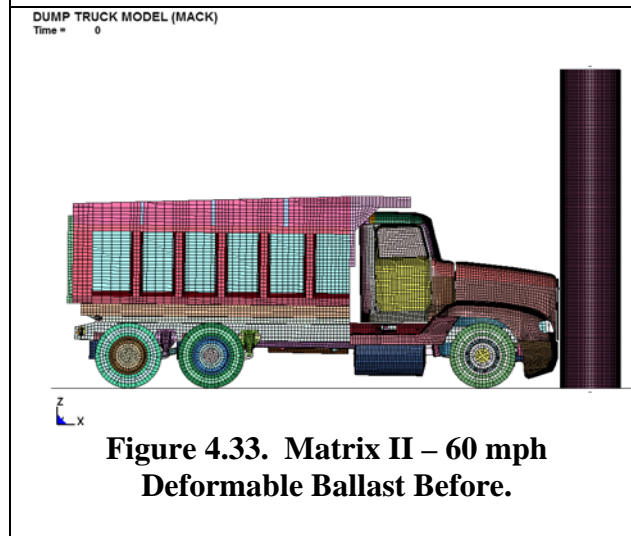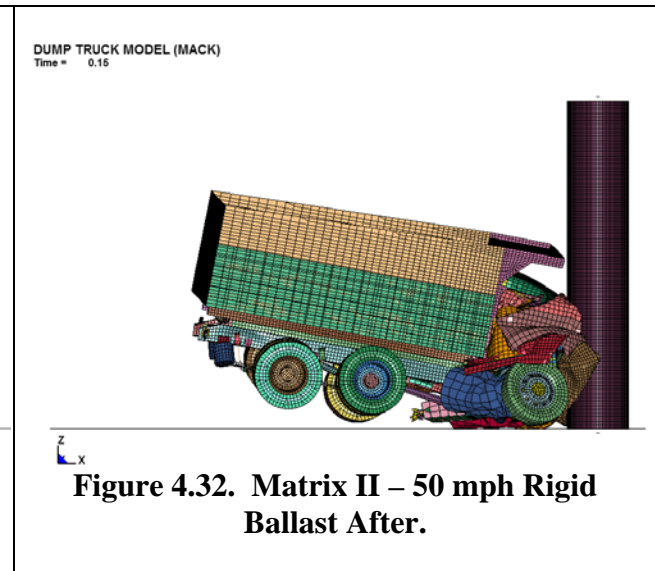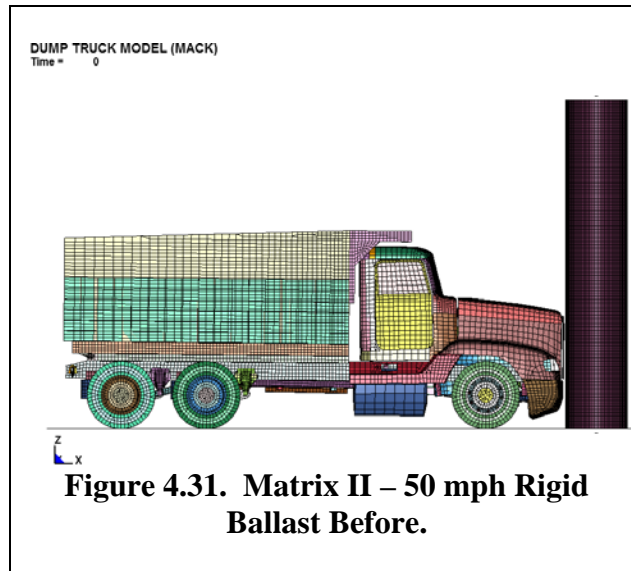
Matrix III

The objective of Matrix III was to analyze the effects of velocity and its corresponding force from an SUT with deformable ballast. A pier diameter of 36 inches was used for each simulation. Velocities of 40, 50, and 60 mph were evaluated. Figures 4.35 and 4.36 depict force versus time as the SUT impacts the pier at 40, 50, and 60 mph. Increases in velocity of the SUT result in increases of the peak force on the pier.
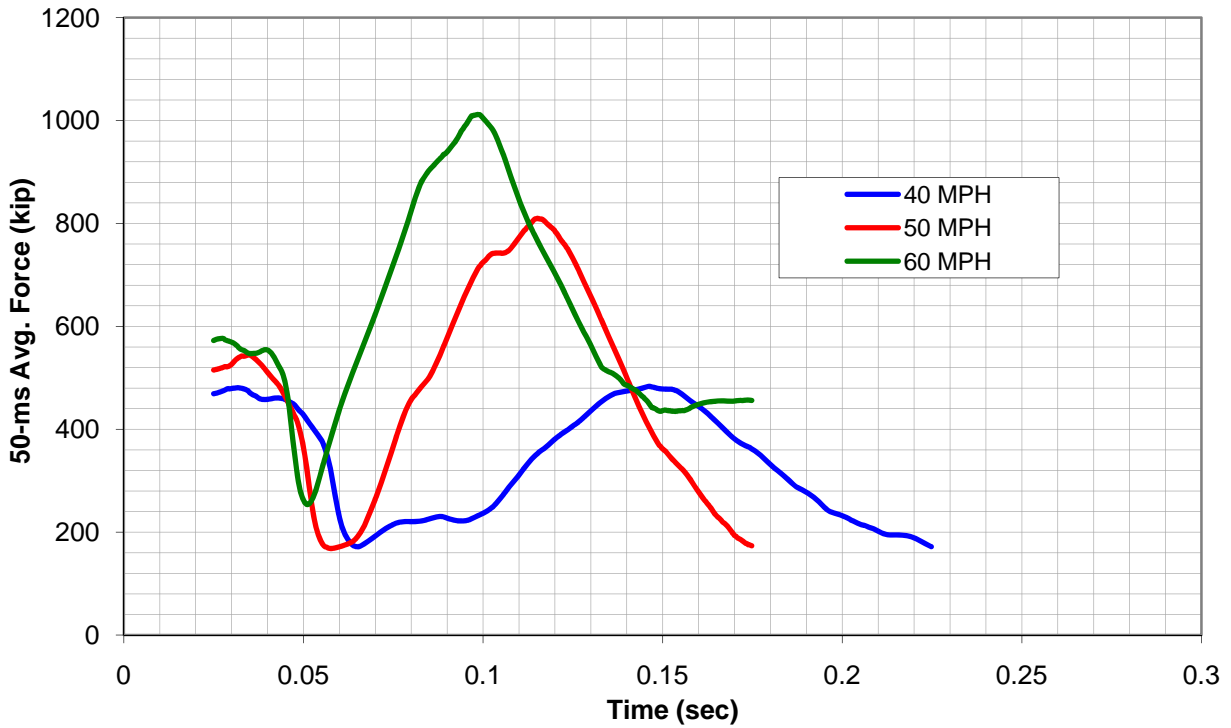
Figure 4.35. Matrix III, SUT, Deformable Ballast, 36-inch Pier, 50-ms Average Resultant Reaction.
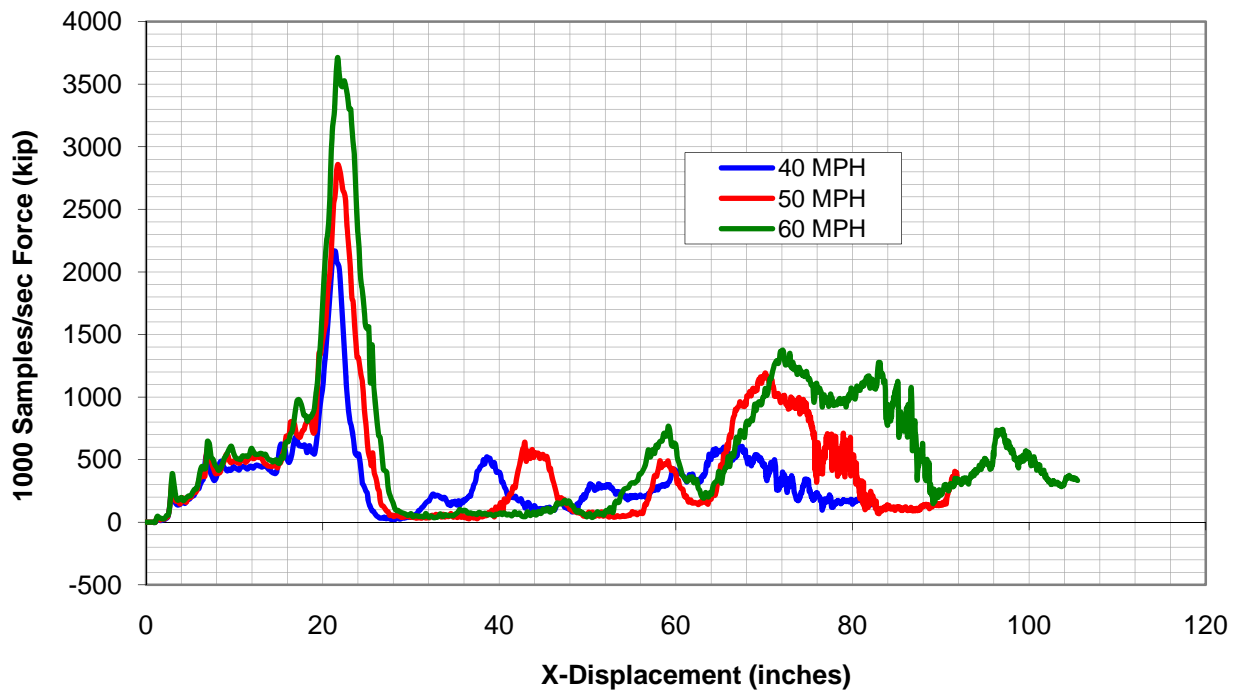
Figure 4.36. Matrix III, SUT, Deformable Ballast, 36-inch Pier, Force versus X-Displacement.

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Figures 4.37 through 4.43 show the relationships between impact velocity and the crush of the vehicle. As the velocity of the impact increases the crush of the SUT is increased. The SUT also crushes at a higher rate with increased velocities.
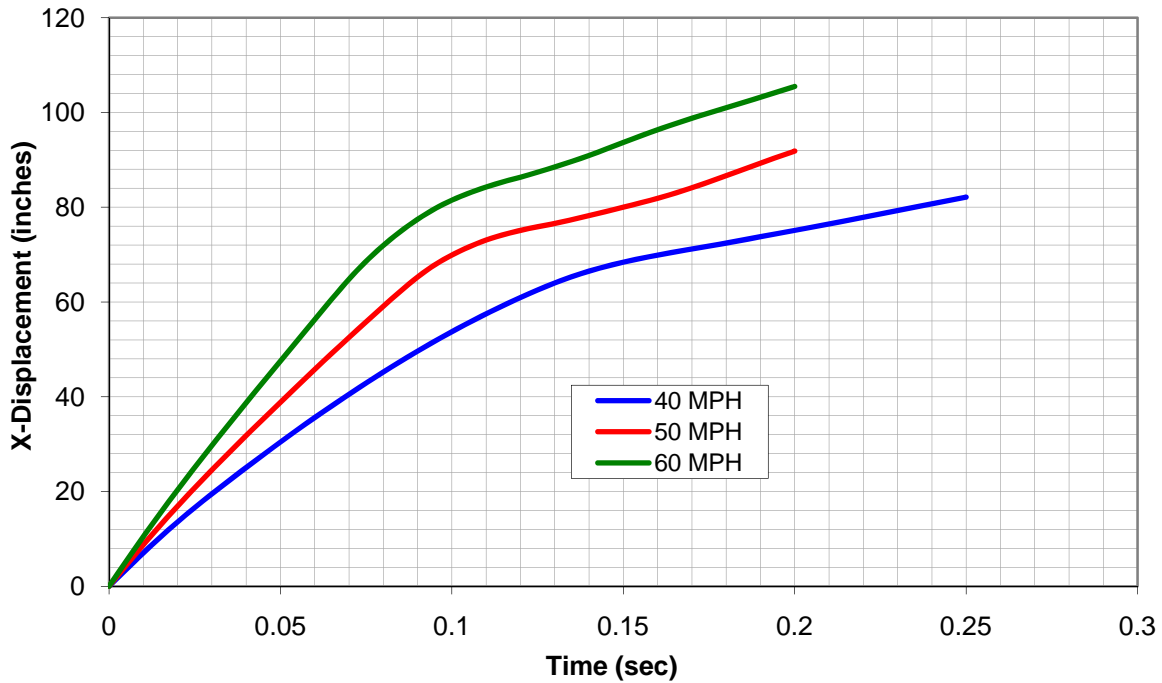
Figure 4.37. Matrix III, SUT, Deformable Ballast, 36-inch Pier, X-Displacement versus Time.
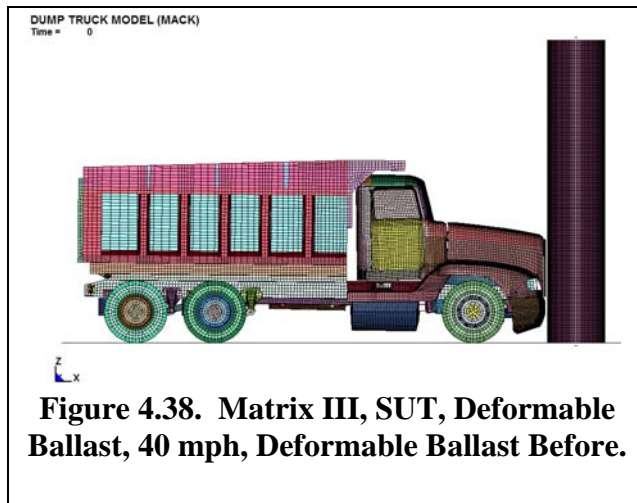
Figure 4.38. Matrix III, SUT, Deformable Ballast, 40 mph, Deformable Ballast Before.
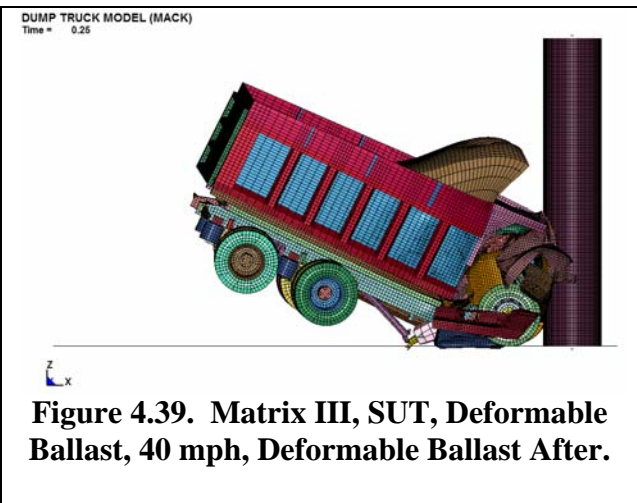
Figure 4.39. Matrix III, SUT, Deformable Ballast, 40 mph, Deformable Ballast After.
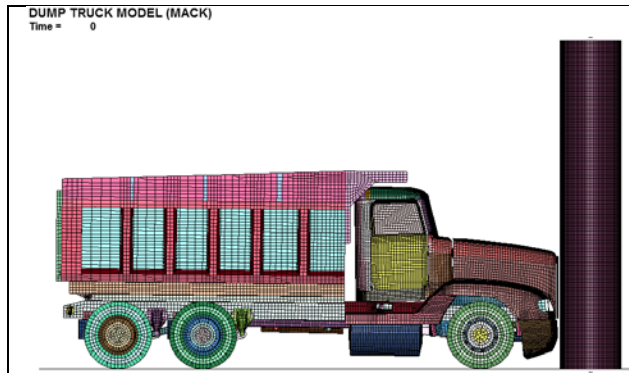
Figure 4.40. Matrix III, SUT, Deformable Ballast, 50 mph, Deformable Ballast Before.
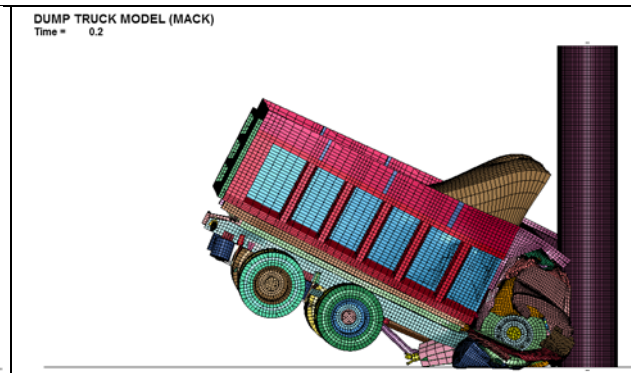
Figure 4.41. Matrix III, SUT, Deformable Ballast, 50 mph, Deformable Ballast After.
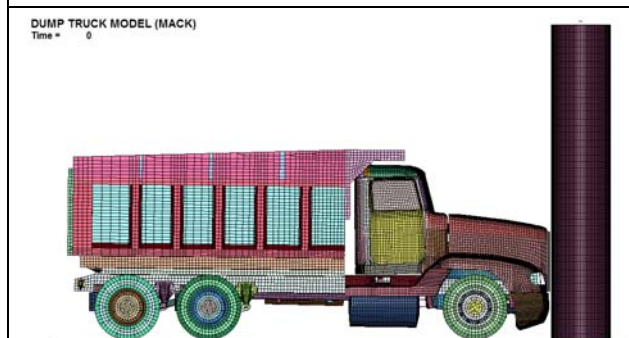
Figure 4.42. Matrix III, SUT, Deformable Ballast, 60 mph, Deformable Ballast Before.
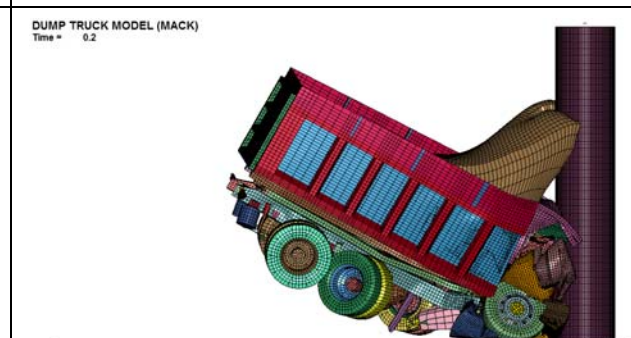
Figure 4.43. Matrix III, SUT, Deformable Ballast, 60 mph, Deformable Ballast After.

TRACTOR-TRAILER SIMULATION CASES

Matrix IV

The objective of Matrix IV was to analyze the effect of velocity on forces imposed by a tractor-trailer during impact into a pier. A pier diameter of 36 inches was used for each simulation. Velocities of 40, 50, and 60 mph were evaluated as shown in Table 4.2. Deformable ballast and trailer with a total vehicle weight of 80,000 lb was used for simulation in Matrix IV.

Table 4.2. Simulation Matrix and Summary for Tractor-Trailer Vehicle.

	Pier Diameter	Vehicle (Weight)	Cargo/Ballast	Impact Speed	50-ms Avg. Force (kip)	
					Engine Block	Ballast
Matrix IV	36 inches	Tractor-Trailer (80,000-lb)	Deformable	40	520	800
	36 inches	Tractor-Trailer (80,000-lb)	Deformable	50	580	
	36 inches	Tractor-Trailer (80,000-lb)	Deformable	60	600	1020
Matrix V	36 inches	Tractor-Trailer (80,000-lb)	Rigid	40	500	> 500
	36 inches	Tractor-Trailer (80,000-lb)	Rigid	50	550	> 2000
	36 inches	Tractor-Trailer (80,000-lb)	Rigid	60	600	> 2000

Figure 4.44 depicts force versus time as the tractor-trailer impacts the pier at 40, 50, and 60 mph. Increases in velocity of the tractor-trailer result in increases of the peak force on the pier. However, the soft cargo impact peak force for the 50 mph case showed a different profile and lesser force than that of the 60 mph and the 40 mph cases. This unexpected forse profile is discussed later. The ballast accounted for 50percent of the total 80-kip weight of the vehicle.
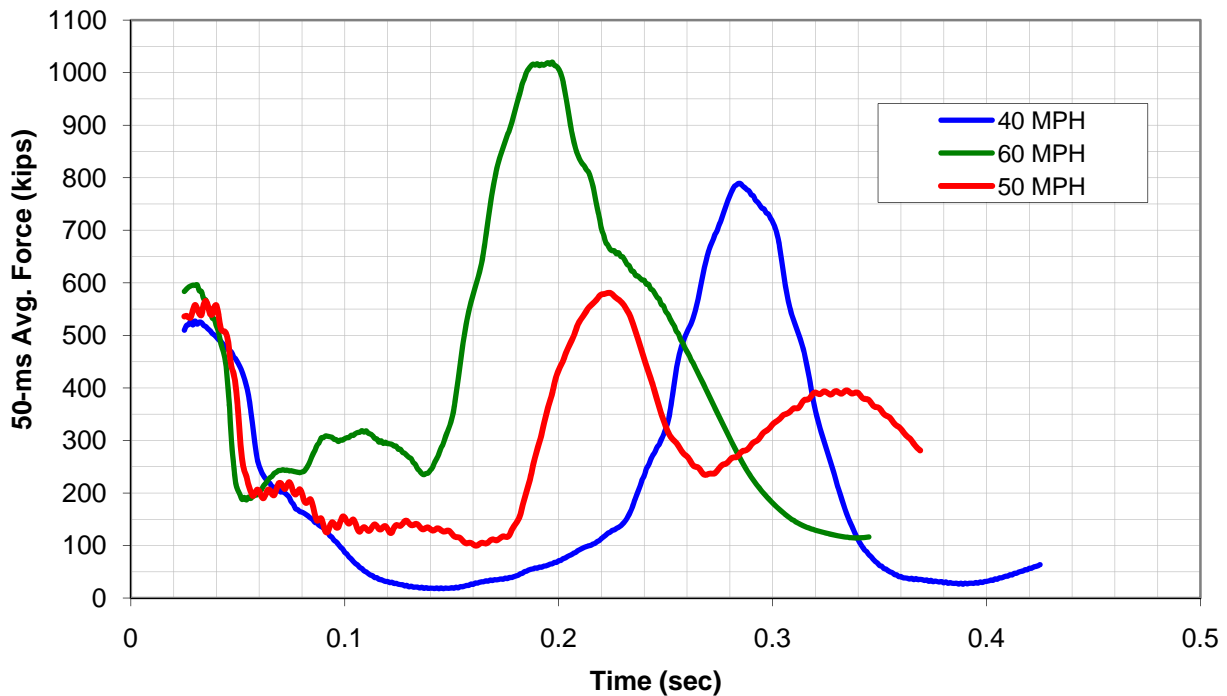
Figure 4.44. Matrix IV, Tractor Trailer, Deformable Ballast, 36-inch Pier, 50-ms Average Resultant Reactions.

It was desired to determine and/or validate the discrepancy in the force curve with regard to the variance in the 50 mph case as compared to the 40 and 60 mph cases. It was initially thought that the force curve should lie between the 60 and 40 mph for both magnitude and time scales. As seen in Figure 4.45, the curve begins to roll off at 0.225 sec and peak again at 0.325 sec. For this study, the 50 mph case was compared against the 60 mph case. It was determined that the drop in force is related to the engine slipping below the trailer during impact.
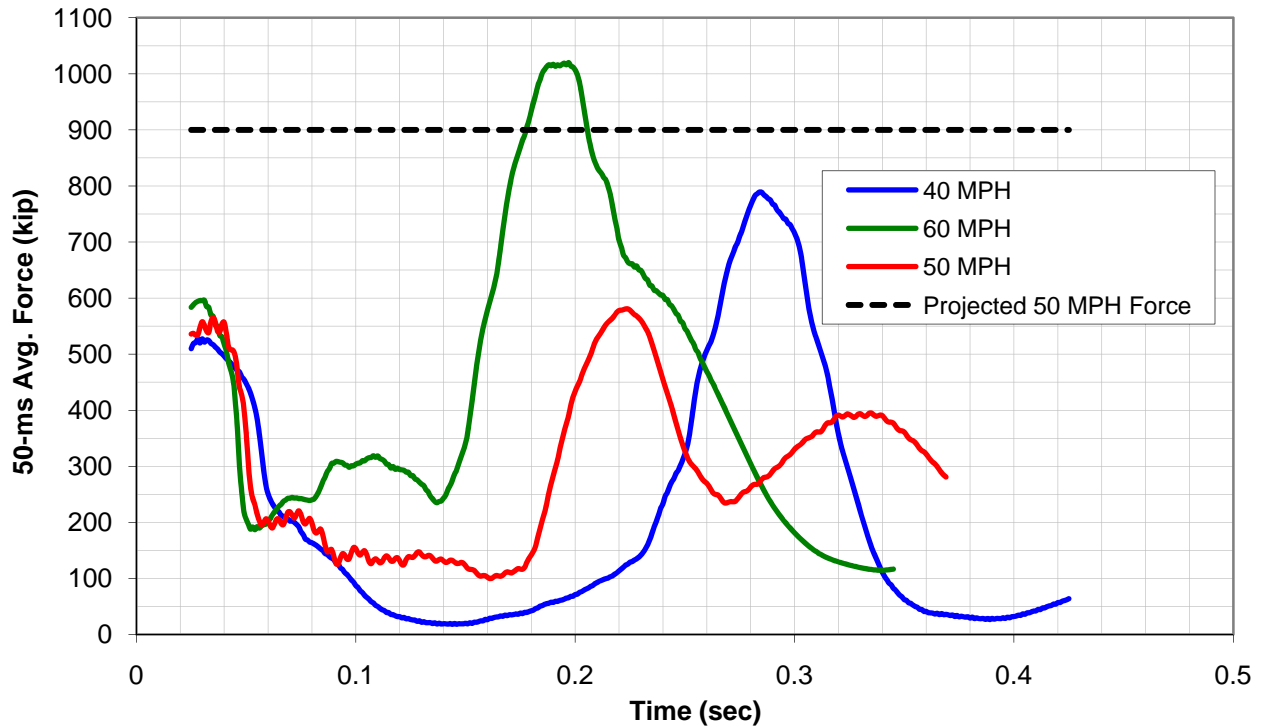
Figure 4.45. Matrix IV, Tractor Trailer, Deformable Ballast, 36-inch Pier, 50-ms Average Resultant Reactions with Proposed 50 mph Force.

Figure 4.46 is an isoperimetric view of the components that cause the peaks in the force curve. From previous analysis it was determined that the two major peaks in the force curve were directly related to the engine block mass and ballast mass (i.e., the trailer) and their interaction. The blue figure represents the trailer structural floor system; the red is the tractor engine block.

Figure 4.47 shows the interaction between the trailer structure and the engine block for both 50 and 60 mph cases. During the impact of the 60 mph case, the trailer structure remains interlocked with the engine block for the duration of the event. Thus, without crushing of the engine block, the force induced from the ballast is directly transferred into the pier. This type of interaction leads to the force profile shown in Figure 4.45. Figures 4.48 and 4.49 show interaction of the trailer with the engine in later stages of crush.
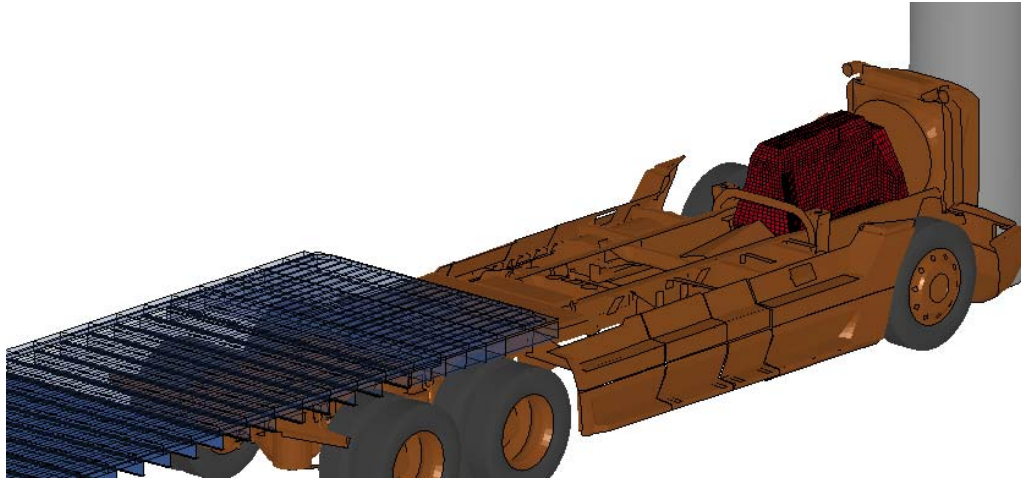
Figure 4.46. Tractor Trailer, System Components Causing Force Imparted on Pier.
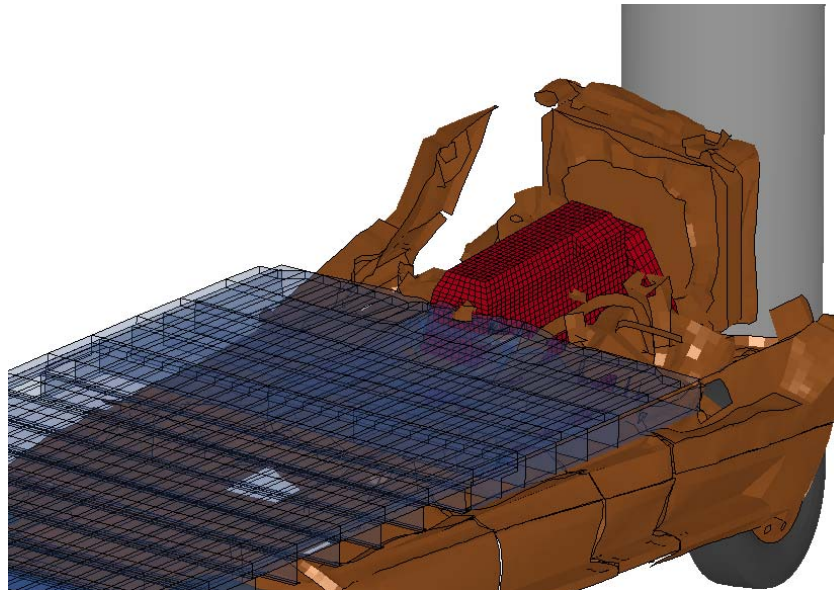
Figure 4.47. Tractor Trailer, 60 mph, Trailer-Engine Interaction Pre-Impact.
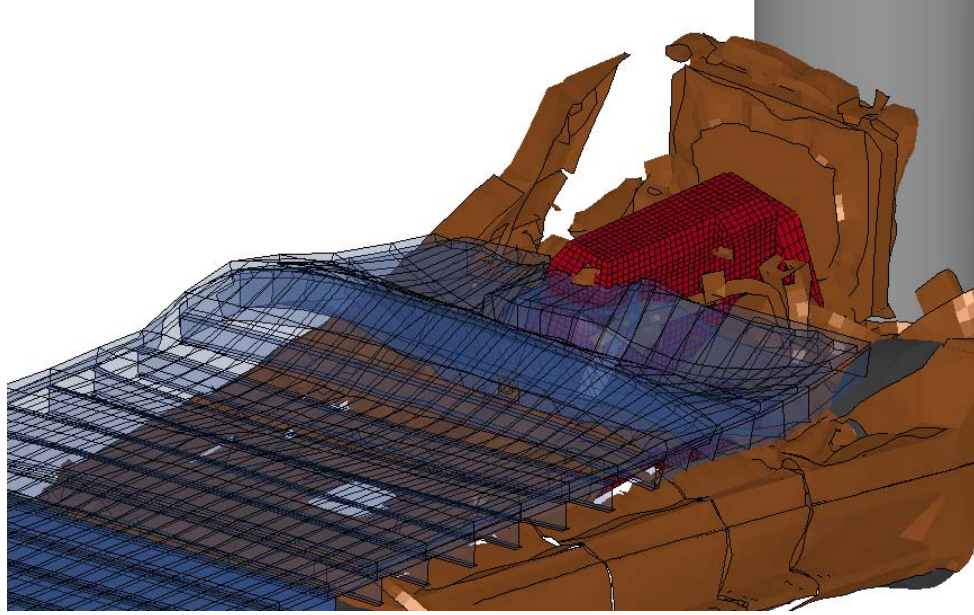
Figure 4.48. Tractor Trailer, 60 mph, Trailer-Engine Interaction Post-Impact.
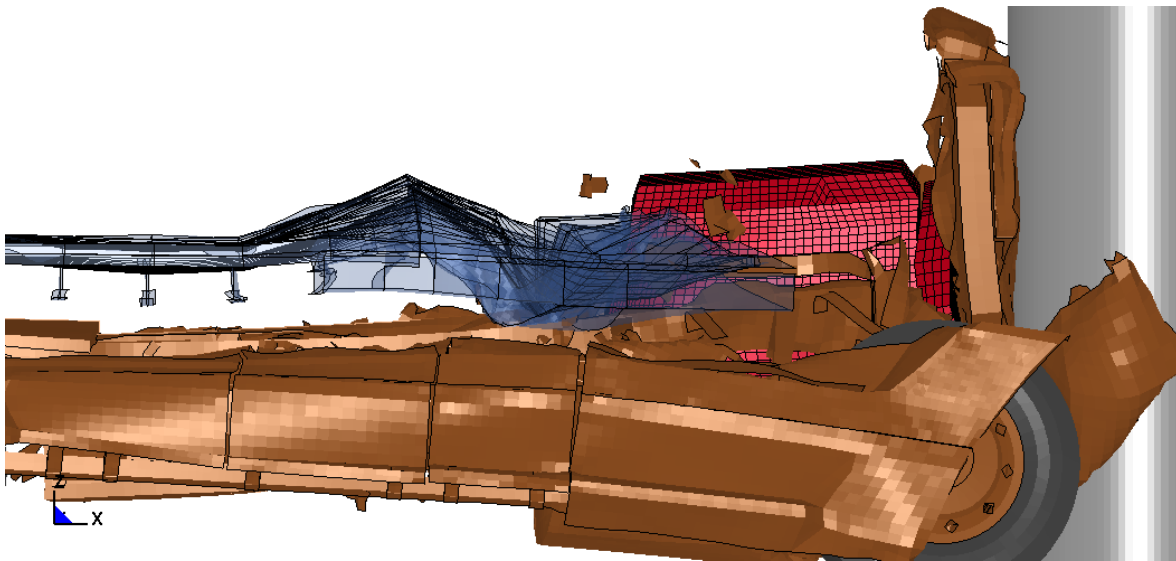
Figure 4.49. Tractor Trailer, 60 mph, Trailer-Engine Interaction Post-Impact (Right View).

When compared with the 60 mph case, the 50 mph case is variable. During the impact of the 50 mph case, the trailer structure does not remain in contact with the engine block for the duration of the event. Thus, the engine block cannot be used as a mechanism to induce force into the pier from the ballast. As seen in the Figures 4.50 through Figure 4.53, the trailer floor structure strikes the engine then slips and rides above the engine. The first peak in the force curve profile is a result of the initial contact between the trailer structure and engine. The valley

occurs during the slip and further crushing of the cab between the trailer and pier. The second peak is a result of the direct impact of the trailer into the pier once the cab is crushed fully.
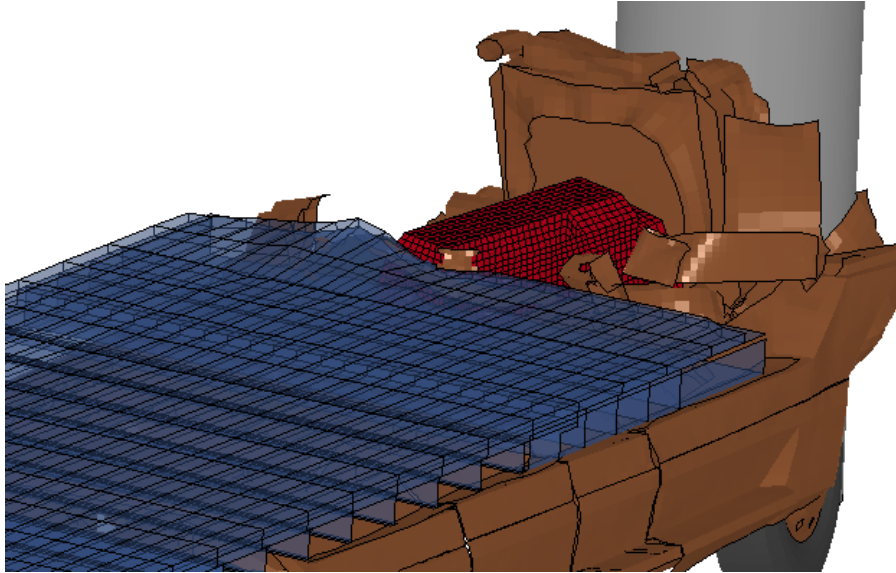
Figure 4.50. Tractor Trailer, 50 mph, Trailer-Engine Interaction Pre-Impact.
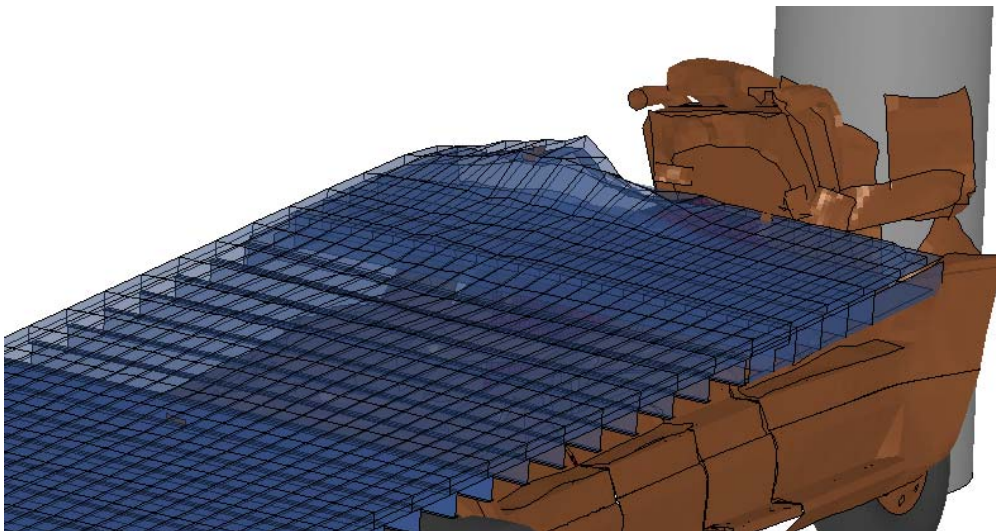
Figure 4.51. Tractor Trailer, 50 mph, Trailer-Engine Interaction Post-Impact.
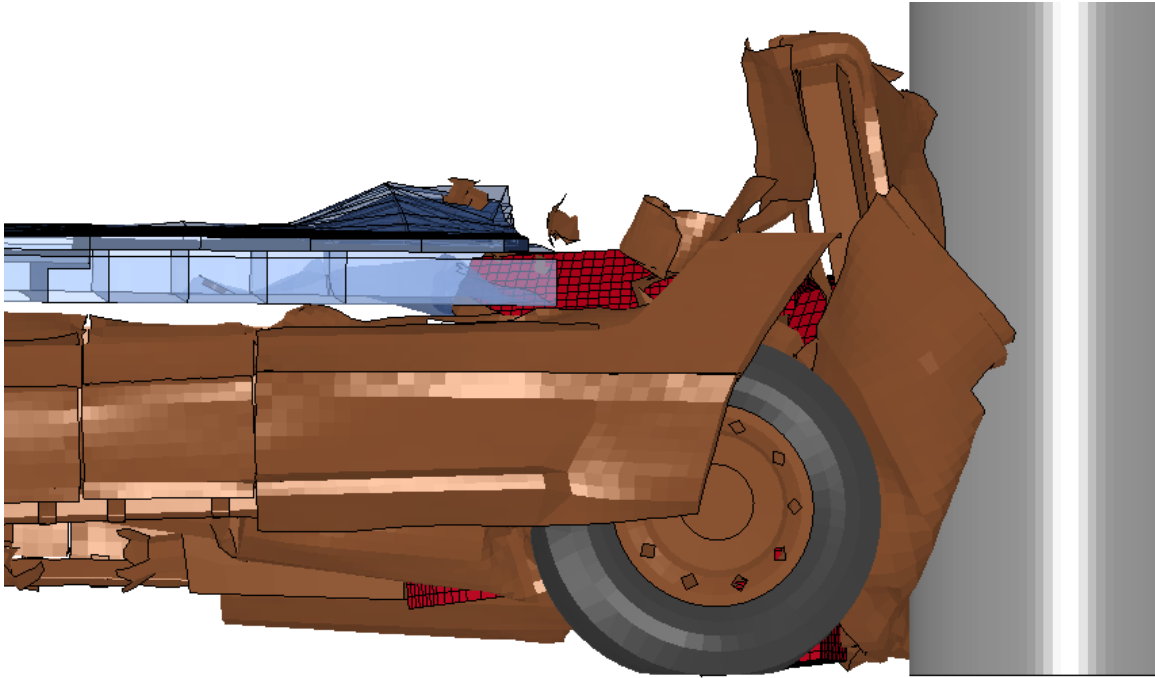
Figure 4.52. Tractor Trailer, 50 mph, Trailer-Engine Interaction Slipping Action Pre-Impact.
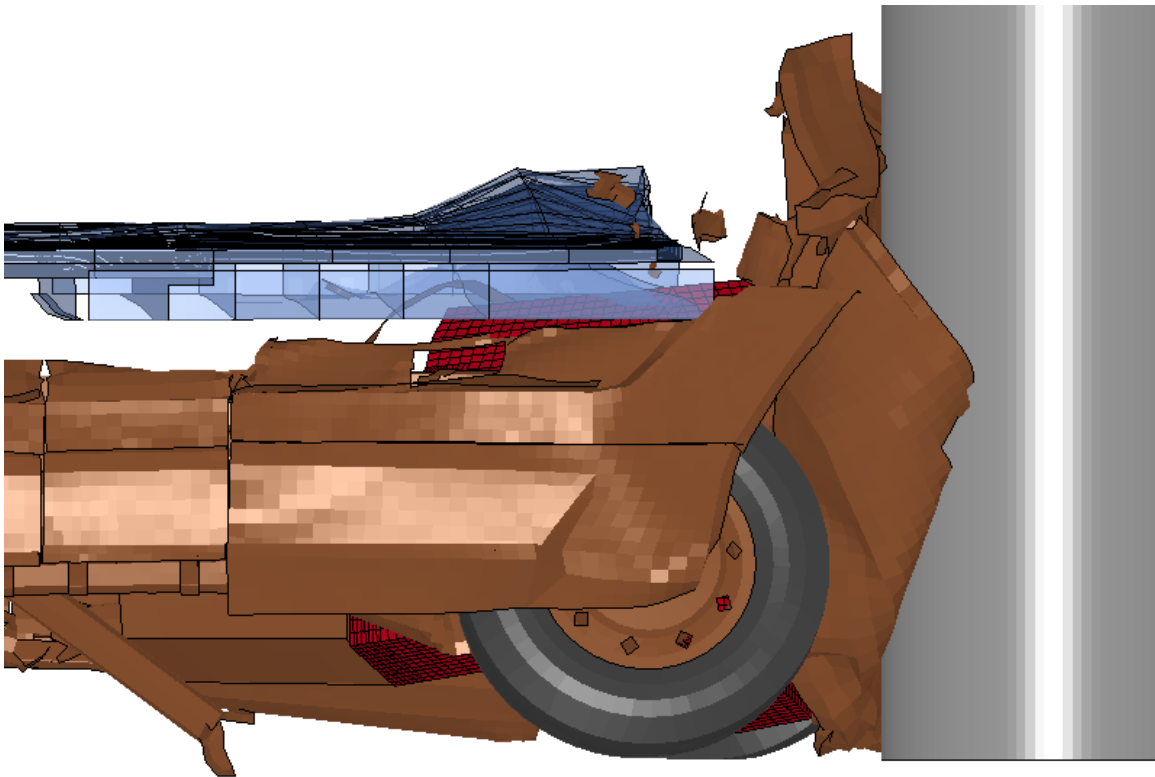
Figure 4.53. Tractor Trailer, 50 mph, Trailer-Engine Interaction Slipping Action Post-Impact.

Again, comparing the 60 mph case against the 50 mph, the contact is constant for 60 mph whereas the contact is variable for the 50 mph case. The two types of behavior are illustrated in Figures 4.54 and 4.55.
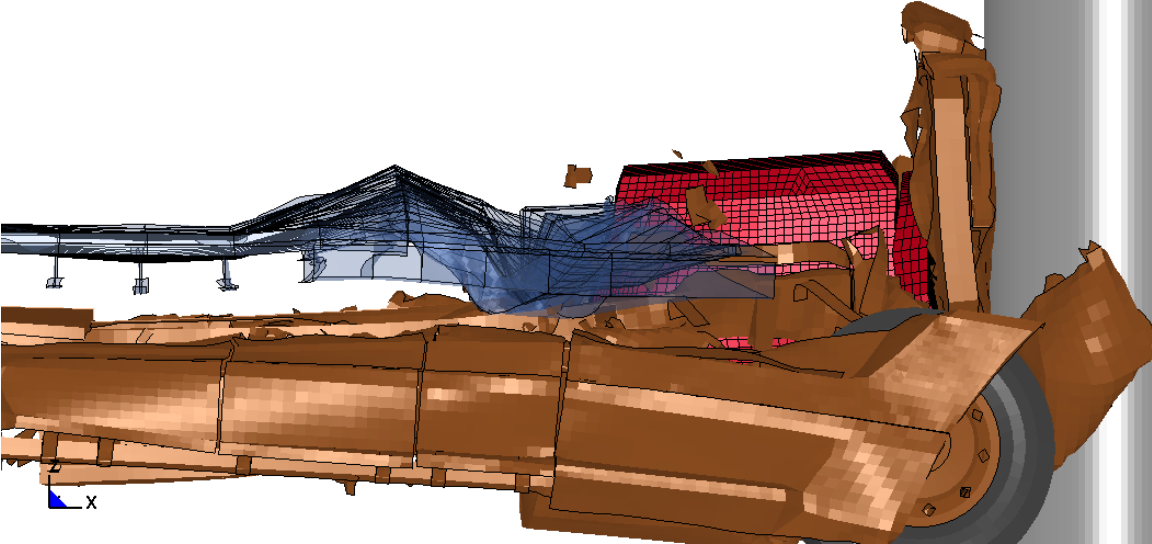
Figure 4.54. Tractor Trailer, 60 mph, Trailer-Engine Locking Interaction.
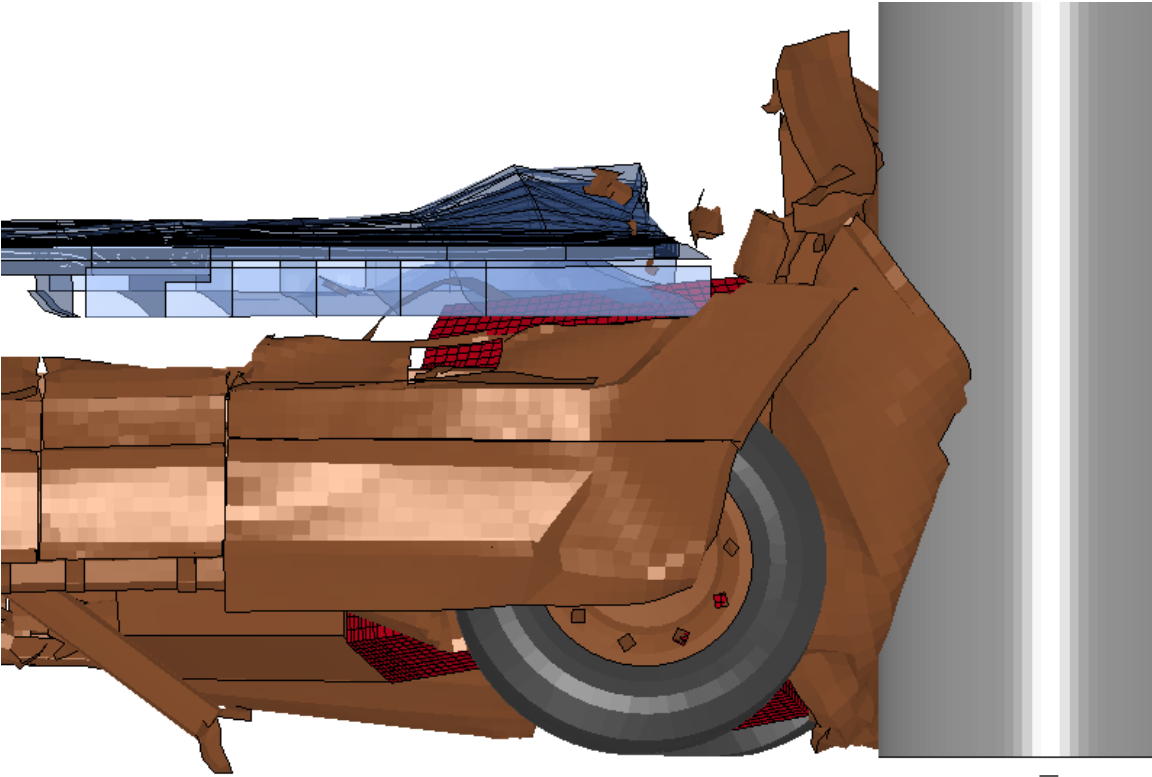
Figure 4.55. Tractor Trailer, 50 mph, Trailer-Engine Slipping Interaction.

Sloshing of deformable cargo is illustrated in Figures 4.56 through 4.59.
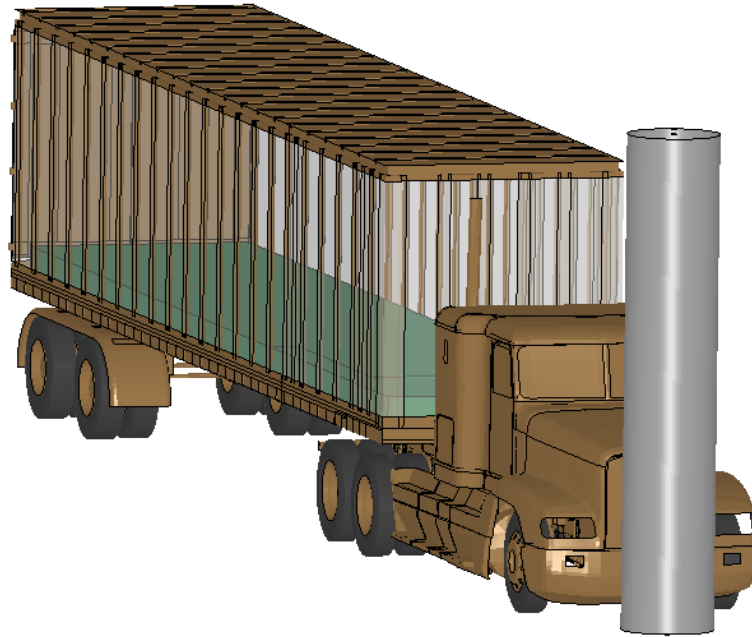
Figure 4.56. Tractor Trailer Deformable Cargo Pre-Impact.
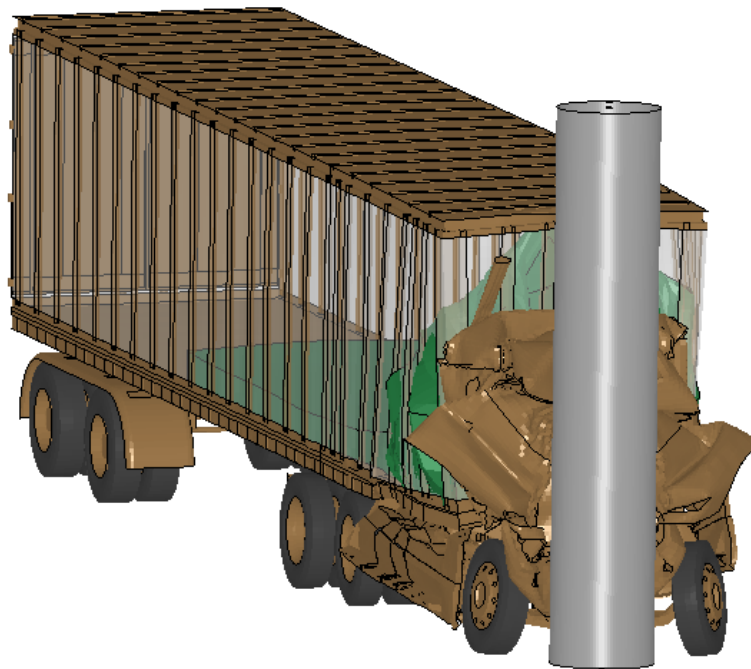
Figure 4.57. Sloshing of Tractor Trailer Cargo.
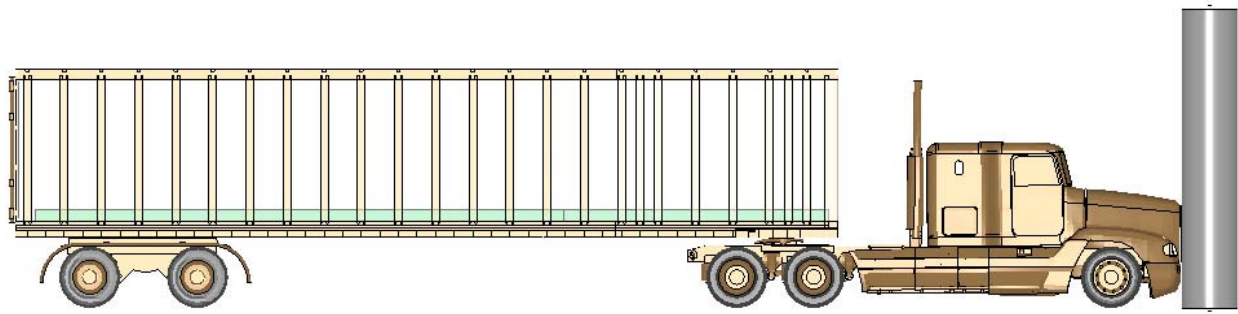
Figure 4.58. Trailer with Deformable Cargo Pre-Impact (Right View).
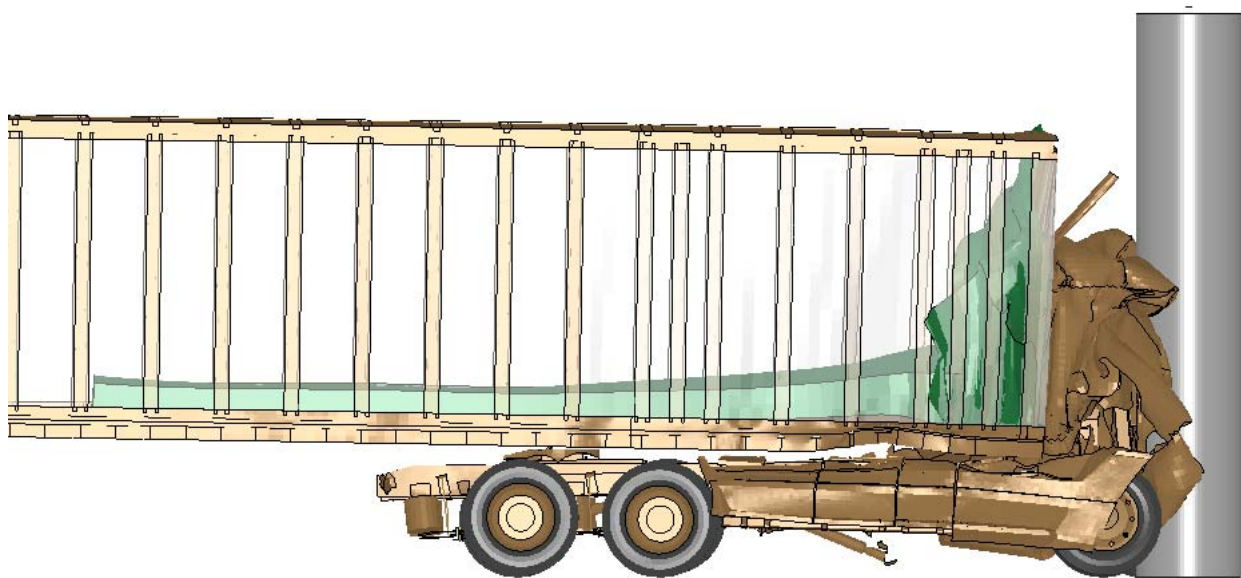
Figure 4.59. Sloshing of Tractor-Trailer Cargo (Right View).

Matrix V

The objective of Matrix V is to analyze the effects of velocity and its corresponding force a tractor-trailer imposes during impact into a pier using a rigid ballast. A pier diameter of 36 inches was used for each simulation. Velocities of 40, 50, and 60 mph were evaluated, as shown in Table 4.2. Rigid ballast in a deformable trailer with a total vehicle weight of 80,000 lb was used in this matrix.

Figure 4.60 depicts force versus time as the tractor-trailer impacts the pier at 40, 50, and 60 mph. Increases in velocity of the tractor-trailer result in increases of the peak force on the pier. Each simulation case for this matrix yielded unstable numerical results once the ballast

impacted the pier. This steep spike in the data is to be expected with the impact of two infinitely rigid bodies.
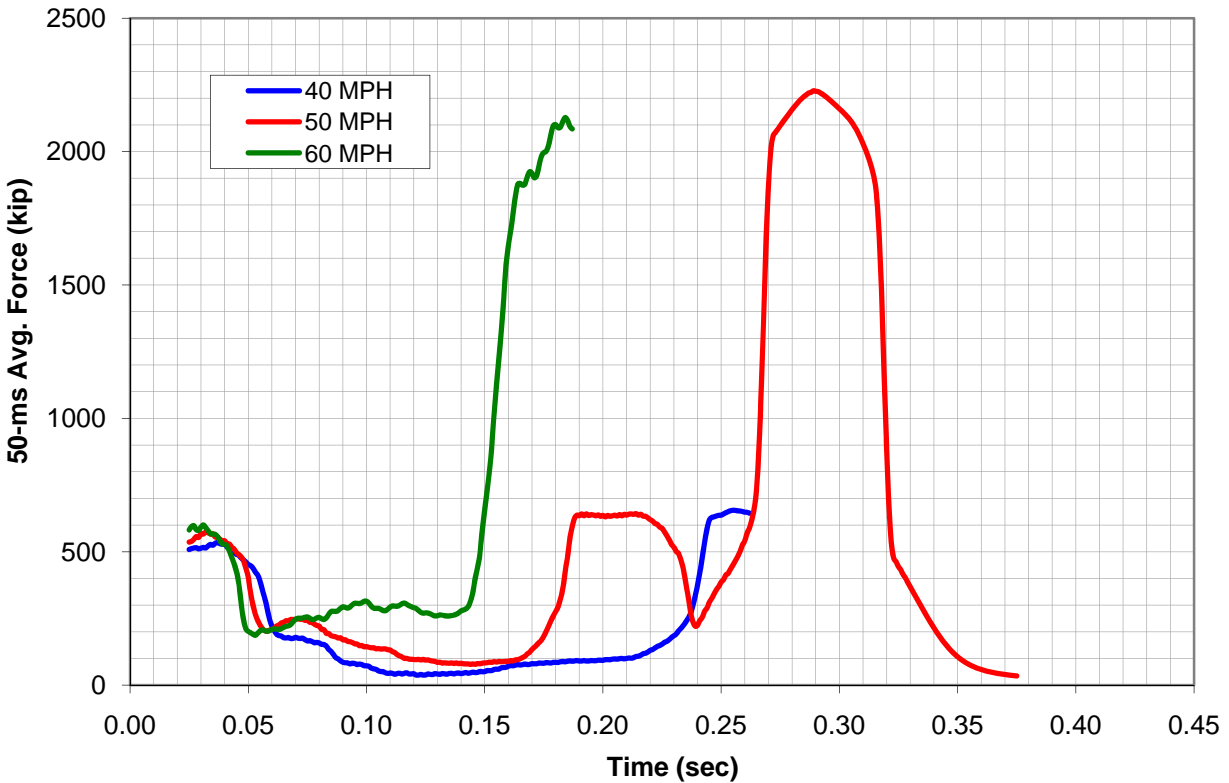
Figure 4.60. Tractor Trailer Matrix V, 36-inch Pier - Resultant Reactions 50-ms Average.

FORCE DISTRIBUTION ALONG THE HEIGHT OF THE PIER

In order to determine the distribution of the impact force along the height of the pier, a redefined pier model was constructed with segments to provide force values at a given height interval of the pier. Figure 4.61 depicts the segments of the pier model. Segments start at 1 ft above the ground and continue up to 9 ft above ground. Each segment covers a 6-inch portion of the pier. There are a total of 16 segments defined as shown in Figure 4.61. These forces are filtered to Society of Automotive Engineers (SAE) class 180 filter and then averaged using a 50-ms moving average.

Figure 4.62 shows the force distribution along the height of the pier over the duration of the impact. A contour view of the force distribution is also shown in Figure 4.63. From these data, the largest peak force can be taken at 0.2 sec. However, the force distribution given at time 0.2 sec may not necessarily correspond to the maximum resultant force.
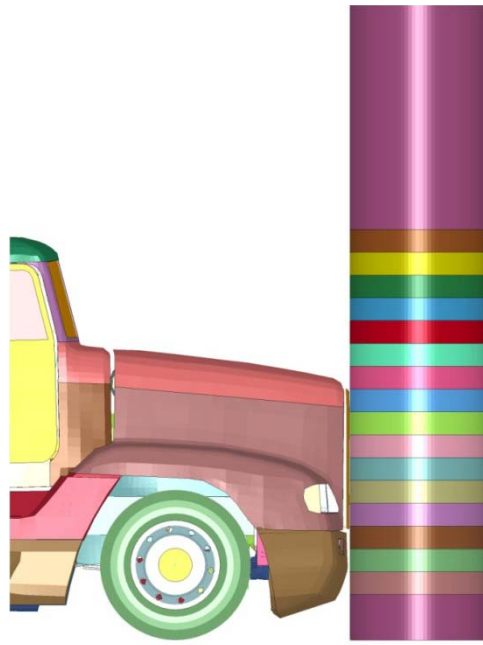
Figure 4.61. Parts Definition for Force Transducer on the Pier.
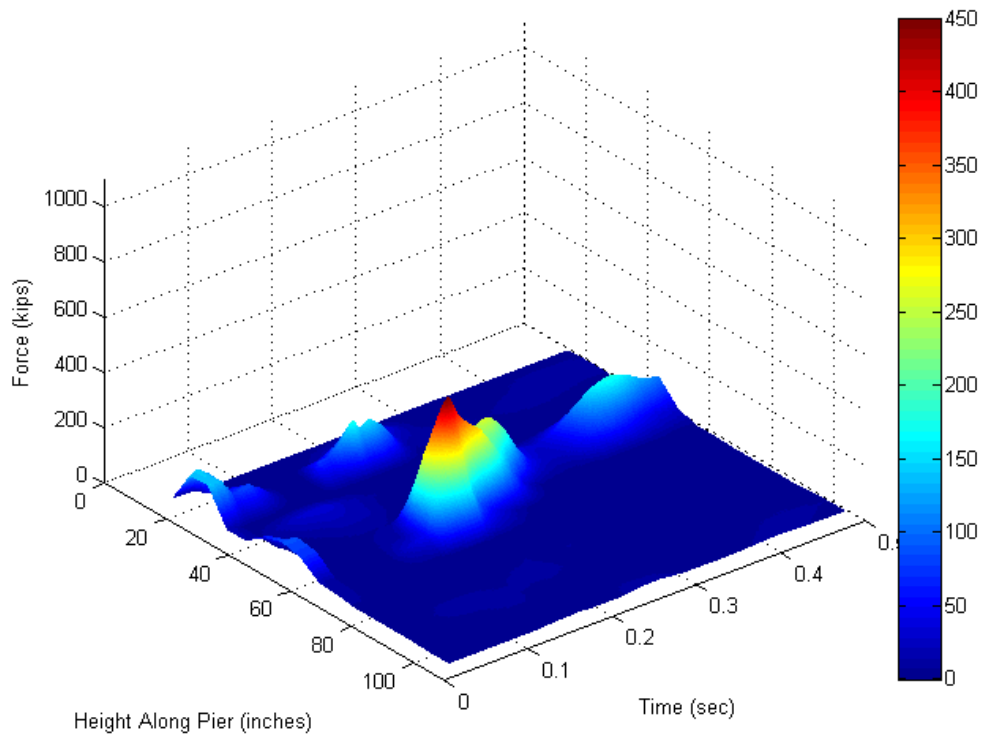
Figure 4.62. Tractor Trailer 50-ms Impact Force Distribution along Pier Height over Time.
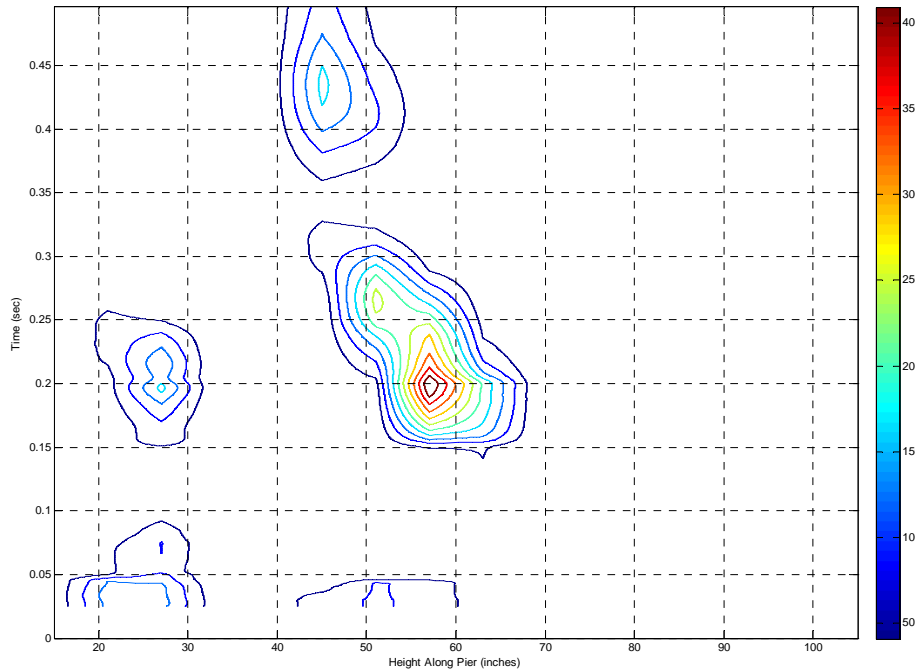
Figure 4.63. Tractor-Trailer 50-ms Impact Force Contour.

Figure 4.64 shows a slice of the force surface at 0.2 sec. This slice shows the force distribution along the height of the pier at 0.2 sec. By summing the area under the force distribution curve, the total impact force on the pier at 0.2 sec could be calculated, as well as the height the resultant force would act along the pier.
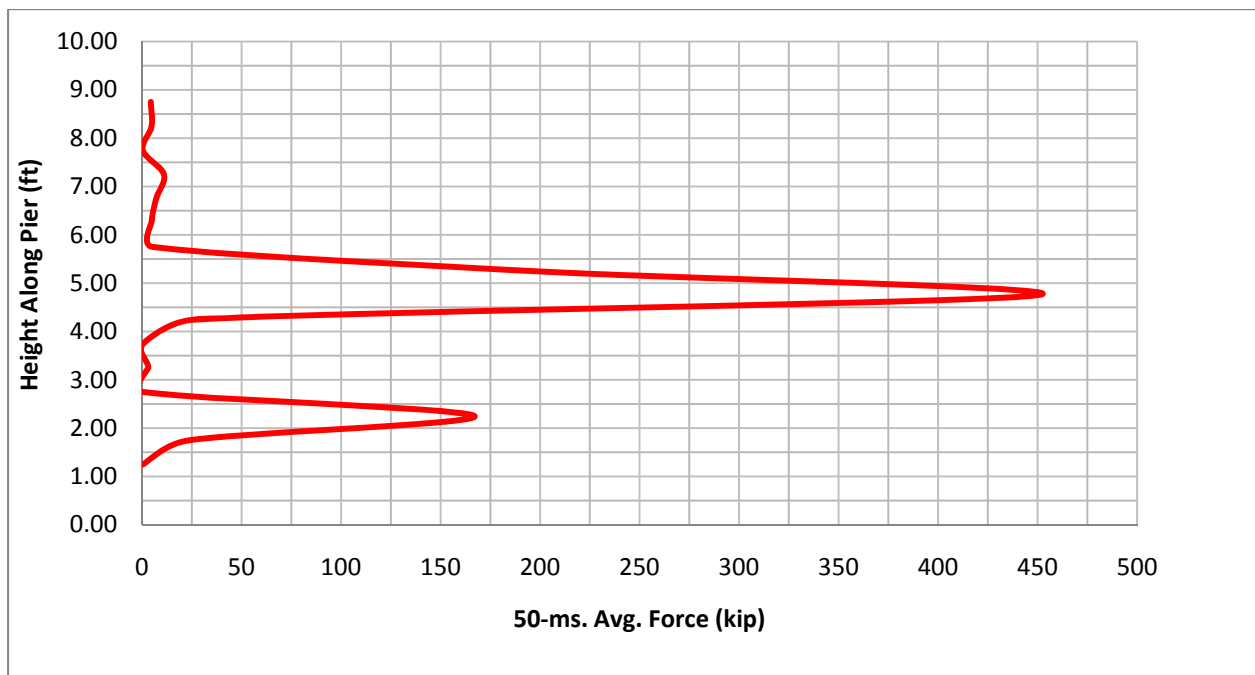
Figure 4.64. Tractor-Trailer Impact Force Distribution along the Height of the Pier at 0.2 sec.

SUMMARY AND CONCLUSIONS

Finite element analyses were conducted to determine the impact force experienced by a bridge pier upon impact by a heavy truck. A beta version of the tractor-trailer model was modified to incorporate needed articulation as well as refinement to enhance its fidelity. The tractor model was also modified to build a 65,000 lb SUT model as detailed in Appendix B. The pier itself was modeled as rigid, and thus these analyses will present the maximum exerted force possible from such impacts. Overall, the analyses conducted with these models showed that the impact force experienced by the pier is much larger than that stated in the AASHTO *LRFD* vehicle collision provisions. The values of the imparted force from the engine block impact ranges from 480 kip to 600 kip, while the values of the imparted force from the ballast impact (albeit through the squeezing of the cab) ranges from 480 kip to more than 2000 kip.

Effect of Pier Diameter

Three different diameters were simulated, 24 inches, 36 inches, and 48 inches as listed in Matrix I in Table 4.1. The results of the analyses indicate that the diameter of the pier does not have significant effect on the impact force exerted by a given truck and speed as seen in Figure 4.65.
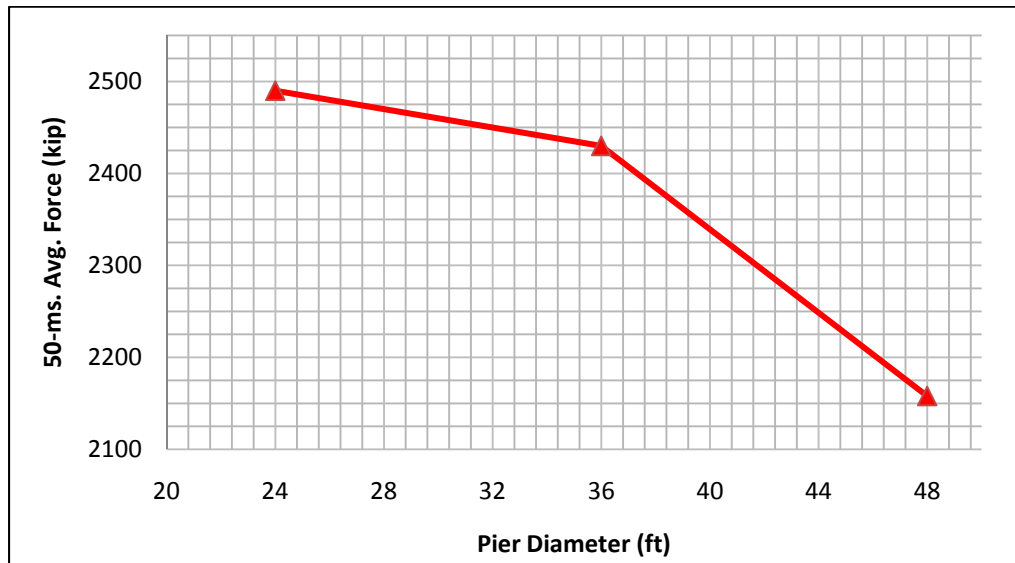
Figure 4.65. SUT Matrix I Force Pier Diameter Relation Summary.

Effect of Truck Speed

Three different speeds were simulated, 40 mph, 50 mph, and 60 mph, as listed in matrices II, III, IV, and V. All of these analyses showed a direct correlation (approximately linear) between the impact force (maximum and the second peak) and the impact speed, with exception to the 50 mph tractor-trailer case. The aforementioned data are represented in Figures 4.66 through 4.67.

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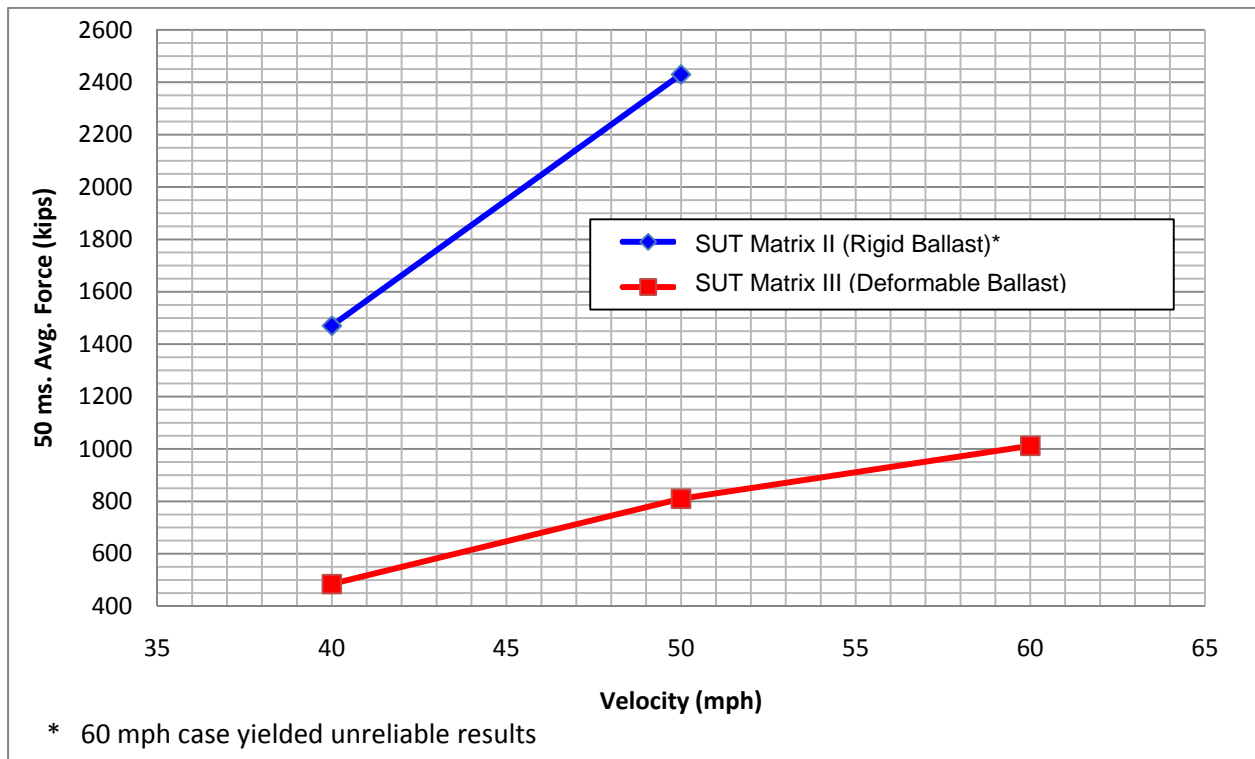
Figure 4.66. SUT Matrix II/III Force Velocity Relation Summary.
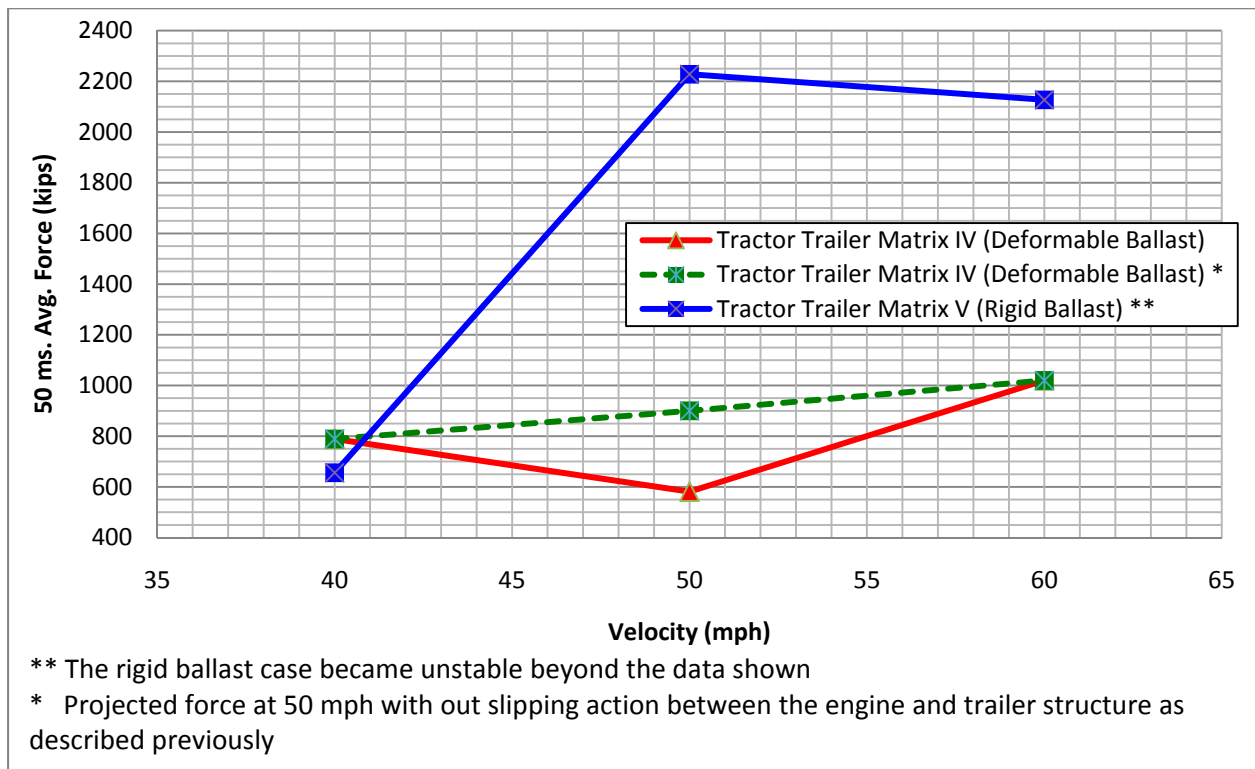
Figure 4.67. Tractor Trailer Matrix IV/V Force Velocity Relation Summary.

Effect of Mass and Body Type

The more significant effect of mass occurred when the ballast (cargo) came into contact with the cab and subsequently imparted additional loading onto the pier. The engine block impact force was not significantly changed among the simulated cases. However, as the cargo impacted the cab, the impact force was lowest for empty haul and largest for rigid mass. The deformable ballast imparted less force on the pier than that of a rigid ballast.

CHAPTER 5. RISK OF COLLISIONS

INTRODUCTION

This chapter describes the estimation of the risk of collision between a heavy vehicle and a bridge pier using data collected in the States of Texas and Minnesota. This study focused on the higher level highway network, such as principal arterials and collectors. Since probability for a bridge pier to fail is greater at higher speeds, under the condition that it is hit by a heavy vehicle, highways meeting the above criteria were selected.

For this project, two types of analyses were conducted. The first one is similar to the approach proposed by the AASHTO *LRFD* vessel impact methodology on waterways. Under this type, the crash risk is estimated for each bridge pier individually, using methodologies commonly used in epidemiology. For the second type of analysis, negative binomial regression models were used to estimate crash risk as a function of truck flow or exposure and various covariates describing the physical characteristics of the road. The models could be used for transportation agencies who are interested in analyzing bridges located on segments or corridor rather than analyzing each bridge individually. The crash risk analysis and the models were estimated for undivided and divided highways separately.

This chapter is divided into six sections. The first section describes the process used to assemble the data. The second section outlines the characteristics of the data. The third section describes the results of the crash risk analysis. The fourth section gives a brief note on the Empirical Bayes (EB) method for refining the safety estimate of a site. The fifth section describes how the methodology developed in this work can be used to estimate the risk for a bridge pier to be hit by a heavy vehicle. Two examples are provided. The last section provides a summary of the analysis.

DATA COLLECTION

This section describes the characteristics of two datasets used in this study. The first part summarizes the characteristics of the Texas data. The second part summarizes the Minnesota data.

Texas Data

The data were collected from three sources.

Crash Data

The crash data were collected from the Texas DPS for the years 1998–2001. Three databases were used for this purpose: the accident, vehicle and driver information, and causing factor (causality) files. The accident file contains detailed information on the highway class,

location, the severity and the time of the crash among others. The vehicle and driver data include information about vehicle type, vehicle model, driver age etc. The causality file contains data on the accident causing factors. All the data were available electronically.

Only crashes that met the following criteria were extracted:

1. Crashes that occurred on interstates, state and US highways;
2. Crashes that occurred on main lanes;
3. Crashes that only involve a heavy vehicle; and,
4. The run-off road crashes not hitting at a bridge pier and the run-off road crashes hitting the bridge pier.

Network Data

The data related to the highway infrastructures were collected using Roadway/Highway Network Inventory (RHiNo) and Texas Reference Marker (TRM), databases managed by the Texas Department of Transportation (TxDOT). The 2003 TRM database was used to identify potential highway segments. The roadway geometric characteristics for segments were included in the analysis when the following conditions were met:

1. Road segments that are only defined as interstates, state, and US highway main lanes; and
2. Sites that have lane width between 8 and 15 ft.

Bridge Location

The location of bridges on the network (crossing on top) detailed below was provided by the TxDOT Transportation Planning and Programming Division. The file contained the location of the bridge to the $1/1000^{th}$ mile, the bridge number, and whether the facility was undivided or divided.

All the databases were linked using the reference-marker system as well as the control-section-mile point. Only segments that had a bridge crossing over the highway were utilized. Since the years between the accident files and TRM files were not the same, some segments could not be matched. They were subsequently removed from the analysis. At the end of the merging process, 2,836 segments were used for the analysis.

Minnesota Data

The data were collected from two sources.

Crash and Network Data

The crash and network data were collected from the Federal Highway Administration's (FHWA) Highway Safety Information System (HSIS), maintained by the University of North

Carolina for the years 2002–2006. Three databases were used for this purpose: the accident, vehicle information, and road files. The accident file contains detailed information on the highway class, location, the severity and the time of the crash among others. The vehicle data include information about vehicle type, vehicle model, etc. The road file contains data related to the highway infrastructures. All the data were available electronically.

Only crashes that met the following criteria were extracted:

1. Crashes that occurred on interstates, state and US highways;
2. Crashes that occurred on main lanes;
3. Crash that only involve a heavy vehicle; and,
4. The run-off road crashes not hitting at a bridge pier and the run-off road crashes hitting the bridge pier.

The roadway geometric characteristics for segments were included in the analysis when the following conditions were met:

1. Road segments that are only defined as interstates, state, and US highway main lanes; and
2. Sites that have a lane width less than or equal to 15 ft.

Bridge Location

The location of bridges on a given network (crossing on top) was provided by the Minnesota Department of Transportation (MnDOT). All the databases were linked using the control-section-mile point. Only segments that had a bridge crossing over the highway were utilized. At the end of the merging process, 606 segments (552 divided segments and 54 undivided segments) were used for the analysis.

SUMMARY OF STATISTICS

This section describes the characteristics of the Texas and Minnesota data used for the analysis. For this analysis, the highway segments were separated into two groups: divided and undivided highways. Divided highways include any segment that is separated by a grassy median (curbed and uncurbed) or a positive barrier where a bridge pier could be located (not verified by site visits). Also, to determine whether crash risk involving heavy vehicles differs between straight (tangent) and curved sections, separate analyzes were performed on both types of segments in Texas. Truck crashes in Texas also include pickup trucks, utility vehicles, and small vans. Although, the crashes involving these vehicles were included in this section, they were removed during the crash risk analysis.

Undivided Highways

Table 5.1 tabulates the summary statistics for variables related to undivided segments in Texas. The 350 undivided segments were extracted from the data. Table 5.1 shows that truck percentages varied from 1.2 percent to 51.6 percent.

Table 5.1. Summary Statistics for Geometric and Operational Variables for Undivided Highways (Texas Data).

Variable	Min	Max	Average (Std Dev)	Sum
Segment Length (mile)	0.10	11.07	0.75 (1.10)	264.19
Lane Width (ft)	9	15	12.23 (1.05)	---
Number of Bridges	1	8	1.21 (0.61)	423
Bridges/Mile	0.09	19.87	3.91 (3.36)	---
Number of Curves	0	7	0.97(1.22)	339
Curves/Mile	0	19.61	2.07(3.00)	
Average Shoulder Width (ft)	0	17	5.81 (3.81)	---
AADT	128	51,250	7,380 (7,222)	---
Truck Percentages	1.2%	51.6%	16.13%	---
Truck AADT	12	5905	928 (790)	---

Table 5.2 summarizes the number of heavy vehicle crashes as a function of level of severity: (K) Fatal, Injury Type A (incapacitating), Injury Type B (non-incapacitating), Injury Type C (possible injury), and PDO (Property Damage Only). The crash data cover a 4-year period (1998–2001). For the 4-year time period, very few crashes involving a heavy vehicle hitting a bridge pier were reported for undivided segments.

Table 5.3 summarizes the statistics for variables related to tangent sections on undivided highways. A total of 156 straight sections located on undivided segments were extracted from the data. Table 5.4 summarizes the number of heavy vehicle crashes as a function of severity levels for undivided straight segments. Table 5.5 summarizes the statistics for variables related to horizontal curves located on undivided highways. Twenty-five horizontal curves on undivided segments were extracted from the data.

Table 5.2. Summary Statistics for Truck Crashes on Undivided Highways (Texas Data).

Severity	Run off Road (ROR)		Hit bridge pier	
	Number of crashes	Percentage	Number of crashes	Percentage
Fatal (K)	20	3.1%	0	0.0%
Incapacitating injury (A)	67	10.5%	1	14.3%
Nonincapacitating injury (B)	140	21.9%	1	14.3%
Possible injury (C)	173	27.0%	3	42.9%
PDO (O)	240	37.5%	2	28.6%
Total	640	100.0%	7	100.0%

Table 5.3. Summary Statistics for Geometric and Operational Variables on Tangent Sections of Undivided Highways (Texas Data).

Variable	Min	Max	Average (Std Dev)	Sum
Segment Length (mile)	0.10	3.73	0.41 (0.46)	64.18
Lane Width (ft)	9.75	15	12.27 (1.09)	---
Number of Bridges	1	3	1.19 (0.49)	423
Bridges/Mile	0.27	19.87	4.68 (3.26)	---
Average Shoulder Width (ft)	0	14	5.49 (3.89)	---
AADT	128	39,750	7,520 (7,315)	---
Truck Percentages	1.2%	51.6%	15.61%	---
Truck AADT	12	5905	919 (816)	---

Table 5.4. Summary Statistics for Truck Crashes on Tangent Sections of Undivided Highways (Texas Data).

Severity	ROR		Hit bridge pier	
	Number of crashes	Percentage	Number of crashes	Percentage
Fatal (K)	7	3.4%	0	0.0%
Incapacitating injury (A)	9	4.4%	0	0.0%
Nonincapacitating injury (B)	55	27.1%	1	50.0%
Possible injury (C)	58	28.6%	0	0.0%
PDO (O)	74	36.5%	1	50.0%
Total	203	100.0%	2	100.0%

Table 5.5. Summary Statistics for Geometric and Operational Variables for Horizontal Curves on Undivided Highways (Texas Data).

Variable	Min	Max	Average (Std Dev)	Sum
Segment Length (mile)	0.10	1.33	0.28 (0.26)	6.94
Lane Width (ft)	10	15	12.08 (1.00)	---
Number of Bridges	1	2	1.12 (0.33)	28
Bridges/Mile	0.75	15.38	6.04 (3.39)	---
Average Shoulder Width (ft)	0	10	6.98 (2.85)	---
Degree of Curvature	0	10	1.65 (2.29)	---
AADT	1,038	17,525	6,933 (5,202)	---
Truck Percentages	1.8%	38.7%	15.85%	---
Truck AADT	54	2453	893 (681)	---

Table 5.6 tabulates the number of heavy vehicle crashes as a function of severity levels for horizontal curves located on undivided segments in Texas. Since there were only 25 horizontal curves on undivided sections, no heavy vehicle hitting a bridge pier was reported.

Table 5.6. Summary Statistics for Truck Crashes on Horizontal Curves of Undivided Highways (Texas Data) .

Severity	ROR		Hit bridge pier	
	Number of crashes	Percentage	Number of crashes	Percentage
Fatal (K)	0	0%	0	---
Incapacitating injury (A)	5	20.8%	0	---
Nonincapacitating injury (B)	3	12.5%	0	---
Possible injury (C)	9	37.5%	0	---
PDO (O)	7	29.2%	0	---
Total	24	100.0%	0	---

Table 5.7 gives the summary statistics for truck crashes on undivided segments. As detailed below, there were very few truck run-off-the-road and hit bridge pier crashes reported on undivided segments. Thus, the summary statistics for highway geometric and operational variables were not provided, and as a result, regression models were not developed.

Table 5.7. Summary Statistics for Truck Crashes on Undivided Highways (Texas Data).

Severity	ROR		Hit bridge pier	
	Number of crashes	Percentage	Number of crashes	Percentage
Fatal (K)	1	33%	0	0%
Incapacitating injury (A)	0	0%	0	0%
Nonincapacitating injury (B)	0	0%	0	0%
Possible injury (C)	0	0%	0	0%
PDO (O)	2	67%	2	100%
Total	3	100.0%	2	100.0%

Divided Highways

Table 5.8 tabulates the summary statistics for variables for divided highway segments in Texas. There were 2486 divided segments used for this part of the analysis.

Table 5.8. Summary Statistics for Geometric and Operational Variables for Divided Highways (Texas Data).

Variable	Min	Max	Average (Std Dev)	Sum
Segment Length (mile)	0.10	13.43	1.15 (1.32)	2,862.92
Lane Width (ft)	8	15	12.01 (0.54)	---
Number of Bridges	1	32	2.25 (1.76)	5,599
Bridges/Mile	0.11	55.56	4.07 (4.37)	---
Number of Curves	0	17	1.11(1.38)	2,765
Curves/Mile	0	33.11	1.58(2.55)	--
Average Outside Shoulder Width (ft)	0	24	9.54 (2.32)	---
Average Inside Shoulder Width (ft)	0	24	5.71 (3.31)	---
AADT	698	334,485	54,877 (54,298)	---
Truck Percentages	1.6%	70.1%	19.08%	---
Truck AADT	168	25,086	6,696 (4,657)	---

Table 5.9 summarizes the number of heavy vehicle crashes as a function of severity levels for divided segments in Texas. Table 5.10 tabulates the summary statistics for variables related to tangent (straight) sections located on divided highway segments. There were 912 tangent sections located on divided segments that were used for this part of the analysis.

Table 5.9. Summary Statistics for Truck Crashes on Divided Highways (Texas Data).

Severity	ROR		Hit bridge pier	
	Number of crashes	Percentage	Number of crashes	Percentage
Fatal (K)	601	1.9%	14	8.8%
Incapacitating injury (A)	2239	6.9%	24	15.1%
Nonincapacitating injury (B)	6177	19.1%	39	24.5%
Possible injury (C)	10557	32.7%	50	31.4%
PDO (O)	12752	39.4%	32	20.1%
Total	32326	100.0%	159	100.0%

Table 5.10. Summary Statistics for Geometric and Operational Variables of Tangent Sections of Divided Highways (Texas Data).

Variable	Min	Max	Average (Std Dev)	Sum
Segment Length (mile)	0.10	7.91	0.78 (0.91)	707.76
Lane Width (ft)	8	15	12.01 (0.47)	---
Number of Bridges	1	14	1.94 (1.26)	1771
Bridges/Mile	0.22	55.56	5.02 (5.02)	---
Average Outside Shoulder Width (ft)	0	20	9.65 (2.09)	---
Average Inside Shoulder Width (ft)	0	20	5.51 (3.15)	---
AADT	1413	2,67,610	53,222 (53,126)	---
Truck Percentages	2.1%	69.4%	19.27%	---
Truck AADT	273	22,574	6711 (4525)	---

Table 5.11 summarizes the number of heavy vehicle crashes as a function of severity levels for tangent sections located on divided segments. Table 5.12 summarizes the statistics for key variables for horizontal curves located on divided highway segments. A total of 540 horizontal curves located on divided segments were used for this part of the analysis. Table 5.13 tabulates the number of heavy vehicle crashes as a function of severity levels for horizontal curves on divided segments.

Table 5.11. Summary Statistics for Truck Crashes on Tangents of Divided Highways (Texas Data).

Severity	ROR		Hit bridge pier	
	Number of crashes	Percentage	Number of crashes	Percentage
Fatal (K)	167	1.9%	5	10.9%
Incapacitating injury (A)	549	6.3%	8	17.4%
Nonincapacitating injury (B)	1684	19.4%	6	13.0%
Possible injury (C)	2900	33.5%	16	34.8%
PDO (O)	3364	38.8%	11	23.9%
Total	8664	100.0%	46	100.0%

Table 5.12. Summary Statistics for Geometric and Operational Variables of Horizontal Curves of Divided Highways (Texas Data).

Variable	Min	Max	Average (Std Dev)	Sum
Segment Length (mile)	0.10	2.84	0.30 (0.22)	161.38
Lane Width (ft)	8	15	12.00 (0.64)	---
Number of Bridges	1	12	1.78 (1.13)	962
Bridges/Mile	1.15	58.82	7.79 (5.77)	---
Average Outside Shoulder Width (ft)	0	24	9.59 (2.28)	---
Average Inside Shoulder Width (ft)	0	16	6.06 (3.46)	---
Degree of Curvature	0	40	1.39 (2.62)	---
AADT	698	3,34,485	63,830 (57,595)	---
Truck Percentages	2.2%	69.7%	17.02%	---
Truck AADT	223	25,086	6973 (4943)	---

Table 5.13. Summary Statistics for Truck Crashes on Horizontal Curves of Divided Highways (Texas Data).

Severity	ROR		Hit bridge pier	
	Number of crashes	Percentage	Number of crashes	Percentage
Fatal (K)	49	1.4%	1	2.0%
Incapacitating injury (A)	238	6.9%	9	18.4%
Nonincapacitating injury (B)	672	19.4%	17	34.7%
Possible injury (C)	1164	33.6%	9	18.4%
PDO (O)	1345	38.8%	13	26.5%
Total	3468	100.0%	49	100.0%

Table 5.14 tabulates the summary statistics for variables related to divided segments in Minnesota. The 552 divided segments were extracted from the data. The heavy vehicle volume ranged from 200 to 10,480 vehicles per day per segment. Table 5.15 summarizes the number of heavy vehicle crashes as a function of severity levels for divided segments in Minnesota.

Table 5.14. Summary Statistics for Geometric and Operational Variables on Divided Highways (Minnesota Data).

Variable	Min	Max	Average (Std Dev)	Sum
Segment Length (mile)	0.002	14.098	1.006 (1.695)	555.319
Lane Width (ft)	11	15	12.23 (0.60)	---
Number of Bridges	1	9	1.59 (1.06)	879
Bridges/Mile	0.16	571.43	12.33 (40.91)	---
Average Outside Shoulder Width (ft)	0	13	9.05 (2.42)	---
AADT	2900	2,02,000	59,882 (46,428)	---
Truck AADT	200	10,480	3,346 (2,110)	---

Table 5.15. Summary Statistics for Truck Crashes on Divided Highways (Minnesota Data).

Severity	ROR Number of crashes	ROR Percentage	Hit bridge pier Number of crashes	Hit bridge pier Percentage
Fatal (K)	5	1.2%	1	3.6%
Incapacitating injury (A)	5	1.2%	0	0%
Nonincapacitating injury (B)	52	12.4%	3	10.7%
Possible injury (C)	78	18.5%	1	3.6%
PDO (O)	281	66.7%	23	82.1%
Total	421	100.0%	28	100.0%

Distribution by Vehicle Body Style

Table 5.16 tabulates the distribution of truck run-off-the road and hit a bridge pier crashes by the heavy vehicle body style on undivided and divided roads. A more exhaustive description of the truck types by body type can be found in Appendix C.

**Table 5.16. Distribution of ROR and Hit Bridge Pier Crashes
by Heavy Vehicle Body Style (Texas).**

TEXAS 6 VEHICLE CLASSIFICATIONS (SEE APPENDIX C)	Vehicle body style	Undivided roads		Divided Roads	
		Run-off-the-road crashes	Hit bridge pier crashes	Run-off-the-road crashes	Hit bridge pier crashes
4	Beverage	0	0	2	0
5	Bob-Tail (includes tractor without trailer)	4	0	196	1
4	Dump	16	0	347	1
4	Fire Truck	0	0	15	0
6	Flatbed, lowboy, platform, float, stake	19	0	601	1
10	Livestock (includes 2-story)	2	0	33	1
5	Garbage	3	0	48	0
5	Mixer (concrete)	3	0	54	0
4	Motor Home or Motor Camper	3	0	36	1
2	Panel/small van (Good Time, etc.)	56	1	4113	13
2	Pickup	336	3	14627	87
9	Pole (log)	1	0	13	0
4	Refrigerator	1	0	115	0
2	Utility vehicle	138	2	8193	29
4	Tank (oil, gas, chemicals, milk)	9	0	212	0
6	Travelall/Carryall	0	0	2	0
4	Van (large, furniture, etc.)	19	1	1965	17
6	Wrecker	1	0	72	0
6	P/U w/camper	0	0	4	0
9	Oilfield equipment (usually special design)	0	0	0	0
11/12	All Other styles not listed above	0	0	72	1
99	Unknown	29	0	1606	7
	Total	**640**	**7**	**32326**	**159**

CRASH RISK ANALYSIS

The crash risk analysis was divided into two parts. The first part consisted of estimating crash risk using probability theories. For the second part, regression models were estimated to estimate the predicted number of run-off-the-road truck crashes and crashes hitting a bridge pier. As described above, the analysis described focuses on segment-based analysis. Also, as detailed above, all crashes involving pickup trucks, utility vehicles, and small vans were removed from Texas data for this part of the analysis.

Crash Probabilities

As discussed by Lord, Washington, and Ivan, the crash process can be represented using theoretical principles. A crash is, in theory, the result of a Bernoulli trial *(2)*. Each time a vehicle enters an intersection, a highway segment, or any other type of entity (a trial) on a given transportation network, it will either crash or not crash. For purposes of consistency a crash is termed a "success" while failure to crash is a "failure." For the Bernoulli trial, a random variable, defined as X, can be generated with the following probability model: if the outcome w is a particular event outcome (e.g., a crash), then $X(w) = 1$, whereas if the outcome is a failure, then $X(w) = 0$. Thus, the probability model becomes

X	1	0
P(x = X)	p	q

where p is the probability of success (a crash) and $q = (1-p)$ is the probability of failure (no crash).

In general, if there are N independent trials (vehicles passing through an intersection, road segment, etc.) that give rise to a Bernoulli distribution, then it is natural to consider the random variable Z that records the number of successes out of the N trials. Under the assumption that all trials are characterized by the same failure process (this assumption is revisited later in the paper), the appropriate probability model that accounts for a series of Bernoulli trials is known as the binomial distribution and is given as:

$$P(Z = n) = \binom{N}{n} p^n (1-p)^{N-n} \tag{5.1}$$

where $n = 0,1,2,\ldots,N$. In Equation (5.1), n is defined as the number of crashes or collisions (successes). The mean and variance of the binomial distribution are $E(Z) = Np$ and $VAR(Z) = Np(1-p)$, respectively.

For typical motor vehicle crashes where the event has a very low probability of occurrence and a large number of trials exist (e.g., million entering vehicles, vehicle-miles-traveled, etc.), it can be shown that the binomial distribution is approximated by a Poisson distribution. Under the Binomial distribution with parameters N and p, let $p = \lambda/N$, so that a

large sample size N will be offset by the diminution of p to produce a constant mean number of events λ for all values of p. Then as $N \rightarrow \infty$, it can be shown that

$$P(Z = n) = \binom{N}{n} \left(\frac{\lambda}{N}\right)^n \left(1 - \frac{\lambda}{N}\right)^{N-n} \cong \frac{\lambda^n}{n!} e^{-\lambda} \tag{5.2}$$

where, $n = 0,1,2,\ldots,N$ and λ is the mean of a Poisson distribution *(3)*.

The approximation illustrated in Equation (5.2) works well when the mean λ and p are assumed to be constant. In practice, however, it is not reasonable to assume that crash probabilities across drivers and across road segments (intersections, etc.) are constant. Specifically, each driver-vehicle combination is likely to have a probability p_i that is a function of driving experience, attentiveness, mental workload, risk adversity, vision, sobriety, reaction times, vehicle characteristics, etc. Furthermore, crash probabilities are likely to vary as a function of the complexity and traffic conditions of the transportation network (road segment, intersection, etc.). All these factors and others will affect to various degrees the individual risk of a crash.

Given the characteristics described above, it can be shown that Bernoulli trials with unequal probability of events lead to over-dispersion commonly observed in crash data *(4, 2)*. To capture this over-dispersion, transportation safety analysts commonly use regression methods involving the Poisson-gamma or negative binomial model, Poisson-lognormal model, or the recently introduced Conway-Maxwell-Poisson model *(5)*.

Using the theoretical principals described above, one can compute the risk for a truck to hit a bridge pier. This analysis was done for undivided and divided highways. It is important to point out that some important assumptions had to be made. For instance, the Truck Annual Average Daily Traffic (TAADT) values are estimates; the risk is the same for each truck, at least one truck ran-off-the-road. Furthermore, the probabilities do not account for the exposure associated with the number of bridge piers located on the sample network. The analysis only used information collected on truck crashes and traffic data.

The risk for a heavy vehicle to run-off-the-road can be estimated using the following equation:

P_{T_ROR} = the number of truck ROR crashes on one mile road section / the number of opportunities estimated from TAADT

The number of opportunities is estimated using the summation of all TAADT on the network for the 4-year time period. The total number of opportunities is estimated as follows:

$$4 \times \text{TAADT} \times 365.$$

The risk for a heavy vehicle to hit a bridge pier is estimated using the probability that the heavy vehicle first had to ROR and then hit a bridge pier. This is defined as a conditional probability:

$P_{HBP|T\_ROR}$ = the number of trucks hitting a bridge pier / the number of Trucks ROR crashes

Now the risk for a truck traveling on the highway to hit a bridge pier on the sample network is given using the relationship:

$$P_{HBP} = P_{HBP|T\_ROR} \times P_{T\_ROR} \qquad (5.3)$$

Table 5.17 summarizes the risk analysis for all divided and undivided highways in Texas. This table shows that a bridge pier is more likely to be hit on an undivided facility than on a divided facility. Also, if a truck leaves the traveled way, it is more likely to hit a bridge pier on an undivided highway than on a divided highway. A heavy vehicle is more likely to run-off-the-road on an undivided highway than on a divided highway.

Table 5.17. Crash Probability Analysis (Texas Data).

Variables	Undivided	Divided
Number of Sites	350	2486
Total Length (miles)	264.2	2862.9
ROR Crashes (4-year)	110	5393
Hit Bridge Pier Crashes (4-year)	1	30
Opportunities	$4.742*10^8$	$2.43*10^{10}$
P_{T_ROR}	$3.799*10^{-7}$	$2.986*10^{-7}$
$P_{HBP|T\_ROR}$	0.0091	0.0056
P_{HBP}	$3.457*10^{-9}$	$1.672*10^{-9}$

Table 5.18 tabulates the risk analysis for straight sections and horizontal curves on divided and undivided highways in Texas. The risk analysis was adjusted to account for the differences in segment lengths; horizontal curves are usually always shorter than tangent sections. Since there were no reported hit bridge pier heavy vehicle crashes on undivided horizontal curves and tangent sections, the crash probability was not developed for those roads. This table shows that a bridge pier is more likely to be hit on a horizontal curve than on a straight section. The tangent and curved sections on undivided roads have a higher risk of running off the road than the tangent sections on divided roads, but are safer than horizontal curves on divided sections. Also, if a truck leaves the traveled way, it is more likely to hit a bridge pier on a horizontal curve than on a straight section.

Table 5.18. Crash Probability Analysis on Tangent Sections and Horizontal Curves (Texas Data).

Severity	Undivided		Divided	
	Tangents	**Curves**	**Tangents**	**Curves**
Number of Sites	156	25	912	540
Total Length (miles)	64.2	6.9	707.8	161.4
ROR crashes (4yrs)	35	2	1422	541
Hit bridge pier crashes (4yrs)	0	0	5	3
Opportunities	$2.094*10^8$	$3.259*10^7$	$8.936*10^9$	$5.498*10^9$
P_{T_ROR}	$3.808*10^{-7}$	$3.462*10^{-7}$	$3.113*10^{-7}$	$3.970*10^{-7}$
P_{HBP/T_ROR}	--	--	0.0035	0.0055
P_{HBP}	--	--	$1.09*10^{-9}$	$2.18*10^{-9}$

Table 5.19 summarizes the risk analysis for divided and undivided highways. The analysis for undivided segments may not be reliable because of very few reported crashes.

Table 5.19. Crash Probability Analysis (Minnesota Data).

Variables	Undivided	Divided
Number of Sites	54	552
Total Length (miles)	26.8	555.3
ROR Crashes (5-year)	3	421
Hit Bridge Pier Crashes (5-year)	2	28
Opportunities	$6.637*10^7$	$2.697*10^9$
P_{T_ROR}	$2.03*10^{-8}$	$3.29*10^{-7}$
$P_{HBP|T\_ROR}$	0.67	0.067
P_{HBP}	$1.35*10^{-8}$	$2.19*10^{-8}$

The crash risk for a pier to be hit will be governed by the TAADT and is given as follows:

$$AF = TAADT \times P_{HBP} \times 365 \qquad (5.4)$$

where,

AF = Annual Frequency the bridge pier is hit;

P_{HBP} = the probability for a bridge pier to be hit by a heavy vehicle.

Regression Analysis

As discussed above, several statistical models were developed for estimating the expected number of truck crashes running-off-the-road and hitting bridge piers. To increase the sample mean, the light trucks in Texas data were included during the model development, but the intercept was later adjusted so that the regression models account for heavy trucks only. The probabilistic structure used for developing the models was the following. The number of crashes at the i-th segment, Y_i, when conditional on its mean μ_i, is assumed to be Poisson distributed and independent over all segments as *(6)*:

$$i = 1, 2, \ldots, I \tag{5.5}$$

The mean of the Poisson is structured as:

$$Y_i \mid \mu_i \sim Po(\mu_i) \tag{5.6}$$

It is usually assumed that $\exp(e_{it})$ is independent and Gamma distributed with a mean equal to 1 and a variance $1 / \phi$ for all i (with $\phi > 0$). With this characteristic, it can be shown that Y_i, conditional on $f(.)$ and ϕ, is distributed as a Negative Binomial (NB) (or Poisson-gamma) random variable with a mean $f(.)$ and a variance $f(.)(1 + f(.)/\phi)$, respectively. The term ϕ is usually defined as the "inverse dispersion parameter" for the NB distribution.

Usually the dispersion parameter ($\alpha = 1/\phi$) or its inverse (ϕ) is assumed to be fixed, but recent research in highway safety has shown that the inverse dispersion parameter could potentially be dependent on the covariates *(7, 8, 6, 2)*. For simplifying the model development, the models were estimated using a fixed dispersion parameter.

An important characteristic associated with the development of statistical relationships is the choice of the functional form linking crashes to the covariates. For this work, two functional forms were used. The first one, defined as a general AADT model, only includes traffic flow as a covariate. This functional is the most popular among transportation safety analysts since they are easy to recalibrate and because flow is often the significant variable associated with crashes *(9)*. The functional form is as follows and was only used for ROR crashes:

$$\mu_i = e^{\ln \beta_0} L_i F_i^{\beta_1} \tag{5.7}$$

where,
μ_i = the estimated number of crashes per year for site i;
F_i = vehicles per day (ADT) for segment i;
L_i = length of segment i in miles; and
$\beta_0, \beta_i, \ldots, \beta_n$ = estimated coefficients.

Table 5.20 summarizes the modeling results for the general TAADT models.

Table 5.20. General TAADT Regression Models for Run-off-the-Road Truck Crashes on All Segments (Texas Data).

Variables	Undivided	Divided
Constant ($\ln \beta_0$)	-6.089 (0.576)	-5.920 (0.231)
Flow (β_1)	0.595 (0.085)	0.636 (0.027)
Inverse Dispersion Parameter (ϕ)	1.013 (0.169)	0.921 (0.028)
-2 Log-likelihood	1090	17144
Deviance	347	2860
DOF	348	2484

Figure 5.1 illustrates the relationship between ROR truck crashes and truck AADT on all road sections. Table 5.21 summarizes the modeling results for the general TAADT models on straight sections and horizontal curves in Texas. Figure 5.2 gives the relationship between ROR truck crashes and truck AADT on tangent sections and horizontal curves.
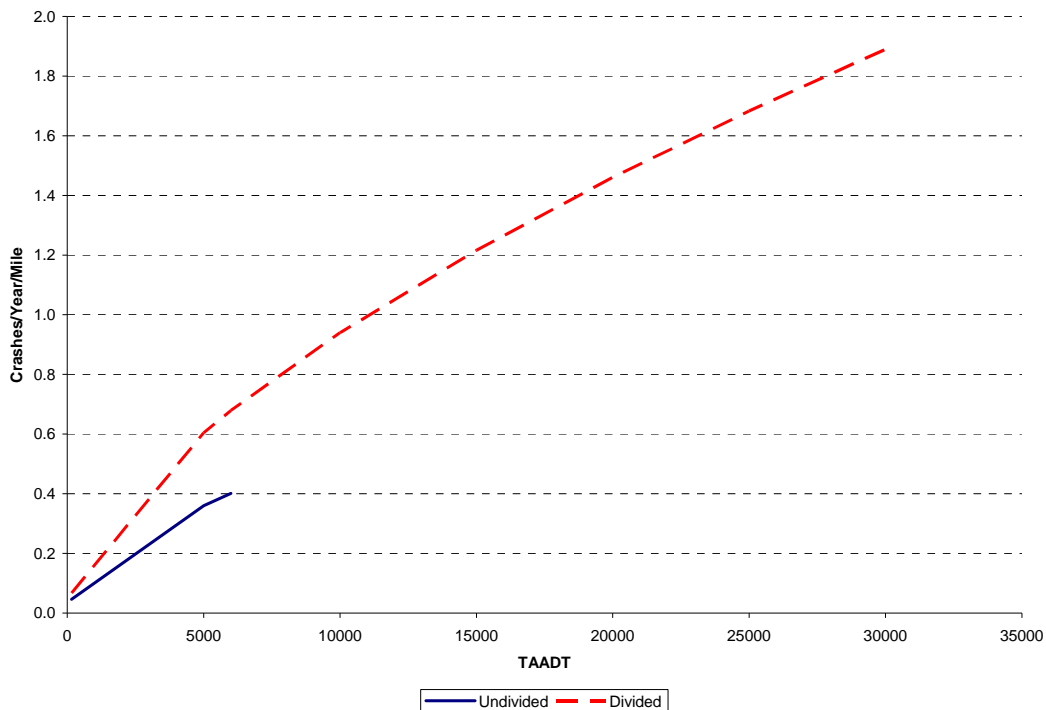
Figure 5.1. Relationship between Truck ROR Crashes and TAADT (Texas Data).

**Table 5.21. General TAADT Regression Models for
Run-off-the-Road Truck Crashes on Tangents and Horizontal Curves
(Texas Data).**

Variables	Undivided	Divided	
	Tangents	Tangents	Curves
Constant ($\ln \beta_0$)	-6.354 (0.923)	-4.676 (0.405)	-5.875 (0.597)
Flow (β_1)	0.645 (0.136)	0.501 (0.047)	0.669 (0.069)
Inverse Dispersion Parameter (ϕ)	0.943 (0.271)	0.767 (0.039)	0.774 (0.055)
-2 Log-likelihood Deviance DOF	405 140 154	5806 1046 910	3054 607 538
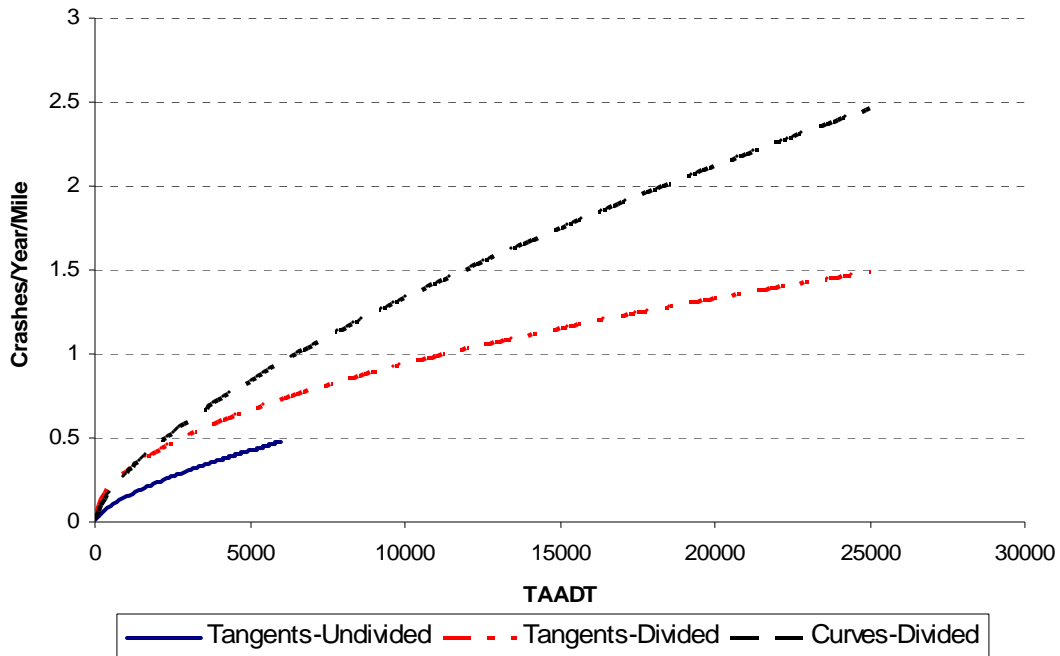
**Figure 5.2. Relationship between Truck ROR Crashes and TAADT on Tangent Sections
and Horizontal Curves (Texas Data).**

Table 5.22 summarizes the modeling results for the general TAADT models on divided segments in Minnesota. Figure 5.3 illustrates the relationship between ROR truck crashes and truck AADT on divided road sections in Minnesota. Per unit of exposure, the Texas model estimate more ROR crashes than the Minnesota model.

**Table 5.22. General TAADT Regression Models for Run-off-the-Road
Truck Crashes on Divided Segments (Minnesota Data).**

Variables	Estimates (Std Err)
Constant ($\ln \beta_0$)	-9.184 (0.902)
Flow (β_1)	0.919 (0.111)
Inverse Dispersion Parameter (ϕ)	2.157 (0.639)
-2 Log-likelihood	1067
Deviance	479
DOF	550
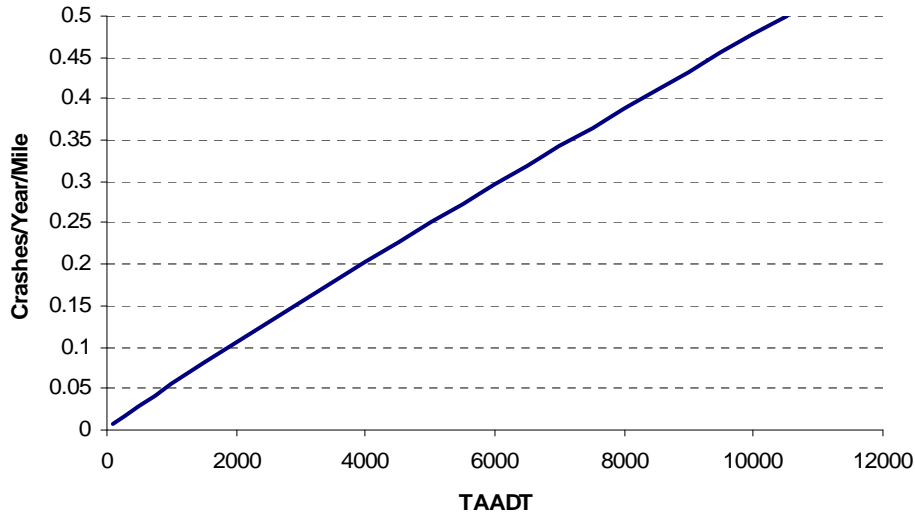
**Figure 5.3. Relationship between Truck ROR Crashes
and TAADT on Divided Segments (Minnesota Data).**

The second functional form models the covariates as a function of crash rate. Some researchers prefer this form to the one described above. The functional form is as follows:

$$\mu_i = \frac{L_i \times F_i \times 365}{1,000,000} e^{\ln \beta_0} e^{(\sum_{j=1}^{n} \beta_j X_j)}$$

(5.8)

where,

   μ_i = the estimated number of crashes per year for site i;

   F_i = vehicles per day (ADT) for segment i;

   L_i = length of segment i in miles;

x_i = a series of covariates; and

$\beta_0, \beta_i, \ldots, \beta_n$ = estimated coefficients.

The coefficients of the regression models were estimated using Statistical Analysis System (SAS) *(10)*. The generalized linear model (GENMOD) procedure in SAS estimates model coefficients using the maximum-likelihood method. Because of the low sample size issue, for some models, the dispersion parameter (or its inverse) was estimated using a weighted regression method *(11)*. The residual deviance statistics were used to assess the goodness-of-fit of the regression models. Only variables that were statistically significant at the 5-percent-level were included in the models. The coefficients were also evaluated for consistency to ensure the sign of each coefficient reflected previously observed crash characteristics. Table 5.23 summarizes the modeling results for the run-off-the-road and hit bridge pier crash rate models on all divided and undivided road sections in Texas.

Table 5.23. Crash Rate Regression Models for Run-off-the-Road and Hit Bridge Pier Truck Crashes on All Road Sections (Texas Data).

Variables	Undivided		Divided	
	ROR Crashes	**Hit Bridge Pier**	**ROR Crashes**	**Hit Bridge Pier**
Constant ($\ln \beta_0$)	0.038 (0.986)	-6.383 (0.601)	-0.638 (0.095)	-7.018 (0.113)
Average Lane Width (β_1)	-0.068 (0.079)	---	---	---
Average Shoulder Width (β_2) (both sides)	-0.031 (0.020)	---	---	---
Average Right Shoulder Width (β_3)	---	---	-0.061 (0.009)	---
Bridge Pier density (β_4)	---	0.210 (0.097)	---	0.098 (0.020)
Curve density (β_5)	0.037 (0.029)	---	0.061 (0.010)	---
Inverse Dispersion Parameter (ϕ)	1.017 (0.175)	0.122 (0.154)	0.911 (0.028)	0.429 (0.147)
-2 Log-likelihood Deviance DOF	1106 363 346	61 30 348	17234 2926 2483	1138 643 2484

Figure 5.4 shows the relationship between truck hitting a bridge pier as a function of truck AADT for segments having 1 bridge per mile and 3 bridges per mile, respectively. As discussed above, bridge piers are more frequently hit on undivided highways.

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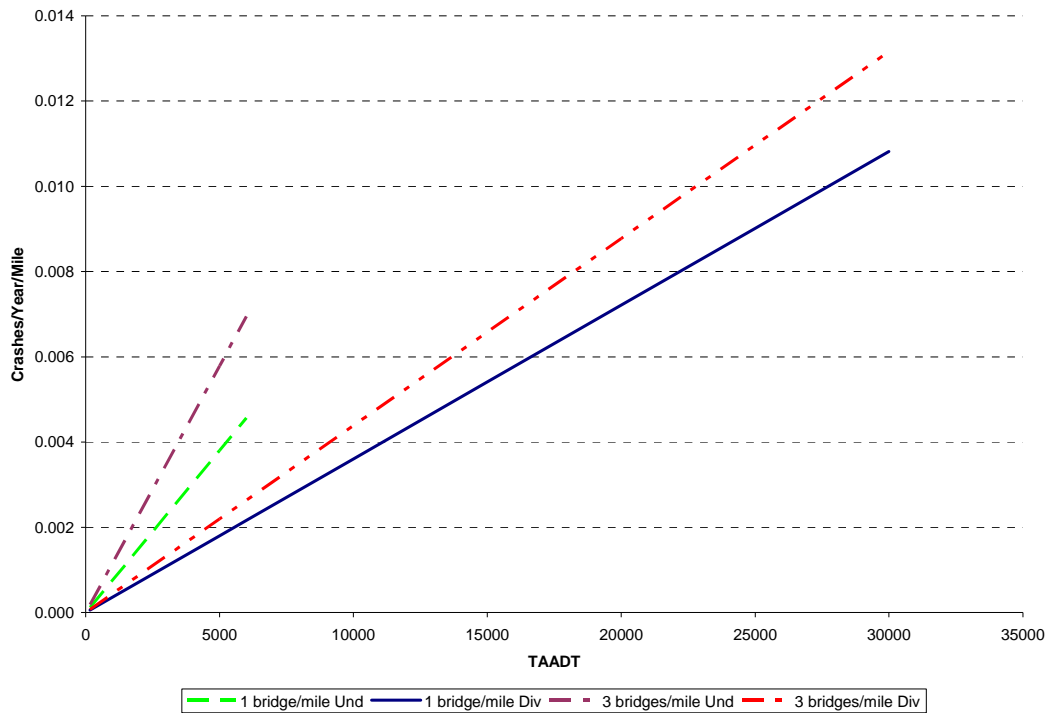
Figure 5.4. Relationship between Truck Hitting Bridge Pier Crashes and TAADT (Texas Data).

Table 5.24 summarizes the modeling results for the run-off-the-road crash models on tangent sections and horizontal curves. Because of small sample size and low sample mean, the ROR crash model for undivided horizontal curves and all hit bridge pier crash models could not be estimated. The estimates for hit bridge pier crashes can be calculated indirectly by multiplying the ROR crash estimates with the probability calculated in Table 5.18. As seen below, with the increase in the degree of curvature, the number of ROR crashes increases.

Figure 5.5 illustrates the relationship between ROR truck crashes and truck AADT with the change in degree of curvature for divided highways. As discussed above, with the increase in degree of curvature, the ROR crashes increase. The result is not surprising and has been documented elsewhere *(12)*.

Table 5.24. Crash Rate Regression Models for Run-off-the-Road Truck Crashes on Tangents and Horizontal Curves (Texas Data).

Variables	Undivided	Divided	
	Tangents	**Tangents**	**Curves**
Constant ($\ln \beta_0$)	-0.627 (0.210)	-0.022 (0.203)	-0.045 (0.259)
Average Shoulder Width (β_1) (both sides)	-0.037 (0.031)	---	---
Average Right Shoulder Width (β_2)	---	-0.102 (0.021)	-0.089 (0.026)
Degree of Curvature (β_3)	---	---	0.057 (0.028)
Inverse Dispersion Parameter (ϕ)	0.902 (0.260)	0.714 (0.037)	0.772 (0.055)
-2 Log-likelihood	410	5889	3055
Deviance	142	1072	606
DOF	154	910	537
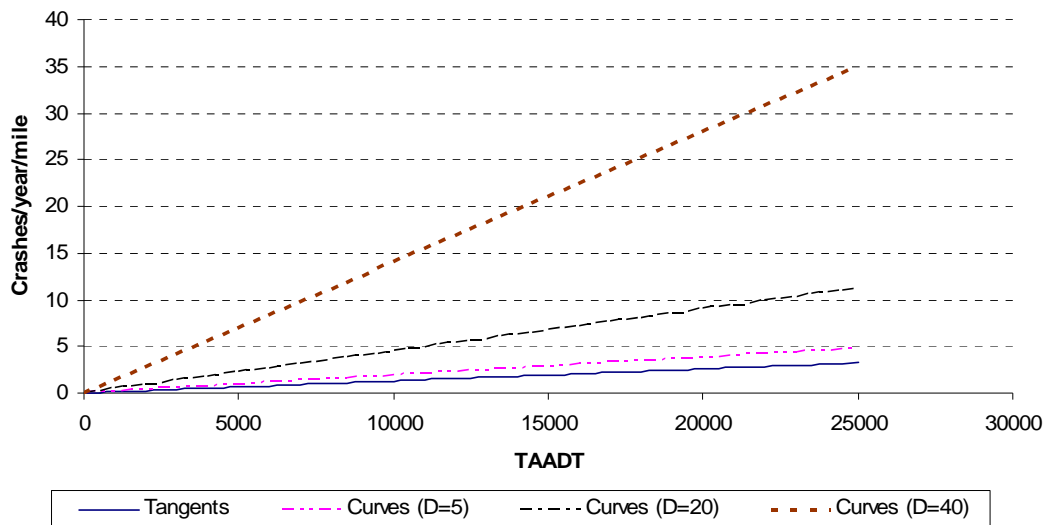
Figure 5.5. Relationship between ROR Crashes and TAADT with the Change in Degree of Curvature on Divided Segments (Texas Data).

Table 5.25 summarizes the modeling results for the run-off-the-road and hit bridge pier crash models on all divided road sections in Minnesota. The functional form used here is as follows:

$$\mu_i = e^{\ln \beta_0} L_i F_i^{\beta_1} e^{(\sum_{j=1}^{n} \beta_j X_j)} \tag{5.9}$$

where,

μ_i = the estimated number of crashes per year for site i;

F_i = vehicles per day (ADT) for segment i;

L_i = length of segment i in miles;

X_j = a series of covariates; and

$\beta_0, \beta_i, \ldots, \beta_n$ = estimated coefficients.

Table 5.25. Crash Regression Models for Run-off-the-Road and Hit Bridge Pier Truck Crashes on Divided Road Sections (Minnesota Data).

Variables	ROR Crashes	Hit Bridge Pier
Constant ($\ln \beta_0$)	-8.414 (1.718)	-14.114 (3.294)
Flow (β_1)	0.943 (0.112)	1.209 (0.401)
Average Lane Width (β_2)	-0.02 (0.103)	---
Average Right Shoulder Width (β_3)	-0.076 (0.035)	---
Bridge Pier Density (β_4)	---	0.011 (0.006)
Inverse Dispersion Parameter (ϕ)	2.187 (0.639)	0.248 (0.204)
-2 Log-likelihood	1063	230
Deviance	477	124
DOF	549	549

Figure 5.6 shows the relationship between truck hitting a bridge pier as a function of truck AADT for divided segments in Minnesota. This figure shows the crash risk as a function of bridge pier density. As opposed to the Texas model, the number of bridge piers per mile has a minimal effect on the total number of truck crashes.
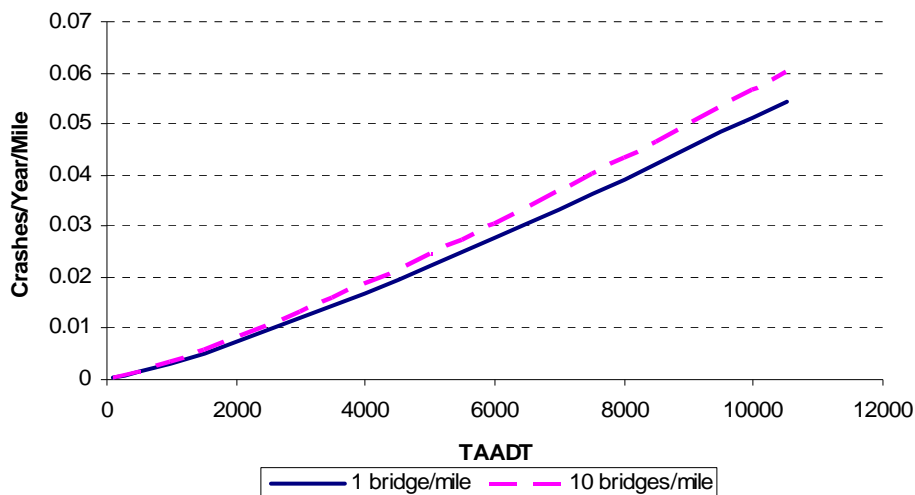
Figure 5.6. Relationship between Truck Hitting Bridge Pier Crashes and TAADT (Minnesota Data).

The models above could be used to estimate the crash risk when highway segments are analyszed.

EMPIRICAL BAYES METHOD

The EB method can be used for refining the safety estimates (i.e., the long-term mean) of a given site. This method has now become the standard approach for conducting safety analyzes. The EB method takes into account crashes that ocurred at the given site and the safety performance of sites having similar characteristics *(13)*. This method can be used for identifying hazardous sites (or sites with promise) *(14, 15)*, evaluating the safety effects of interventions, or assessing the potential safety benefits due to site improvements *(13)*.

The EB estimate for site *i* over a period *t* can be estimated using the following equation *(9)*:

$$\hat{\hat{\mu}}_{it} = (1 - \omega_{it}) y_{it} + \omega_{it} \hat{\mu}_{it} \tag{5.10}$$

where,

$\hat{\hat{\mu}}_{it}$ = EB estimate in crashes per year for given site *i* and year *t*;

ω_{it} = weight factor for given site *i* and year *t*;

y_{it} = observed number of crashes for given site *i* and year *t*;

$\hat{\mu}_{it}$ = the estimated number of crashes by crash prediction models for given site *i* and, year *t* (usually estimated using a NB model).

The weight factor ω_{it} is given as follows:

$$\omega_{it} = 1/(1 + \hat{\mu}_{it} / \phi) \tag{5.11}$$

where,

ϕ = the inverse dispersion parameter for the given dataset (note: in the safety literature, analysts sometimes report the dispersion parameter $\alpha = 1/\phi$). This value is given by SAS.

APPLICATION OF METHODOLOGY

This section describes two example problems illustrating the application of analysis procedures. The first example covers the crash risk analysis when a new bridge is constructed on an existing freeway. The second example describes the comparison of the hit brige pier crash risk between two corridors.

Example 1: Crash Risk Estimate for an Individual Bridge

Due to an increased in residential activities located in a community located in the eastern part of Texas, an overpass is planned to be constructed on top of IH-10. At that location, the present TAADT is 10,000 vehicles/day. The highway segment has four lanes. The typical lane width is 12 ft, and the right shoulder width is equal to 10 ft. Both traveled ways are separated by a 40-ft median.

Using the values found in Table 5.17, the probability for a truck to hit bridge pier (P_{HBP}) on a divided highway is estimated to be $1.672*10^{-9}$. The annual frequency (AF) the bridge pier is hit can be calculated using Equation (5.4):

$$AF = TAADT \times P_{HBP} \times 365$$
$$AF = 10,000 \times 1.672 \times 10^{-9} \times 365 = 0.0061 \text{ crashes/year.}$$

This value means that a pier on this bridge may be hit about once every 164 years, if we assume that every factors, such as the number of lanes and vehicular traffic, remain constant.

Example 2: Crash Risk Estimate for Corridor Study

Due to a train derailment, a bridge spanning on top of that railway has been damaged. With temporary stabilization procedures, the bridge can still be used by passenger cars as well as light trucks until a new bridge is built. Due to current legal actions, the new bridge is not expected to be completed for another three years. During this time period, the state transportation agency will have to re-route heavy vehicles to another highway located within the vicinity of the damaged bridge. The alternative route is a four-lane undivided highway that is about 10 miles in length. The bridge density is 2 bridges per mile. One bridge pier has been hit over the last five years on this alternative route by a heavy truck. The alternative route's truck average annual daily traffic is 7,000 vehicles per day. By re-routing, the heavy vehicle's traffic on this route is increased to 12,000 vehicles per day. The agency would like to know what is the increased risk for bridge piers to be hit given the anticipated augmentation in TAADT traffic over the next three years. This assessment will help the agency decide whether additional measures are needed to protect bridges along that route.

Step 1: Calculate the crash risk on the alternative route with existing traffic.

Using Equation (5.8) and Table 5.23, the expected hit bridge crashes is given as:

$$\mu_i = \frac{F_i \times L_i \times 365}{1,000,000} \beta_0 e^{\sum_{i=1}^{n} x_i \beta_i}$$

Here F_i is the TAADT, which will be 7000 vehicles per day.

$$\hat{\mu} = \frac{7000 \times 10 \times 365}{1,000,000} e^{-6.383} \times e^{0.210*2} = 0.0661 \text{ crashes/year.}$$

Thus the predicted frequency of a heavy truck to hit a bridge pier is 0.0661 crashes/year. Over the last five years, the predicted crashes would be 0.113*5= 0.331 crashes (for the 5-year period).

Step 2: Calculate the EB estimate with the existing traffic.

Using Equation (5.10), the EB estimate is given as:

$$\hat{\mu}_{it} = (1 - \omega_{it}) y_{it} + \omega_{it} \hat{\mu}_{it}$$

The weight factor ω_{it} in Equation (5.11) is given as follows:

$$\omega_{it} = 1/(1 + \hat{\mu}_{it} / \phi)$$
$$\omega_{it} = 1/(1 + (0.331/0.122)) = 0.269$$

The EB estimate for hit bridge crashes over the last five years is:

$$\hat{\mu}_{it} = (1 - 0.269)*1 + 0.269 \times 0.567 = 0.884$$

Thus the EB estimate is 0.884/5= 0.177 crashes/year.

Step 3: Calculate the EB estimate on the alternative route with the new and existing traffic.

Assuming that all the factors remains constant, the EB estimate for heavy truck hit bridge pier crashes in the next year is given as:

$$\hat{\mu}_{it} = \frac{12000}{7000} \times 0.177 = 0.303 \text{ crashes/year}$$

Thus, we can expect a hit bridge pier crash by heavy truck in the next three years on this route (0.909 crashes in the next three years).

SUMMARY

This chapter described the crash risk analysis of the heavy vehicle run-off-the-road and hitting bridge pier crashes. The document was divided into five sections. The first section described about the process used for collecting the Texas and Minnesota data. The data for Texas were provided by DPS and TxDOT and contained information about the location bridges crossing over the sample network. The crash and network data for the State of Minnesota were

provided by the FHWA for the crash and roadway inventory data (HSIS) and the MnDOT for the location of bridges.

The second section provided important summary statistics about the geometric, operational, and crash data for undivided and divided highways in Texas and Minnesota. Separate statistics were also provided for tangent sections and horizontal curves in Texas. The geometric data include segment length, lanewidth, shoulder width, median width, and number of curves, among others. The summary statistics for the average annual daily traffic and estimated truck average annual daily traffic were provided.

The third section described the methodology for estimating the risk of a heavy vehicle to hit a bridge pier. The methodology was separated into two parts. The first part focused on the individual risk of a bridge pier to be hit by a truck. This part of the methodology is very similar to the risk analysis proposed AAHSTO for bridge piers located on waterways. The crash probability analysis using the Texas data showed that the undivided segments have higher risk for a truck to run-off-the-road than for divided segments. Also, tangent sections are safer than horizontal curves for undivided highway segments. The second part focused on developing the regression models for heavy vehicle running-off-the-road and hit bridge pier crashes. Separate models were developed for undivided and divided roads, and as well for the straight sections and horizontal curves. Initially, models were developed with truck flow as the only variable. Later on, the models were developed with different variables that are known to influence the running-off-the road and hit bridge pier crashes.

The fourth section described how the EB method can be used to improve the precision of estimates of a given site. The EB method can be used with the models described in the fourth section. Finally, two examples were provided to describe how the risk analysis can be used for individual sites and corridor studies.

CHAPTER 6. TEST PLAN FOR PHASE 2

For this project, truck-to-pier collisions that have occurred in recent years have been investigated and finite element computer simulations have been performed to develop information about forces generated on piers. Some understanding of the phenomenon and the range of force magnitudes has been developed. There is a need to supplement that understanding with physical testing. Several design concepts for full-scale testing were developed for this project. These concepts were reviewed by the project panel in a meeting at Texas Transportation Insitute on April 14, 2009. A brief description of each design concept is presented as follows.

CONCEPT 1 – SINGLE 30-INCH DIAMETER WITH BRACE

For this concept, a single 30-inch diameter pier will be constructed 20 ft above grade. The pier will be supported by a 30-inch diameter drilled shaft embedded 20 ft below grade. Reinforcement in the drilled shaft and pier will consist of 16 #9 vertical reinforcing bars evenly spaced within #3 rebar spiral with a 6-inch pitch. A steel tubing brace (HSS12x12) will be attached to the top of the pier and will be supported at grade by a 36-inch diameter drilled shaft located approximately 20 ft from the center of the pier. This drilled shaft will be embedded approximately 15 ft below grade. Reinforcement in this drilled shaft will consist of 18 #9 vertical reinforcing steel bars constructed within #3 spiral stirrups with a 6-inch pitch. Please refer to the drawings labeled Concept 1 (Figures 6.1 and 6.2) for additional information.

CONCEPT 2 – SINGLE 54-INCH DIAMETER WITH BRACE

For this concept, a single 54-inch diameter pier will be constructed 20 ft above grade. The pier will be supported by a 54-inch diameter drilled shaft embedded 20 ft below grade. Reinforcement in the drilled shaft and pier will consist of 24 #9 vertical reinforcing bars evenly spaced within a #3 rebar spiral with a 6-inch pitch. A steel tubing brace (HSS12x12) will be attached to the top of the pier and will be supported at grade by a 48-inch diameter drilled shaft located approximately 21 ft from the center of the pier. This drilled shaft will be embedded approximately 20 ft below grade. Reinforcement in this drilled shaft will consist of 22 #9 vertical reinforcing steel bars constructed within #3 spiral stirrups with a 6-inch pitch. Please refer to the drawings labeled Concept 2 (Figures 6.3 and 6.4) for additional information.

CONCEPT 3 – RETROFIT WALL DESIGN BETWEEN TWO 30-INCH BRIDGE PIERS IN 2-PIER BENT

For this concept, two 30-inch diameter piers will be constructed 20 ft above grade. The piers will be constructed 24 ft on centers. Each pier will be supported by a 30-inch diameter drilled shaft embedded 20 ft below grade. Reinforcement in the drilled shafts and piers will consist of 16 #9 vertical reinforcing bars evenly spaced within a #3 rebar spiral with a 6-inch pitch. The piers will be structurally connected using a steel strut at the top of the piers and a

concrete wall constructed between the piers at grade. The steel tube strut (HSS12x12) will be rigidly connected to the top of each of the piers. A 30-inch wide by 48-inch high concrete wall will be constructed between the piers at grade. This concrete wall will serve to provide additional structural resistance to vehicular impacts on the 30-inch diameter pier. Longitudinal reinforcement in the concrete wall will be doweled into the sides of the piers. Transverse stirrup reinforcement will be constructed in the concrete wall. Please refer to the drawings labeled Concept 3 (Figures 6.5 and 6.6) for additional information.

CONCEPT 4 – INSTRUMENTED PIER FOR MEASURING COLLISION FORCES

This device, shown in Figure 6.7, is designed to measure impacting force from the large truck using instrumented load cells with strain gages. Force data measured from these strain gages will be more accurate than data obtained from vehicle mounted accelerometers. The instrumented pier will be supported by a support frame. This support frame will be designed to resist the impact loads applied to the pier.

SELECTED CONCEPT

These concepts for a test pier were reviewed and discussed during a project panel meeting on April 14, 2009. The panel selected Concept 4 for Phase 2 testing. A detailed design of this concept will be developed during Phase 2 of this project.

FULL-SCALE CRASH TESTS

Two full-scale crash tests with a tractor-trailer and deformable cargo are planned for phase 2 of this study. The first test will be performed using a tractor-trailer weighing 80,000 lb and impacting the instrumented pier at 50 mph. Parameters for the second test will be set after the first test is performed.
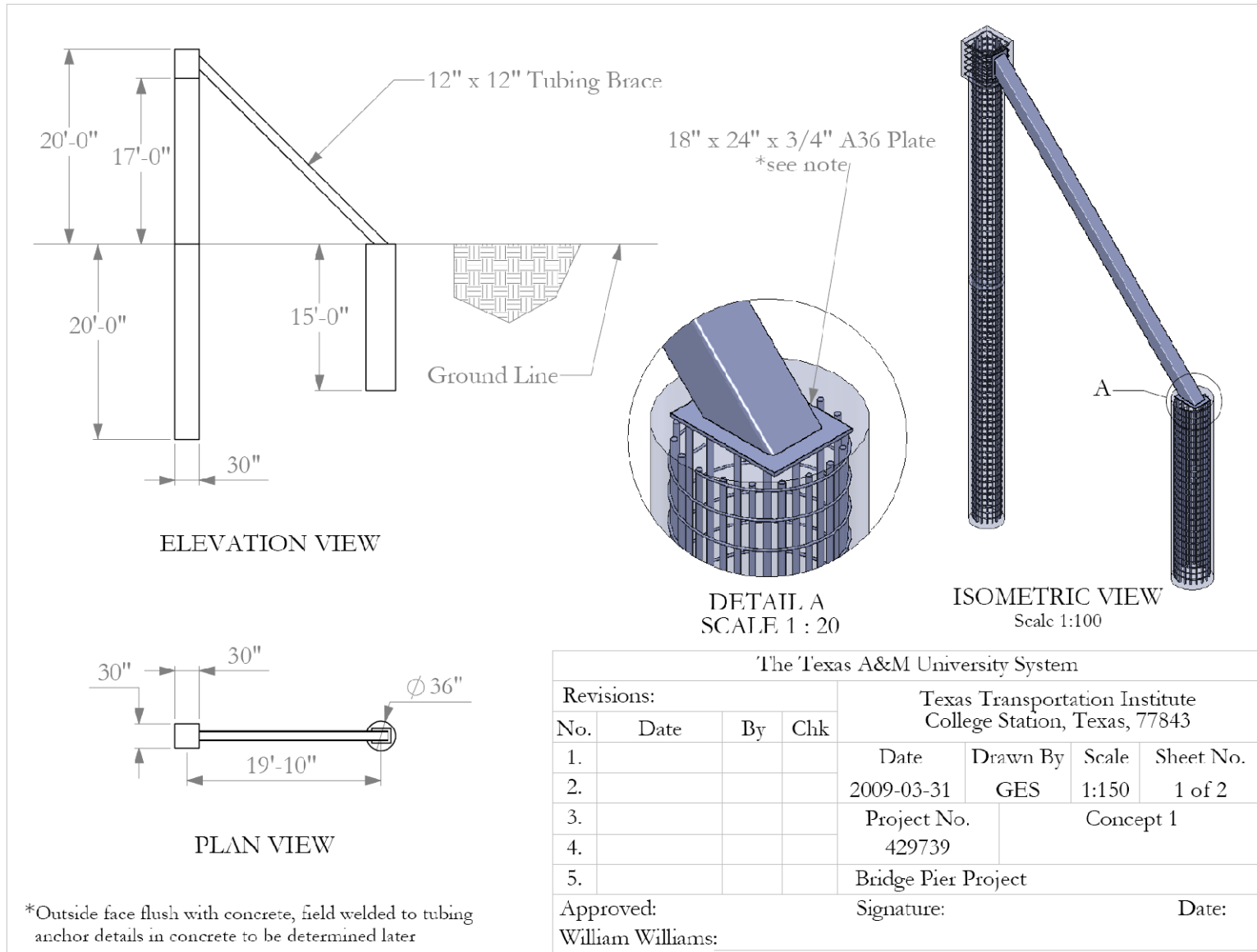
Figure 6.1. Concept 1 – Construction Details.

ELEVATION VIEW COLUMN

36"

40'-0"

39'-6"

ELEVATION VIEW PIER

15'-0"

14'-6"

PLAN VIEW COLUMN

Ø 30"

#9 x 16, evenly spaced

#3 rebar spiral, 6" pitch

30"

30"

3 3/16" ℄

PLAN VIEW PIER

Ø 36"

#3 rebar spiral, 6" pitch

#9 x 18, evenly spaced

3 3/16" ℄

*Plan Views Scale 1:20

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1.				Date	Drawn By	Scale	Sheet No.
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3.				Project No.		Concept 1 - Rebar Details	
4.				429739			
5.				Bridge Pier Project			

Figure 6.2. Concept 1 – Rebar Details.

54"

20'-0"

17'-0"

12" x 12" Tubing Brace

18" x 24" x 3'4" A36 Plate
*see note

20'-0"

20'-0"

Ground Line

48"

A

ELEVATION VIEW

DETAIL A
SCALE 1 : 30

ISOMETRIC VIEW
Scale 1:100

Ø 54"

54"

Ø 48"

54"

20'-8"

PLAN VIEW

*Outside face flush with concrete, field welded to tubing.
 Anchor details in concrete to be determined later

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Approved:				Signature:		Date:
William Williams:						

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Figure 6.3. Concept 2 – Construction Details.

40'-0"

39'-6"

**ELEVATION VIEW
COLUMN**

20'-0"

19'-6"

**ELEVATION VIEW
PIER**

⌀ 54"

#9 x 24, evenly spaced

#3 rebar spiral, 6" pitch

$3\frac{3}{16}$" ℄

#9 x 22, evenly spaced

**PLAN VIEW
COLUMN**

⌀ 48"

$3\frac{3}{16}$" ℄

**PLAN VIEW
PIER**

Plan Views
Scale 1:25

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Figure 6.4. Concept 2 – Rebar Details.

12" x 12" Tubing Brace

30"

30"

18" x 18" x 3/4" A36 Plate
*see note

20' 0"

17'-0"

30" wide concrete wall

48"

20'-0"

Ground Line

24'-0"

PLAN VIEW

DETAIL A
SCALE 1 : 20

A

ISOMETRIC VIEW

*Outside face flush with concrete, field welded to tubing.
Anchor details in concrete to be determined later.

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3.			Project No.		Concept 3	
4.			429739			
5.			Bridge Pier Project			

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Figure 6.5. Concept 3 – Construction Details.

ELEVATION VIEW
COLUMNS

40'-0"
39'-6"
37'-0"

Ø 1 1/8" x 16 evenly spaced

#3 rebar spiral, 6" pitch

30"
Ø 30"
30"
3 3/16" ℄

PLAN VIEW
COLUMNS
Scale 1:20

30"
27"
4"
1 3/4" ℄
1 3/4" ℄
8" TYP.
#4 stirrups @ 12"
1 3/4" ℄
1 3/4" ℄
48"
45"
4"

END VIEW
WALL
Scale 1:15

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3.				Project No. · Concept 3 - Rebar Details
4.				429739
5.				Bridge Pier Project

Figure 6.6. Concept 3 – Rebar Details.

14'-6"

Very Strong
Structural Member

Load Cell

∅ 30"

20'-0"

17'-0"

14'-6"

Heavy Wall Steel Tube

Load Cell

Ground Line

20'-0"

12'-0"

30"   30"

ELEVATION VIEW

30"

14'-6"

PLAN VIEW

30"

Load Cell

20'-0"

Heavy Wall
Steel Tube

Ground Line

20' 0"

SIDE VIEW

Note: Field Wielding, Load cell, Anchor Details
to be determined later.

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3.				Project No. 429739 — Sheet1
4.				
5.				Bridge Pier Project
Approved: Dr. Buth:				Signature: — Date:

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Figure 6.7. Concept 4 – Simulated Pier for Measuring Collision Load.

Very Strong
Structural
Member

Heavy Wall
Steel Tube

A

Heavy Wall
Steel Tube

Very Strong
Structural
Member

Load Cell

Very Strong
Structural
Member

DETAIL A
SCALE 1 : 40

20'-0"

3D View

Note: Field Wielding, Load cell, Anchor Details
to be determined later.

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3.				Project No.		Sheet2	
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5.				Bridge Pier Project			

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Figure 6.7. Concept 4 – Simulated Pier for Measuring Collision Load (Continued).

CHAPTER 7. DISCUSSION

The current AASHTO *LRFD Bridge Design Specifications* require that "abutments and piers located within a distance of 30.0 ft of the edge of the roadway, or within a distance of 50.0 ft to the centerline of a railway track, shall be designed for an equivalent static force of 400 kip . . ." Further guidance is not given to the designer. Also, detailed warrants for application of this requirement are not stated.

The objective of this effort is to address warrants for application of this requirement and the validity of magnitude of the design force. Work performed in this portion of the project included an investigation of collisions of trucks with bridge piers that occurred on the highway, finite element analyses of truck collisions with bridge piers, and a formulation of a methodology for estimating the risk of a truck colliding with a bridge pier.

Nineteen accidents involving trucks colliding with bridge piers were investigated and are reported. Several accidents resulted in partial or complete structural failure of the pier. Failure mechanisms consisted of two shear failure planes – one extending upward from the applied load at approximately 45 degrees and the other extending downward at approximately 45 degrees.

Finite element analyses of trucks colliding with bridge piers were performed using the LS-DYNA computer program. Parameters investigated included type of truck (65,000-lb SUT and 80,000-lb tractor-trailer), type of cargo (deformable and rigid), impact speed (40, 50, and 60 mph), and diameter of pier (24, 36, and 48 inches). The analyses indicate that, within the range of parameters studied, forces imposed on a pier can be much higher than 400 kips and that the magnitude of force is highly dependent on the cargo type (deformable or rigid). As expected, higher impact speeds generate higher forces. The effect of pier diameter on magnitude of force was not strong.

Results of research reported herein and other research reviewed indicate that collision forces generated on an assumed rigid bridge pier during a collision be a truck traveling at usual highway speeds is strongly dependent on structure of the vehicle and properties of payload being carried. For typical trucks with soft, deformable payloads, forces generated are expected to be less than 1000 kips. For more rigid payloads, short duration dynamic forces can be as high as 2500 to 3000 kips.

A methodology for estimating the risk for a heavy vehicle to leave the traveled-way and hit a bridge pier is presented. The methodology is divided into two components: crash risk analysis and regression models. Crash and highway network data for the States of Texas and Minnesota were used for developing the methodology. The data collected in this project included average annual daily traffic, estimated truck average annual daily traffic, segment length, lanewidth, shoulder width, median width, and the number of curves, among others. The first component of the methodology is very similar to the risk analysis tools proposed by AASHTO for bridge piers located on waterways. The crash probability analysis using the Texas data showed that undivided segments have higher risk for a truck to run-off-the-road than for divided segments. Also, tangent sections experienced less run-off-the-road crashes than

horizontal curves for undivided highway segments. The second component focused on developing the regression models for heavy vehicle running-off-the-road and hit bridge pier crashes. Separate models were developed for undivided and divided roads, as well for the straight tangent sections and horizontal curves. Initially, models were developed with truck flow as the only variable. Subsequently, additional models were developed with different variables that are known to influence running-off-the road and hit bridge pier crashes. Finally, two examples are provided to describe how the methodology can be used for individual sites and corridor studies.

The researchers recommended four concepts for a test pier were developed and reviewed by the project panel. A load measuring test pier was recommended by the researchers and selected by the panel for use in full-scale truck crash tests in Phase 2 of this study.

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APPENDIX A. SHEAR CAPACITY CALCULATIONS

ACCIDENT #1: FM 2110 OVER I-30, TEXARKANA, TX

Texas Transportation Institute
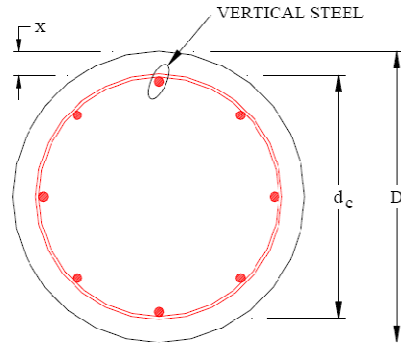
LRFD Section 5

Project #: __424977__

Sponsor: __TxDOT__

Subject: __Circular Concrete Shaft Shear Capacity - Accident #1 FM 2110 over I-30, Texarkana, TX__

1.) Given the following Design Data:

$$f_c := \begin{pmatrix} 3.05 \\ 4.00 \end{pmatrix} \cdot ksi$$
Compressive Strength of Concrete (ksi)

$$f_{yt} := 40 ksi$$
yield strength of spiral reinforcement (ksi)

Diameter of Column Impacted:

$$D := 30 in$$

Shear Reinforcement size:

$$Stirrup_{size} := \frac{2}{8} in$$

$$A_V := \pi \cdot \left(Stirrup_{size}^2 \right) \cdot 0.25 \cdot 2$$

$$A_V = 0.098 \cdot in^2 \qquad \text{Area 2 Legs}$$

Size of Vertical Steel (8~#9's)

$$Vertical_{size} := 1.128 in \qquad \text{Dia. of Longitudinal Steel (in.)}$$

$$X := 2.25 in \qquad \text{Distance to Stirrup Center (in.)}$$

$$d_c := D - 2 \cdot X \qquad \text{Diameter of Spiral Steel}$$

$$d_c = 25.5 \cdot in \qquad \text{Diameter of Spiral Steel (in.)}$$

$$psi = \frac{lbf}{in^2} \qquad ksi = \frac{kip}{in^2} \qquad kips \equiv 1000 lbf$$

VERTICAL STEEL

COLUMN SECTION

COLUMN SIDE VIEW

97

![Texas Transportation Institute logo] **LRFD Section 5**

Subject: __Circular Concrete Shaft Shear Capacity - Accident #1 FM 2110 over I-30, Texarkana, TX__

$s_{spa} := 6in$ Pitch in Spiral Stirrup

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

$b_V := D$ Width of section
(see Figure C5.8.2.9-3 page 5-57)

$D_r := D - X \cdot 2 - Stirrup_{size} - Vertical_{size}$
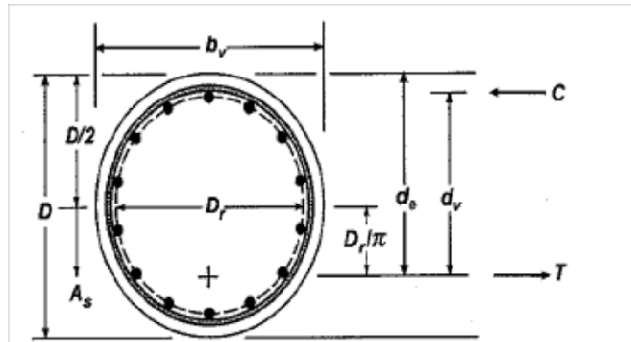
$D_r = 24.122 \cdot in$

Figure C5.8.2.9-2 Illustration of Terms b_v, d_v and d_e for Circular Sections.

$d_e := \dfrac{D}{2} + \dfrac{D_r}{\pi}$ $d_e = 22.678 \cdot in$ LRFD C5.8.2.9-2, pg. 5-67 Interim 2008

$d_V := 0.90 \cdot d_e$ see section commentary pag. 5-67   (LRFD C5.8.2.9)

$d_V = 20.41 \cdot in$

$a_g := 2in$ Maximum Aggregate size in Concrete

Subject: __Circular Concrete Shaft Shear Capacity - Accident #1 FM 2110 over I-30, Texarkana, TX__

2.) Calculate the minimum transverse reinforcement as per Section 5.8.2.5-1 pg. 5-63 Interim 2008

$$A_{transneeded} := 0.0316 \sqrt{\frac{f_c}{ksi}} \cdot ksi \cdot b_v \cdot \frac{s_{spa}}{f_{yt}}$$

$$A_{transneeded} = \begin{pmatrix} 0.2483 \\ 0.2844 \end{pmatrix} \cdot in^2 \qquad A_v = 0.098 \cdot in^2 \qquad \text{Actual Area of Transverse Steel Provided} \; (in^2)$$

3.) Determine β & θ as per LRFD Section 5.8.3.4.2, Interim 2008, page 5-72

Calculate Crack Spacing Parameter sxe as per LRFD Eq. 5.8.3.4.2-5, Interim 2008, page 5-72

$$s_x := d_v \qquad \text{LRFD 5.8.3.4.2, Interim 2008, pg. 5-73}$$

$$s_{xe} := s_x \cdot \left(\frac{1.38in}{a_g + 0.63in} \right) \qquad s_{xe} = 10.71 \cdot in$$

$$\varepsilon_s := 0.006 \qquad \text{LRFD Section 5.8.3.4.2, pg. 5-74 Interim 2008, based on maximum strain from severe impact condition}$$

Therefore, Calculate β:

$$\beta := \frac{4.8}{\left(1 + 750 \cdot \varepsilon_s\right)} \cdot \frac{51}{\left(39 + \frac{s_{xe}}{in}\right)} \qquad \beta = 0.895 \qquad \text{LRFD Eq. 5.8.3.4.2-2, Interim 2008, pg. 5-72}$$

$$\theta := \left(29 + 3500 \cdot \varepsilon_s\right) deg \qquad \theta = 50 \cdot deg \qquad \text{LRFD Eq. 5.8.3.4.2-3, Interim 2008, pg 5-72}$$

Subject: __Circular Concrete Shaft Shear Capacity - Accident #1 FM 2110 over I-30, Texarkana, TX__

4.) Calculate V_c ~ Strength Attributable to Concrete as per Section 5.8.3.3 pg. 5-61:

with:

$f_c = \begin{pmatrix} 3.05 \\ 4 \end{pmatrix} \cdot ksi$ Compressive strength of column concrete (psi)

$D = 30 \cdot in$ $b_v = 30 \cdot in$

$d_v = 20.41 \cdot in$

$$V_c := \left[(0.0316 \cdot \beta) \cdot \sqrt{\frac{f_c}{ksi}} \cdot ksi \cdot b_v \cdot d_v \right]$$

See Equation 5.8.3.3-3, pg. 5-70, Interim 2008.. for 1 shear plane

$D = 30 \cdot in$ $f_c = \begin{pmatrix} 3.05 \\ 4 \end{pmatrix} \cdot ksi$ $V_c = \begin{pmatrix} 30.256 \\ 34.65 \end{pmatrix} \cdot kips$ Nominal shear strength of concrete alone for corresponding column Dia.

5.) Calculate V_s ~ Strength Attributable to Shear Reinforcement:

$Stirrup_{size} = 0.25 \cdot in$ Dia. of Stirrup Steel (in.)

$A_v = 0.098 \cdot in^2$ Area of Stirrup Steel (in^2) .... 2 Legs

$s_{spa} = 6 \cdot in$ Spiral spacing / pitch (inches)

Calculate the Stirrup Angle in degrees:

Stirrup Angles (degrees)

$\alpha := atan\left(\left(\frac{s_{spa}}{\pi \cdot d_c} \right) \right)$ $\alpha = 4.283 \cdot deg$

Subject: __Circular Concrete Shaft Shear Capacity - Accident #1 FM 2110 over I-30, Texarkana, TX__

$$V_s := \frac{A_v \cdot f_{yt} \cdot d_v \cdot (\cot(\theta) + \cot(\alpha)) \cdot \sin(\alpha)}{s_{spa}}$$

LFRD 5.8.3.3-4

$V_s = 14.158 \cdot kips$ $D = 30 \cdot in$

$V_s = 14.158 \cdot kips$ Nominal shear strength as per Section 5.8.3.3-4 pg. 5-61

6.) Calculate the Nominal Shear Capacity of Column for two Failure Plane Mechanism considering the strength of the concrete and the spiral reinforcing steel:

Concrete Shear Strength
(single shear plane)

Spiral Reinforcment Strength
(single shear plane)

$$V_c = \begin{pmatrix} 30.256 \\ 34.65 \end{pmatrix} \cdot kips$$

$$V_s = 14.158 \cdot kips$$

$\phi_v := 0.9$ (LRFD 5.5.4.2.1, Interm 2008, pg. 5-25

$$V_r := (V_c + V_s) \cdot \phi_v \cdot 2$$ LRFD Eq. 5.8.3.3-1, Interim 2008, pg. 5-70, Shear Resistance 2 Shear planes

Column Dia.

Nominal Shear Capacity of
Column (kips)

$D = 30 \cdot in$

$$V_r = \begin{pmatrix} 79.947 \\ 87.855 \end{pmatrix} \cdot kips$$

3050 psi Concrete

4000 psi Concrete

ACCIDENT #2: BRIDGE AT MILE POST 232 ON IH-45

Texas Transportation Institute
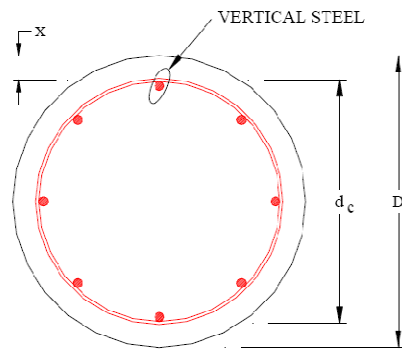
LRFD Section 5

Project #: __424977__

Sponsor: __TxDOT__

Subject: __Circular Concrete Shaft Shear Capacity - Accident #2 Bridge @ Milepost 232 I-45__

1.) Given the following Design Data:

$f_c := \begin{pmatrix} 3.05 \\ 4.00 \end{pmatrix} \cdot ksi$ Compressive Strength of Concrete (ksi)

$f_{yt} := 40ksi$ yield strength of spiral reinforcement (ksi)

VERTICAL STEEL

COLUMN SECTION

Diameter of Column Impacted:

$D := 30in$

Shear Reinforcement size:

$Stirrup_{size} := \frac{2}{8}in$

$A_v := \pi \cdot \left(Stirrup_{size}^2 \right) \cdot 0.25 \cdot 2$

$A_v = 0.098 \cdot in^2$ Area 2 Legs

Size of Vertical Steel (8~#9's)

$Vertical_{size} := 1.128in$ Dia. of Longitudinal Steel (in.)

$X := 2.25in$ Distance to Stirrup Center (in.)

$d_c := D - 2 \cdot X$ Diameter of Spiral Steel

$d_c = 25.5 \cdot in$ Diameter of Spiral Steel (in.)

$psi \equiv \frac{lbf}{in^2}$ $ksi \equiv \frac{kip}{in^2}$ $kips \equiv 1000lbf$

COLUMN SIDE VIEW

Subject: __Circular Concrete Shaft Shear Capacity - Accident #2 Bridge @ Milepost 232 I-45__

$s_{spa} := 6 in$ Pitch in Spiral Stirrup

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

$b_v := D$ Width of section (see Figure C5.8.2.9-3 page 5-57)

$D_r := D - X \cdot 2 - Stirrup_{size} - Vertical_{size}$
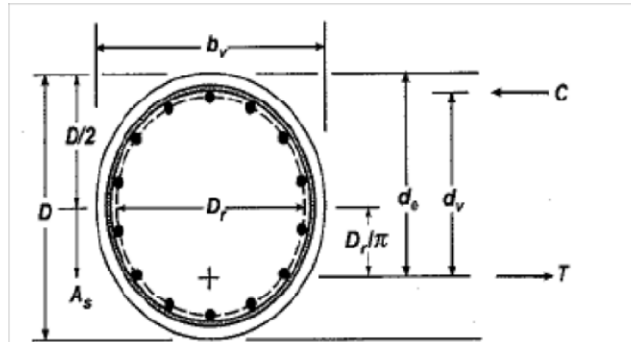
$D_r = 24.122 \cdot in$

Figure C5.8.2.9-2 Illustration of Terms b_v, d_v and d_e for Circular Sections.

$d_e := \dfrac{D}{2} + \dfrac{D_r}{\pi}$ $d_e = 22.678 \cdot in$ LRFD C5.8.2.9-2, pg. 5-67 Interim 2008

$d_v := 0.90 \cdot d_e$ see section commentary pag. 5-67   (LRFD C5.8.2.9)

$d_v = 20.41 \cdot in$

$a_g := 2 in$ Maximum Aggregate size in Concrete

Subject: __Circular Concrete Shaft Shear Capacity - Accident #2 Bridge @ Milepost 232 I-45__

2.) Calculate the minimum transverse reinforcement as per Section 5.8.2.5-1 pg. 5-63 Interim 2008

$$A_{transneeded} := 0.0316 \sqrt{\frac{f_c}{ksi}} \cdot ksi \cdot b_v \cdot \frac{s_{spa}}{f_{yt}}$$

$$A_{transneeded} = \begin{pmatrix} 0.2483 \\ 0.2844 \end{pmatrix} \cdot in^2 \qquad A_v - 0.098 \cdot in^2 \qquad \begin{array}{l} \text{Actual Area of Transverse Steel Provided} \\ (in^2) \end{array}$$

3.) Determine β & θ as per LRFD Section 5.8.3.4.2, Interim 2008, page 5-72

Calculate Crack Spacing Parameter sxe as per LRFD Eq. 5.8.3.4.2-5, Interim 2008, page 5-72

$$s_x := d_v \qquad \text{LRFD 5.8.3.4.2, Interim 2008, pg. 5-73}$$

$$s_{xe} := s_x \cdot \left(\frac{1.38in}{a_g + 0.63in} \right) \qquad s_{xe} = 10.71 \cdot in$$

$$\varepsilon_s := 0.006 \qquad \begin{array}{l} \text{LRFD Section 5.8.3.4.2, pg. 5-74 Interim 2008, based on maximum strain from severe} \\ \text{impact condition} \end{array}$$

Therefore, Calculate β:

$$\beta := \frac{4.8}{(1 + 750 \cdot \varepsilon_s)} \cdot \frac{51}{\left(39 + \frac{s_{xe}}{in} \right)} \qquad \beta = 0.895 \qquad \text{LRFD Eq. 5.8.3.4.2-2, Interim 2008, pg. 5-72}$$

$$\theta := (29 + 3500 \cdot \varepsilon_s) deg \qquad \theta = 50 \cdot deg \qquad \text{LRFD Eq. 5.8.3.4.2-3, Interim 2008, pg 5-72}$$

Subject: __Circular Concrete Shaft Shear Capacity - Accident #2 Bridge @ Milepost 232 I-45__

4.) Calculate V_c ~ Strength Attributable to Concrete as per Section 5.8.3.3 pg. 5-61:

with:

$$f'_c = \begin{pmatrix} 3.05 \\ 4 \end{pmatrix} \cdot ksi \qquad \text{Compressive strength of column concrete (psi)}$$

$$D = 30 \cdot in \qquad b_V = 30 \cdot in$$

$$d_V = 20.41 \cdot in$$

$$V_c := \left[(0.0316 \cdot \beta) \cdot \sqrt{\frac{f'_c}{ksi}} \cdot ksi \cdot b_V \cdot d_V \right] \qquad \begin{array}{l} \text{See Equation 5.8.3.3-3, pg. 5-70, Interim 2008.. for 1} \\ \text{shear plane} \end{array}$$

$$D = 30 \cdot in \qquad f'_c = \begin{pmatrix} 3.05 \\ 4 \end{pmatrix} \cdot ksi \qquad V_c = \begin{pmatrix} 30.256 \\ 34.65 \end{pmatrix} \cdot kips \qquad \begin{array}{l} \text{Nominal shear strength of concrete alone} \\ \text{for corresponding column Dia.} \end{array}$$

5.) Calculate V_s ~ Strength Attributable to Shear Reinforcement:

$$Stirrup_{size} = 0.25 \cdot in \qquad \text{Dia. of Stirrup Steel (in.)}$$

$$A_V = 0.098 \cdot in^2 \qquad \text{Area of Stirrup Steel (in}^2) \dots \text{2 Legs}$$

$$s_{spa} = 6 \cdot in \qquad \text{Spiral spacing / pitch (inches)}$$

Calculate the Stirrup Angle in degrees:

Stirrup Angles (degrees)

$$\alpha := atan\left(\left(\frac{s_{spa}}{\pi \cdot d_c}\right)\right) \qquad \alpha = 4.283 \cdot deg$$

Subject: __Circular Concrete Shaft Shear Capacity - Accident #2 Bridge @ Milepost 232 I-45__

$$V_s := \frac{A_v \cdot f_{yt} \cdot d_v \cdot (\cot(\theta) + \cot(\alpha)) \cdot \sin(\alpha)}{s_{spa}}$$

LFRD 5.8.3.3-4

$V_s = 14.158 \cdot \text{kips}$ $D = 30 \cdot \text{in}$

$V_s = 14.158 \cdot \text{kips}$ Nominal shear strength as per Section 5.8.3.3-4 pg. 5-61

6.) Calculate the Nominal Shear Capacity of Column for two Failure Plane Mechanism considering the strength of the concrete and the spiral reinforcing steel:

Concrete Shear Strength (single shear plane)

Spiral Reinforcement Strength (single shear plane)

$$V_c - \binom{30.256}{34.65} \cdot \text{kips}$$ $$V_s - 14.158 \cdot \text{kips}$$

$\phi_v := 0.9$ (LRFD 5.5.4.2.1, Interm 2008, pg. 5-25

$$V_r := (V_c + V_s) \cdot \phi_v \cdot 2$$ LRFD Eq. 5.8.3.3-1, Interim 2008, pg. 5-70, Shear Resistance 2 Shear planes

Column Dia.

Nominal Shear Capacity of Column (kips)

$D = 30 \cdot \text{in}$

$$V_r = \binom{79.947}{87.855} \cdot \text{kips}$$

3050 psi Concrete

4000 psi Concrete

ACCIDENT #3: TANCHAUHUA STREET OVER IH-37

Texas Transportation Institute

LRFD Section 5

Project #: __424977__

Sponsor: __TxDOT__

Subject: __Circular Concrete Pier Shear Capacity - Accident #3 Tancauhua St. over IH-37, Corpus Christi, TX__

1.) Given the following Design Data:

$$f_c := \begin{pmatrix} 3.05 \\ 4.00 \end{pmatrix} \cdot ksi \qquad \text{Compressive Strength of Concrete (ksi)}$$

$$f_{yt} := 40ksi \qquad \text{yield strength of spiral reinforcement (ksi)}$$

Diameter of Column Impacted:

$$D := 30in$$

Shear Reinforcement size:

$$Stirrup_{size} := \frac{2}{8}in$$

$$A_v := \pi \cdot \left(Stirrup_{size}^2\right) \cdot 0.25 \cdot 2$$

$$A_v = 0.098 \cdot in^2 \qquad \text{Area 2 Legs}$$

Size of Vertical Steel (8~#9's)

$$Vertical_{size} := 1.128in \qquad \text{Dia. of Longitudinal Steel (in.)}$$

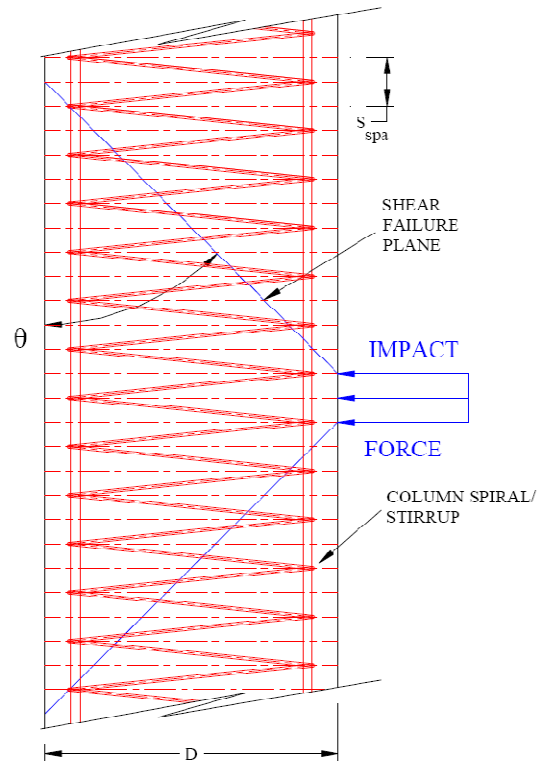$$X := 2.313in \qquad \text{Distance to Stirrup Center (in.)}$$

$$d_c := D - 2 \cdot X \qquad \text{Diameter of Spiral Steel}$$

$$d_c = 25.374 \cdot in \qquad \text{Diameter of Spiral Steel (in.)}$$
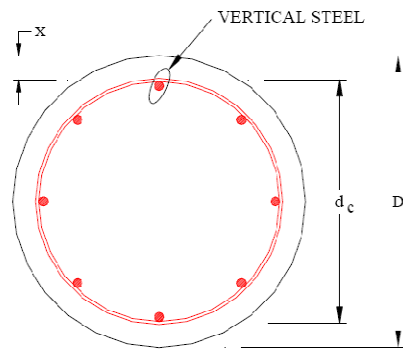
$$psi \equiv \frac{lbf}{in^2} \qquad ksi \equiv \frac{kip}{in^2} \qquad kips \equiv 1000lbf$$

COLUMN SECTION

COLUMN SIDE VIEW

Subject: __Circular Concrete Pier Shear Capacity - Accident #3 Tancauhua St. over IH-37, Corpus Christi, TX__

$s_{spa} := 6in$ Pitch in Spiral Stirrup

$\gamma_{con} := 150pcf$ Unit weight of concrete (lbf/ft³)

$b_v := D$ Width of section (see Figure C5.8.2.9-3 page 5-57)

$D_r := D - X \cdot 2 - Stirrup_{size} - Vertical_{size}$
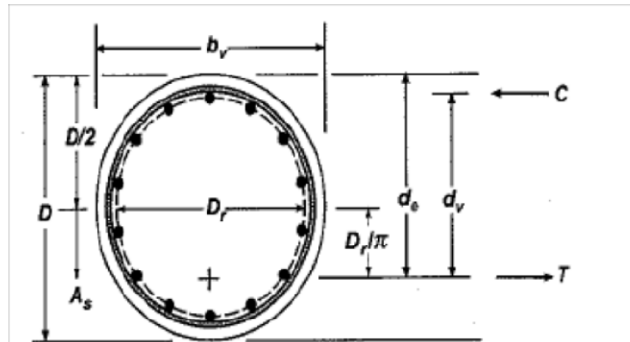
$D_r = 23.996 \cdot in$

Figure C5.8.2.9-2 Illustration of Terms b_v, d_v, and d_e for Circular Sections.

$d_e := \dfrac{D}{2} + \dfrac{D_r}{\pi}$ $d_e = 22.638 \cdot in$ LRFD C5.8.2.9-2, pg. 5-67 Interim 2008

$d_v := 0.90 \cdot d_e$ see section commentary pag. 5-67   (LRFD C5.8.2.9)

$d_v = 20.374 \cdot in$

$a_g := 2in$ Maximum Aggregate size in Concrete

Subject: __Circular Concrete Pier Shear Capacity - Accident #3 Tancauhua St. over IH-37, Corpus Christi, TX__

2.) Calculate the minimum transverse reinforcement as per Section 5.8.2.5-1 pg. 5-63 Interim 2008

$$A_{transneeded} := 0.0316 \sqrt{\frac{f_c}{ksi}} \cdot ksi \cdot b_v \cdot \frac{s_{spa}}{f_{yt}}$$

$$A_{transneeded} = \begin{pmatrix} 0.2483 \\ 0.2844 \end{pmatrix} \cdot in^2 \qquad A_v = 0.098 \cdot in^2 \qquad \text{Actual Area of Transverse Steel Provided } (in^2)$$

3.) Determine β & θ as per LRFD Section 5.8.3.4.2, Interim 2008, page 5-72

Calculate Crack Spacing Parameter sxe as per LRFD Eq. 5.8.3.4.2-5, Interim 2008, page 5-72

$$s_x := d_v \qquad \text{LRFD 5.8.3.4.2, Interim 2008, pg. 5-73}$$

$$s_{xe} := s_x \cdot \left(\frac{1.38in}{a_g + 0.63in} \right) \qquad s_{xe} = 10.691 \cdot in$$

$$\varepsilon_s := 0.006 \qquad \text{LRFD Section 5.8.3.4.2, pg. 5-74 Interim 2008, based on maximum strain from severe impact condition}$$

Therefore, Calculate β:

$$\beta := \frac{4.8}{\left(1 + 750 \cdot \varepsilon_s\right)} \cdot \frac{51}{\left(39 + \frac{s_{xe}}{in}\right)} \qquad \beta = 0.896 \qquad \text{LRFD Eq. 5.8.3.4.2-2, Interim 2008, pg. 5-72}$$

$$\theta := \left(29 + 3500 \cdot \varepsilon_s\right) deg \qquad \theta = 50 \cdot deg \qquad \text{LRFD Eq. 5.8.3.4.2-3, Interim 2008, pg 5-72}$$

Subject: __Circular Concrete Pier Shear Capacity - Accident #3 Tancauhua St. over IH-37, Corpus Christi, TX__

4.) Calculate V_c ~ Strength Attributable to Concrete as per Section 5.8.3.3 pg. 5-61:

with:

$f_c = \begin{pmatrix} 3.05 \\ 4 \end{pmatrix} \cdot ksi$ 　　　　Compressive strength of column concrete (psi)

$D = 30 \cdot in$ 　　　　$b_v = 30 \cdot in$

$d_v = 20.374 \cdot in$

$$V_c := \left[(0.0316 \cdot \beta) \cdot \sqrt{\frac{f_c}{ksi}} \cdot ksi \cdot b_v \cdot d_v \right]$$ 　　See Equation 5.8.3.3-3, pg. 5-70, Interim 2008.. for 1 shear plane

$D = 30 \cdot in$ 　　　$f_c = \begin{pmatrix} 3.05 \\ 4 \end{pmatrix} \cdot ksi$ 　　　$V_c = \begin{pmatrix} 30.214 \\ 34.602 \end{pmatrix} \cdot kips$ 　　Nominal shear strength of concrete alone for corresponding column Dia.

5.) Calculate V_s ~ Strength Attributable to Shear Reinforcement:

$Stirrup_{size} = 0.25 \cdot in$ 　　Dia. of Stirrup Steel (in.)

$A_v = 0.098 \cdot in^2$ 　　Area of Stirrup Steel (in^2) .... 2 Legs

$s_{spa} = 6 \cdot in$ 　　Spiral spacing / pitch (inches)

Calculate the Stirrup Angle in degrees:

Stirrup Angles (degrees)

$\alpha := atan\left(\left(\frac{s_{spa}}{\pi \cdot d_c}\right)\right)$ 　　　　$\alpha = 4.304 \cdot deg$

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Texas
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LRFD Section 5

Project #: __424977__

Sponsor: __TxDOT__

Subject: __Circular Concrete Pier Shear Capacity - Accident #3 Tancauhua St. over IH-37, Corpus Christi, TX__

$$V_s := \frac{A_v \cdot f_{yt} \cdot d_v \cdot (\cot(\theta) + \cot(\alpha)) \cdot \sin(\alpha)}{s_{spa}}$$ LFRD 5.8.3.3-4

$V_s = 14.137 \cdot \text{kips}$ $D = 30 \cdot \text{in}$

$V_s - 14.137 \cdot \text{kips}$ Nominal shear strength as per Section 5.8.3.3-4 pg. 5-61

6.) Calculate the Nominal Shear Capacity of Column for two Failure Plane Mechanism considering the strength of the concrete and the spiral reinforcing steel:

Concrete Shear Strength
(single shear plane)

Spiral Reinforcment Strength
(single shear plane)

$$V_c = \begin{pmatrix} 30.214 \\ 34.602 \end{pmatrix} \cdot \text{kips}$$ $V_s = 14.137 \cdot \text{kips}$

$\phi_v := 0.9$ (LRFD 5.5.4.2.1, Interm 2008, pg. 5-25

$$V_r := (V_c + V_s) \cdot \phi_v \cdot 2$$ LRFD Eq. 5.8.3.3-1, Interim 2008, pg. 5-70, Shear Resistance 2 Shear planes

Column Dia.

Nominal Shear Capacity of Column (kips)

$D = 30 \cdot \text{in}$

$$V_r - \begin{pmatrix} 79.833 \\ 87.73 \end{pmatrix} \cdot \text{kips}$$ 3050 psi Concrete

4000 psi Concrete

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ACCIDENT #4: IH-35 AND US 77

Texas Transportation Institute

LRFD Section 5

Project #: __424977__

Sponsor: __TxDOT__

Subject: __Circular Concrete Pier Shear Capacity - Accident #4 IH35 & US 77__

1.) Given the following Design Data:

$$f_c := \begin{pmatrix} 3.05 \\ 4.00 \end{pmatrix} \cdot ksi$$

Compressive Strength of Concrete (ksi)

$f_{yt} := 40ksi$ — yield strength of spiral reinforcement (ksi)

Diameter of Column Impacted:

$D := 30in$

Shear Reinforcement size:

$$Stirrup_{size} := \frac{2}{8}in$$

$$A_v := \pi \cdot \left(Stirrup_{size}^2\right) \cdot 0.25 \cdot 2$$

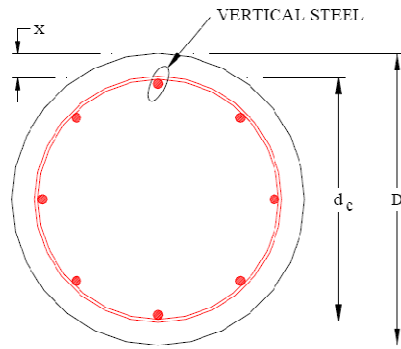$$A_v = 0.098 \cdot in^2 \qquad \text{Area 2 Legs}$$

Size of Vertical Steel (8~#9's)

$Vertical_{size} := 1.128in$ — Dia. of Longitudinal Steel (in.)

$X := 2.25in$ — Distance to Stirrup Center (in.)

$d_c := D - 2 \cdot X$ — Diameter of Spiral Steel

$d_c = 25.5 \cdot in$ — Diameter of Spiral Steel (in.)

$$psi \equiv \frac{lbf}{in^2} \qquad ksi \equiv \frac{kip}{in^2} \qquad kips \equiv 1000lbf$$

VERTICAL STEEL

COLUMN SECTION

COLUMN SIDE VIEW

IMPACT

FORCE

SHEAR FAILURE PLANE

COLUMN SPIRAL/ STIRRUP

Subject: **Circular Concrete Pier Shear Capacity - Accident #4 IH35 & US 77**

$s_{spa} := 6in$ Pitch in Spiral Stirrup

$\gamma_{con} := 150pcf$ Unit weight of
concrete (lbf/ft³)

$b_v := D$ Width of section
(see Figure C5.8.2.9-3 page 5-57)

$D_r := D - X \cdot 2 - Stirrup_{size} - Vertical_{size}$
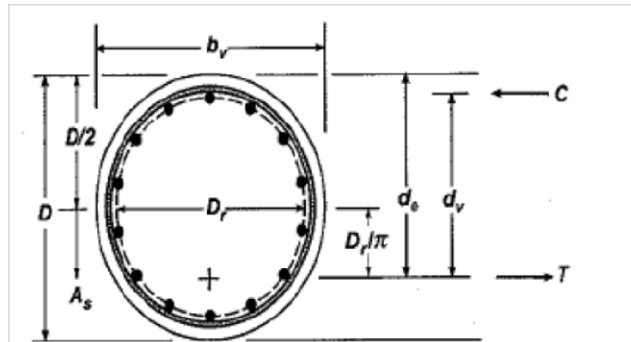
$D_r = 24.122 \cdot in$

Figure C5.8.2.9-2 Illustration of Terms b_v, d_v and d_e for Circular Sections.

$d_e := \dfrac{D}{2} + \dfrac{D_r}{\pi}$ $d_e = 22.678 \cdot in$ LRFD C5.8.2.9-2, pg. 5-67 Interim 2008

$d_v := 0.90 \cdot d_c$ see section commentary pag. 5-67   (LRFD C5.8.2.9)

$d_v = 20.41 \cdot in$

$a_g := 2in$ Maximum Aggregate size in Concrete

Subject: __Circular Concrete Pier Shear Capacity - Accident #4 IH35 & US 77__

2.) Calculate the minimum transverse reinforcement as per Section 5.8.2.5-1 pg. 5-63 Interim 2008

$$A_{transneeded} := 0.0316 \sqrt{\frac{f_c}{ksi}} \cdot ksi \cdot b_v \cdot \frac{s_{spa}}{f_{yt}}$$

$$A_{transneeded} = \begin{pmatrix} 0.2483 \\ 0.2844 \end{pmatrix} \cdot in^2 \qquad A_v = 0.098 \cdot in^2 \qquad \begin{array}{l} \text{Actual Area of Transverse Steel Provided} \\ \text{(in}^2\text{)} \end{array}$$

3.) Determine β & θ as per LRFD Section 5.8.3.4.2, Interim 2008, page 5-72

Calculate Crack Spacing Parameter sxe as per LRFD Eq. 5.8.3.4.2-5, Interim 2008, page 5-72

$$s_x := d_v \qquad \text{LRFD 5.8.3.4.2, Interim 2008, pg. 5-73}$$

$$s_{xe} := s_x \cdot \left(\frac{1.38in}{a_g + 0.63in} \right) \qquad s_{xe} = 10.71 \cdot in$$

$$\varepsilon_s := 0.006 \qquad \begin{array}{l} \text{LRFD Section 5.8.3.4.2, pg. 5-74 Interim 2008, based on maximum strain from severe} \\ \text{impact condition} \end{array}$$

Therefore, Calculate β:

$$\beta := \frac{4.8}{\left(1 + 750 \cdot \varepsilon_s\right)} \cdot \frac{51}{\left(39 + \frac{s_{xe}}{in}\right)} \qquad \beta = 0.895 \qquad \text{LRFD Eq. 5.8.3.4.2-2, Interim 2008, pg. 5-72}$$

$$\theta := \left(29 + 3500 \cdot \varepsilon_s\right) deg \qquad \theta = 50 \cdot deg \qquad \text{LRFD Eq. 5.8.3.4.2-3, Interim 2008, pg 5-72}$$

Subject: __Circular Concrete Pier Shear Capacity - Accident #4 IH35 & US 77__

4.) <u>Calculate V_c ~ Strength Attributable to Concrete as per Section 5.8.3.3 pg. 5-61:</u>

with:

$$f_c = \begin{pmatrix} 3.05 \\ 4 \end{pmatrix} \cdot ksi \qquad \text{Compressive strength of column concrete (psi)}$$

$$D = 30 \cdot in \qquad b_v = 30 \cdot in$$

$$d_v = 20.41 \cdot in$$

$$V_c := \left[(0.0316 \cdot \beta) \cdot \sqrt{\frac{f_c}{ksi}} \cdot ksi \cdot b_v \cdot d_v \right]$$

See Equation 5.8.3.3-3, pg. 5-70, Interim 2008.. for 1 shear plane

$$D = 30 \cdot in \qquad f_c = \begin{pmatrix} 3.05 \\ 4 \end{pmatrix} \cdot ksi \qquad V_c = \begin{pmatrix} 30.256 \\ 34.65 \end{pmatrix} \cdot kips$$

Nominal shear strength of concrete alone for corresponding column Dia.

5.) <u>Calculate V_s ~ Strength Attributable to Shear Reinforcement:</u>

$$Stirrup_{size} = 0.25 \cdot in \qquad \text{Dia. of Stirrup Steel (in.)}$$

$$A_v = 0.098 \cdot in^2 \qquad \text{Area of Stirrup Steel (in}^2) .... \text{2 Legs}$$

$$s_{spa} = 6 \cdot in \qquad \text{Spiral spacing / pitch (inches)}$$

Calculate the Stirrup Angle in degrees:

Stirrup Angles (degrees)

$$\alpha := atan\left(\left(\frac{s_{spa}}{\pi \cdot d_c}\right)\right) \qquad \alpha = 4.283 \cdot deg$$

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LRFD Section 5

Project #: __424977__

Sponsor: __TxDOT__

Subject: __Circular Concrete Pier Shear Capacity - Accident #4 IH35 & US 77__

$$V_s := \frac{A_v \cdot f_{yt} \cdot d_v \cdot (\cot(\theta) + \cot(\alpha)) \cdot \sin(\alpha)}{s_{spa}}$$

LFRD 5.8.3.3-4

$V_s = 14.158 \cdot kips$ $D = 30 \cdot in$

$V_s = 14.158 \cdot kips$ Nominal shear strength as per Section 5.8.3.3-4 pg. 5-61

6.) Calculate the Nominal Shear Capacity of Column for two Failure Plane Mechanism considering the strength of the concrete and the spiral reinforcing steel:

Concrete Shear Strength (single shear plane)

Spiral Reinforcment Strength (single shear plane)

$$V_c = \begin{pmatrix} 30.256 \\ 34.65 \end{pmatrix} \cdot kips$$ $V_s = 14.158 \cdot kips$

$\phi_v := 0.9$ (LRFD 5.5.4.2.1, Interm 2008, pg. 5-25

$$V_r := (V_c + V_s) \cdot \phi_v \cdot 2$$ LRFD Eq. 5.8.3.3-1, Interim 2008, pg. 5-70, Shear Resistance 2 Shear planes

Column Dia.

Nominal Shear Capacity of Column (kips)

$D = 30 \cdot in$

$$V_r = \begin{pmatrix} 79.947 \\ 87.855 \end{pmatrix} \cdot kips$$

3050 psi Concrete

4000 psi Concrete

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ACCIDENT #5: FM 2207 OVER IH-20

Texas Transportation Institute

LRFD Section 5

Project #: __424977__

Sponsor: __TxDOT__

Subject: __Circular Concrete Pier Shear Capacity - Accident #5 FM#2207 over IH-20__

1.) <u>Given the following Design Data:</u>

$f_c := \begin{pmatrix} 3.05 \\ 4.00 \end{pmatrix} \cdot ksi$ Compressive Strength of Concrete (ksi)

$f_{yt} := 40ksi$ yield strength of spiral reinforcement (ksi)

Diameter of Column Impacted:

$D := 30in$

Shear Reinforcement size:

$Stirrup_{size} := \frac{2}{8}in$

$A_v := \pi \cdot \left(Stirrup_{size}^2 \right) \cdot 0.25 \cdot 2$

$A_v = 0.098 \cdot in^2$ Area 2 Legs

Size of Vertical Steel (8~#9's)
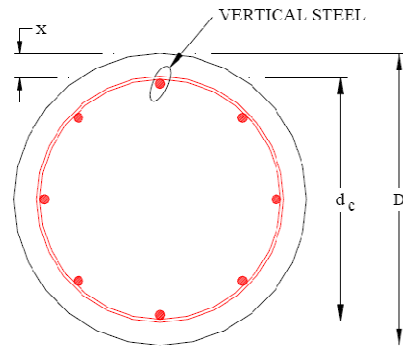
$Vertical_{size} := 1.128in$ Dia. of Longitudinal Steel (in.)

$X := 2.25in$ Distance to Stirrup Center (in.)
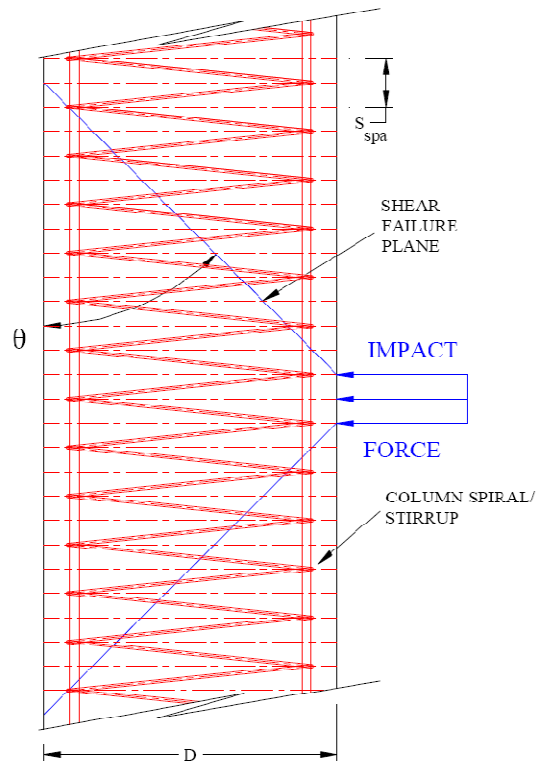
$d_c := D - 2 \cdot X$ Diameter of Spiral Steel

$d_c = 25.5 \cdot in$ Diameter of Spiral Steel (in.)

$psi \equiv \frac{lbf}{in^2}$ $ksi \equiv \frac{kip}{in^2}$ $kips \equiv 1000lbf$

VERTICAL STEEL

COLUMN SECTION

SHEAR FAILURE PLANE

IMPACT

FORCE

COLUMN SPIRAL/ STIRRUP

COLUMN SIDE VIEW

Subject: __Circular Concrete Pier Shear Capacity - Accident #5 FM#2207 over IH-20__

$s_{spa} := 6in$ Pitch in Spiral Stirrup

$\gamma_{con} := 150pcf$ Unit weight of concrete (lbf/ft³)

$b_v := D$ Width of section (see Figure C5.8.2.9-3 page 5-57)

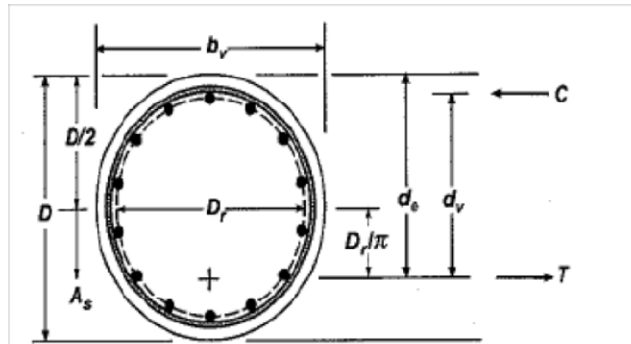$D_r := D - X \cdot 2 - Stirrup_{size} - Vertical_{size}$

$D_r = 24.122 \cdot in$

Figure C5.8.2.9-2 Illustration of Terms b_v, d_v and d_e for Circular Sections.

$d_e := \dfrac{D}{2} + \dfrac{D_r}{\pi}$ $d_e = 22.678 \cdot in$ LRFD C5.8.2.9-2, pg. 5-67 Interim 2008

$d_v := 0.90 \cdot d_e$ see section commentary pag. 5-67   (LRFD C5.8.2.9)

$d_v = 20.41 \cdot in$

$a_g := 2in$ Maximum Aggregate size in Concrete

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Texas
Transportation
Institute

LRFD Section 5

Project #: __424977__

Sponsor: __TxDOT__

Subject: __Circular Concrete Pier Shear Capacity - Accident #5 FM#2207 over IH-20__

2.) Calculate the minimum transverse reinforcement as per Section 5.8.2.5-1 pg. 5-63 Interim 2008

$$A_{transneeded} := 0.0316 \sqrt{\frac{f_c}{ksi}} \cdot ksi \cdot b_v \cdot \frac{s_{spa}}{f_{yt}}$$

$$A_{transneeded} = \begin{pmatrix} 0.2483 \\ 0.2844 \end{pmatrix} \cdot in^2 \qquad A_v = 0.098 \cdot in^2 \qquad \text{Actual Area of Transverse Steel Provided (in}^2)$$

3.) Determine β & θ as per LRFD Section 5.8.3.4.2, Interim 2008, page 5-72

Calculate Crack Spacing Parameter sxe as per LRFD Eq. 5.8.3.4.2-5, Interim 2008, page 5-72

$$s_x := d_v \qquad \text{LRFD 5.8.3.4.2, Interim 2008, pg. 5-73}$$

$$s_{xe} := s_x \cdot \left(\frac{1.38in}{a_g + 0.63in} \right) \qquad s_{xe} = 10.71 \cdot in$$

$$\varepsilon_s := 0.006 \qquad \text{LRFD Section 5.8.3.4.2, pg. 5-74 Interim 2008, based on maximum strain from severe impact condition}$$

Therefore, Calculate β:

$$\beta := \frac{4.8}{(1 + 750 \cdot \varepsilon_s)} \cdot \frac{51}{\left(39 + \frac{s_{xe}}{in}\right)} \qquad \beta = 0.895 \qquad \text{LRFD Eq. 5.8.3.4.2-2, Interim 2008, pg. 5-72}$$

$$\theta := (29 + 3500 \cdot \varepsilon_s) deg \qquad \theta = 50 \cdot deg \qquad \text{LRFD Eq. 5.8.3.4.2-3, Interim 2008, pg 5-72}$$

Subject: __Circular Concrete Pier Shear Capacity - Accident #5 FM#2207 over IH-20__

4.) Calculate $V_c \sim$ Strength Attributable to Concrete as per Section 5.8.3.3 pg. 5-61:

with:

$$f_c = \begin{pmatrix} 3.05 \\ 4 \end{pmatrix} \cdot ksi \qquad \text{Compressive strength of column concrete (psi)}$$

$$D = 30 \cdot in \qquad b_v = 30 \cdot in$$

$$d_v = 20.41 \cdot in$$

$$V_c := \left[(0.0316 \cdot \beta) \cdot \sqrt{\frac{f_c}{ksi}} \cdot ksi \cdot b_v \cdot d_v \right] \qquad \text{See Equation 5.8.3.3-3, pg. 5-70, Interim 2008.. for 1 shear plane}$$

$$D = 30 \cdot in \qquad f_c = \begin{pmatrix} 3.05 \\ 4 \end{pmatrix} \cdot ksi \qquad V_c = \begin{pmatrix} 30.256 \\ 34.65 \end{pmatrix} \cdot kips \qquad \text{Nominal shear strength of concrete alone for corresponding column Dia.}$$

5.) Calculate $V_s \sim$ Strength Attributable to Shear Reinforcement:

$$Stirrup_{size} = 0.25 \cdot in \qquad \text{Dia. of Stirrup Steel (in.)}$$

$$A_v = 0.098 \cdot in^2 \qquad \text{Area of Stirrup Steel (in}^2\text{) .... 2 Legs}$$

$$s_{spa} = 6 \cdot in \qquad \text{Spiral spacing / pitch (inches)}$$

Calculate the Stirrup Angle in degrees:

Stirrup Angles (degrees)

$$\alpha := atan\left(\left(\frac{s_{spa}}{\pi \cdot d_c}\right)\right) \qquad \alpha = 4.283 \cdot deg$$

Subject: **Circular Concrete Pier Shear Capacity - Accident #5 FM#2207 over IH-20**

$$V_s := \frac{A_v \cdot f_{yt} \cdot d_v \cdot (\cot(\theta) + \cot(\alpha)) \cdot \sin(\alpha)}{s_{spa}}$$

LFRD 5.8.3.3-4

$V_s = 14.158 \cdot kips$ $D = 30 \cdot in$

$V_s = 14.158 \cdot kips$ Nominal shear strength as per Section 5.8.3.3-4 pg. 5-61

6.) Calculate the Nominal Shear Capacity of Column for two Failure Plane Mechanism considering the strength of the concrete and the spiral reinforcing steel:

Concrete Shear Strength (single shear plane)

Spiral Reinforcment Strength (single shear plane)

$$V_c = \begin{pmatrix} 30.256 \\ 34.65 \end{pmatrix} \cdot kips$$ $V_s = 14.158 \cdot kips$

$\phi_v := 0.9$ (LRFD 5.5.4.2.1, Interm 2008, pg. 5-25

$$V_r := (V_c + V_s) \cdot \phi_v \cdot 2$$ LRFD Eq. 5.8.3.3-1, Interim 2008, pg. 5-70, Shear Resistance 2 Shear planes

Column Dia.

Nominal Shear Capacity of Column (kips)

$D = 30 \cdot in$

$$V_r = \begin{pmatrix} 79.947 \\ 87.855 \end{pmatrix} \cdot kips$$ 3050 psi Concrete

4000 psi Concrete

ACCIDENT #7: PYKE ROAD OVER IH-10

Texas Transportation Institute

LRFD Section 5

Project #: __424977__

Sponsor: __TxDOT__

Subject: __Circular Concrete Pier Shear Capacity - Accident #7 Pyke Road over IH-10__

1.) Given the following Design Data:

$$f_c := \begin{pmatrix} 3.05 \\ 4.00 \end{pmatrix} \cdot ksi \qquad \text{Compressive Strength of Concrete (ksi)}$$

$$f_{yt} := 40 ksi \qquad \text{yield strength of spiral reinforcement (ksi)}$$

**** NO DRAWINGS DETAILS FROM PHOTOS ***

Diameter of Column Impacted:

$$D := 30 in$$

Shear Reinforcement size:

$$Stirrup_{size} := \frac{2}{8} in$$

$$A_v := \pi \cdot \left(Stirrup_{size}^2 \right) \cdot 0.25 \cdot 2$$

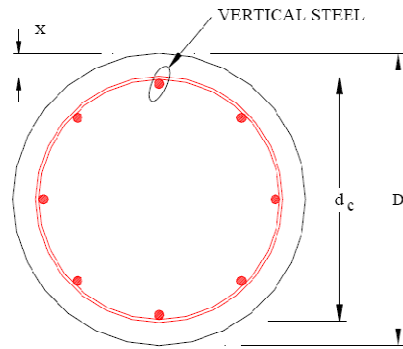$$A_v = 0.098 \cdot in^2 \qquad \text{Area 2 Legs}$$

Size of Vertical Steel (8~#9's)

$$Vertical_{size} := 1.128 in \qquad \text{Dia. of Longitudinal Steel (in.)}$$

$$X := 2.25 in \qquad \text{Distance to Stirrup Center (in.)}$$
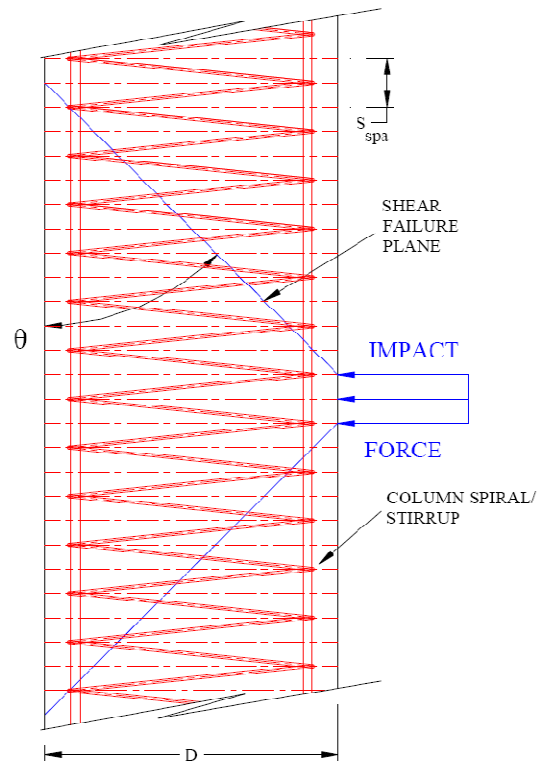
$$d_c := D - 2 \cdot X \qquad \text{Diameter of Spiral Steel}$$

$$d_c = 25.5 \cdot in \qquad \text{Diameter of Spiral Steel (in.)}$$

$$psi \equiv \frac{lbf}{in^2} \qquad ksi \equiv \frac{kip}{in^2} \qquad kips \equiv 1000 lbf$$

VERTICAL STEEL

COLUMN SECTION

COLUMN SIDE VIEW

S_spa

SHEAR FAILURE PLANE

θ

IMPACT

FORCE

COLUMN SPIRAL/ STIRRUP

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Subject: __Circular Concrete Pier Shear Capacity - Accident #7 Pyke Road over IH-10__

$s_{spa} := 6\text{in}$ Pitch in Spiral Stirrup

$\gamma_{con} := 150\text{pcf}$ Unit weight of concrete (lbf/ft^3)

$b_v := D$ Width of section (see Figure C5.8.2.9-3 page 5-57)

$D_r := D - X \cdot 2 - Stirrup_{size} - Vertical_{size}$
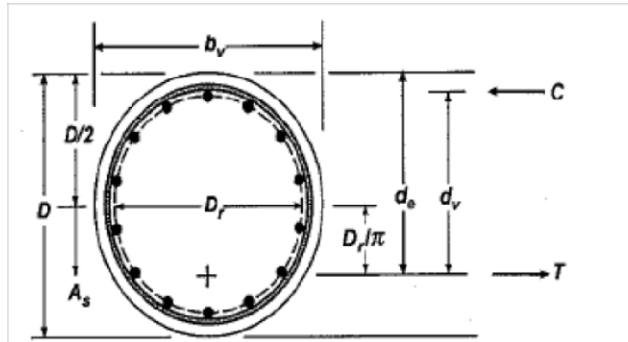
$D_r = 24.122 \cdot \text{in}$

Figure C5.8.2.9-2 Illustration of Terms b_v, d_v and d_e for Circular Sections.

$d_e := \dfrac{D}{2} + \dfrac{D_r}{\pi}$ $d_e = 22.678 \cdot \text{in}$ LRFD C5.8.2.9-2, pg. 5-67 Interim 2008

$d_v := 0.90 \cdot d_e$ see section commentary pag. 5-67   (LRFD C5.8.2.9)

$d_v = 20.41 \cdot \text{in}$

$a_g := 2\text{in}$ Maximum Aggregate size in Concrete

Texas
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LRFD Section 5

Project #: __424977__

Sponsor: __TxDOT__

Subject: __Circular Concrete Pier Shear Capacity - Accident #7 Pyke Road over III-10__

2.) Calculate the minimum transverse reinforcement as per Section 5.8.2.5-1 pg. 5-63 Interim 2008

$$A_{transneeded} := 0.0316 \sqrt{\frac{f_c}{ksi}} \cdot ksi \cdot b_v \cdot \frac{s_{spa}}{f_{yt}}$$

$$A_{transneeded} = \begin{pmatrix} 0.2483 \\ 0.2844 \end{pmatrix} \cdot in^2 \qquad A_v = 0.098 \cdot in^2 \qquad \text{Actual Area of Transverse Steel Provided}$$
$$(in^2)$$

3.) Determine β & θ as per LRFD Section 5.8.3.4.2, Interim 2008, page 5-72

Calculate Crack Spacing Parameter sxe as per LRFD Eq. 5.8.3.4.2-5, Interim 2008, page 5-72

$$s_x := d_v \qquad \text{LRFD 5.8.3.4.2, Interim 2008, pg. 5-73}$$

$$s_{xe} := s_x \cdot \left(\frac{1.38in}{a_g + 0.63in} \right) \qquad s_{xe} = 10.71 \cdot in$$

$$\varepsilon_s := 0.006 \qquad \text{LRFD Section 5.8.3.4.2, pg. 5-74 Interim 2008, based on maximum strain from severe impact condition}$$

Therefore, Calculate β:

$$\beta := \frac{4.8}{(1 + 750 \cdot \varepsilon_s)} \cdot \frac{51}{\left(39 + \frac{s_{xe}}{in} \right)} \qquad \beta = 0.895 \qquad \text{LRFD Eq. 5.8.3.4.2-2, Interim 2008, pg. 5-72}$$

$$\theta := (29 + 3500 \cdot \varepsilon_s) deg \qquad \theta = 50 \cdot deg \qquad \text{LRFD Eq. 5.8.3.4.2-3, Interim 2008, pg 5-72}$$

Subject: __Circular Concrete Pier Shear Capacity - Accident #7 Pyke Road over IH-10__

4.) Calculate V_c ~ Strength Attributable to Concrete as per Section 5.8.3.3 pg. 5-61:

with:

$f_c = \begin{pmatrix} 3.05 \\ 4 \end{pmatrix} \cdot ksi$ Compressive strength of column concrete (psi)

$D = 30 \cdot in$ $b_v = 30 \cdot in$

$d_v = 20.41 \cdot in$

$$V_c := \left[(0.0316 \cdot \beta) \cdot \sqrt{\frac{f_c}{ksi}} \cdot ksi \cdot b_v \cdot d_v \right]$$

See Equation 5.8.3.3-3, pg. 5-70, Interim 2008.. for 1 shear plane

$D = 30 \cdot in$ $f_c = \begin{pmatrix} 3.05 \\ 4 \end{pmatrix} \cdot ksi$ $V_c = \begin{pmatrix} 30.256 \\ 34.65 \end{pmatrix} \cdot kips$

Nominal shear strength of concrete alone for corresponding column Dia.

5.) Calculate V_s ~ Strength Attributable to Shear Reinforcement:

$Stirrup_{size} = 0.25 \cdot in$ Dia. of Stirrup Steel (in.)

$A_v = 0.098 \cdot in^2$ Area of Stirrup Steel (in^2) .... 2 Legs

$s_{spa} = 6 \cdot in$ Spiral spacing / pitch (inches)

Calculate the Stirrup Angle in degrees:

Stirrup Angles (degrees)

$\alpha := atan\left(\left(\frac{s_{spa}}{\pi \cdot d_c}\right)\right)$ $\alpha = 4.283 \cdot deg$

Subject: __Circular Concrete Pier Shear Capacity - Accident #7 Pyke Road over IH-10__

$$V_s := \frac{A_v \cdot f_{yt} \cdot d_v \cdot (\cot(\theta) + \cot(\alpha)) \cdot \sin(\alpha)}{s_{spa}}$$ LFRD 5.8.3.3-4

$V_s = 14.158 \cdot kips$ $D = 30 \cdot in$

$V_s = 14.158 \cdot kips$ Nominal shear strength as per Section 5.8.3.3-4 pg. 5-61

6.) Calculate the Nominal Shear Capacity of Column for two Failure Plane Mechanism considering the strength of the concrete and the spiral reinforcing steel:

Concrete Shear Strength   Spiral Reinforcment Strength
(single shear plane)   (single shear plane)

$$V_c = \begin{pmatrix} 30.256 \\ 34.65 \end{pmatrix} \cdot kips$$ $V_s = 14.158 \cdot kips$

$\phi_v := 0.9$ (LRFD 5.5.4.2.1, Interm 2008, pg. 5-25

$V_r := (V_c + V_s) \cdot \phi_v \cdot 2$ LRFD Eq. 5.8.3.3-1, Interim 2008, pg. 5-70, Shear Resistance 2 Shear planes

Column Dia.   Nominal Shear Capacity of Column (kips)

$D = 30 \cdot in$

$$V_r = \begin{pmatrix} 79.947 \\ 87.855 \end{pmatrix} \cdot kips$$ 3050 psi Concrete

4000 psi Concrete

ACCIDENT #8: SH 14 OVER IH-45

Texas Transportation Institute

LRFD Section 5

Project #: __424977__

Sponsor: __TxDOT__

Subject: __Circular Concrete Pier Shear Capacity - Accident #8 SH 14 ov IH-45__

1.) Given the following Design Data:

$$f_c := \begin{pmatrix} 3.05 \\ 4.00 \end{pmatrix} \cdot ksi$$

Compressive Strength of Concrete (ksi)

$f_{yt} := 60ksi$ yield strength of spiral reinforcement (ksi)

Diameter of Column Impacted:

$D := 30in$

Shear Reinforcement size:

$$Stirrup_{size} := \frac{3}{8}in$$

$$A_v := \pi \cdot \left(Stirrup_{size}^2 \right) \cdot 0.25 \cdot 2$$

$A_v = 0.221 \cdot in^2$ Area 2 Legs

Size of Vertical Steel (8~#9's)

$Vertical_{size} := 1.128in$ Dia. of Longitudinal Steel (in.)

$X := 2.25in$ Distance to Stirrup Center (in.)

$d_c := D - 2 \cdot X$ Diameter of Spiral Steel

$d_c = 25.5 \cdot in$ Diameter of Spiral Steel (in.)

$$psi \equiv \frac{lbf}{in^2} \qquad ksi \equiv \frac{kip}{in^2} \qquad kips \equiv 1000lbf$$

VERTICAL STEEL

COLUMN SECTION

SHEAR FAILURE PLANE

IMPACT

FORCE

COLUMN SPIRAL/ STIRRUP

COLUMN SIDE VIEW

Subject: __Circular Concrete Pier Shear Capacity - Accident #8 SH 14 ov IH-45__

$s_{spa} := 6in$ Pitch in Spiral Stirrup

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

$b_v := D$ Width of section (see Figure C5.8.2.9-3 page 5-57)

$D_r :- D - X \cdot 2 - Stirrup_{size} - Vertical_{size}$
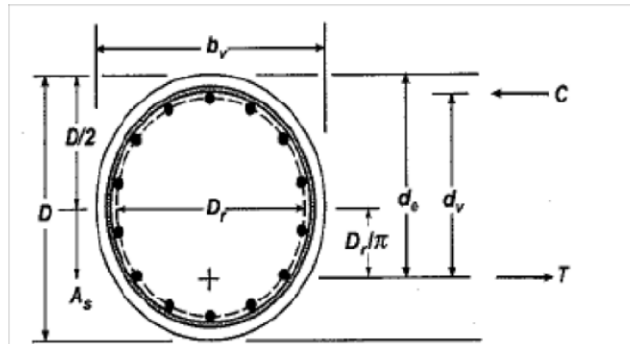
$D_r = 23.997 \cdot in$

Figure C5.8.2.9-2 Illustration of Terms b_v, d_v and d_e for Circular Sections.

$d_e := \dfrac{D}{2} + \dfrac{D_r}{\pi}$ $d_e = 22.638 \cdot in$ LRFD C5.8.2.9-2, pg. 5-67 Interim 2008

$d_v := 0.90 \cdot d_e$ see section commentary pag. 5-67   (LRFD C5.8.2.9)

$d_v - 20.375 \cdot in$

$a_g := 2in$ Maximum Aggregate size in Concrete

Subject: __Circular Concrete Pier Shear Capacity - Accident #8 SH 14 ov IH-45__

2.) Calculate the minimum transverse reinforcement as per Section 5.8.2.5-1 pg. 5-63 Interim 2008

$$A_{transneeded} := 0.0316 \sqrt{\frac{f_c}{ksi}} \cdot ksi \cdot b_v \cdot \frac{s_{spa}}{f_{yt}}$$

$$A_{transneeded} = \begin{pmatrix} 0.1656 \\ 0.1896 \end{pmatrix} \cdot in^2 \qquad A_v = 0.221 \cdot in^2 \qquad \text{Actual Area of Transverse Steel Provided} \\ (in^2)$$

3.) Determine β & θ as per LRFD Section 5.8.3.4.2, Interim 2008, page 5-72

Calculate Crack Spacing Parameter sxe as per LRFD Eq. 5.8.3.4.2-5, Interim 2008, page 5-72

$$s_x := d_v \qquad \text{LRFD 5.8.3.4.2, Interim 2008, pg. 5-73}$$

$$s_{xe} := s_x \cdot \left(\frac{1.38in}{a_g + 0.63in} \right) \qquad s_{xe} = 10.691 \cdot in$$

$$\varepsilon_s := 0.006 \qquad \text{LRFD Section 5.8.3.4.2, pg. 5-74 Interim 2008, based on maximum strain from severe impact condition}$$

Therefore, Calculate β:

$$\beta := \frac{4.8}{(1 + 750 \cdot \varepsilon_s)} \cdot \frac{51}{\left(39 + \frac{s_{xe}}{in} \right)} \qquad \beta - 0.896 \qquad \text{LRFD Eq. 5.8.3.4.2-2, Interim 2008, pg. 5-72}$$

$$\theta := (29 + 3500 \cdot \varepsilon_s) deg \qquad \theta = 50 \cdot deg \qquad \text{LRFD Eq. 5.8.3.4.2-3, Interim 2008, pg 5-72}$$

Subject: __Circular Concrete Pier Shear Capacity - Accident #8 SH 14 ov IH-45__

4.) Calculate V_c ~ Strength Attributable to Concrete as per Section 5.8.3.3 pg. 5-61:

with:

$$f_c = \begin{pmatrix} 3.05 \\ 4 \end{pmatrix} \cdot ksi \qquad \text{Compressive strength of column concrete (psi)}$$

$D = 30 \cdot in \qquad b_V = 30 \cdot in$

$d_V = 20.375 \cdot in$

$$V_c := \left[(0.0316 \cdot \beta) \cdot \sqrt{\frac{f_c}{ksi}} \cdot ksi \cdot b_V \cdot d_V \right] \qquad \begin{array}{l} \text{See Equation 5.8.3.3-3, pg. 5-70, Interim 2008.. for 1} \\ \text{shear plane} \end{array}$$

$$D = 30 \cdot in \qquad f_c = \begin{pmatrix} 3.05 \\ 4 \end{pmatrix} \cdot ksi \qquad V_c = \begin{pmatrix} 30.215 \\ 34.602 \end{pmatrix} \cdot kips \qquad \begin{array}{l} \text{Nominal shear strength of concrete alone} \\ \text{for corresponding column Dia.} \end{array}$$

5.) Calculate V_s ~ Strength Attributable to Shear Reinforcement:

$Stirrup_{size} = 0.375 \cdot in$ Dia. of Stirrup Steel (in.)

$A_V = 0.221 \cdot in^2$ Area of Stirrup Steel (in^2) .... 2 Legs

$s_{spa} = 6 \cdot in$ Spiral spacing / pitch (inches)

Calculate the Stirrup Angle in degrees:

Stirrup Angles (degrees)

$$\alpha := atan\left(\left(\frac{s_{spa}}{\pi \cdot d_c}\right)\right) \qquad \alpha = 4.283 \cdot deg$$

Texas
Transportation
Institute

LRFD Section 5

Project #: __424977__

Sponsor: __TxDOT__

Subject: __Circular Concrete Pier Shear Capacity - Accident #8 SH 14 ov IH-45__

$$V_s := \frac{A_v \cdot f_{yt} \cdot d_v \cdot (\cot(\theta) + \cot(\alpha)) \cdot \sin(\alpha)}{s_{spa}}$$

LFRD 5.8.3.3-4

$V_s = 47.701 \cdot \text{kips}$ 　　　 $D = 30 \cdot \text{in}$

$V_s = 47.701 \cdot \text{kips}$ 　　　 Nominal shear strength as per Section 5.8.3.3-4 pg. 5-61

6.) Calculate the Nominal Shear Capacity of Column for two Failure Plane Mechanism considering the strength of the concrete and the spiral reinforcing steel:

Concrete Shear Strength 　　　 Spiral Reinforcment Strength
(single shear plane) 　　　 (single shear plane)

$$V_c = \begin{pmatrix} 30.215 \\ 34.602 \end{pmatrix} \cdot \text{kips} \qquad V_s = 47.701 \cdot \text{kips}$$

$\phi_v := 0.9$ 　　　 (LRFD 5.5.4.2.1, Interm 2008, pg. 5-25

$V_r := (V_c + V_s) \cdot \phi_v \cdot 2$ 　　　 LRFD Eq. 5.8.3.3-1, Interim 2008, pg. 5-70, Shear Resistance 2 Shear planes

Column Dia. 　　　 Nominal Shear Capacity of Column (kips)

$D = 30 \cdot \text{in}$

$$V_r = \begin{pmatrix} 140.249 \\ 148.145 \end{pmatrix} \cdot \text{kips}$$ 　　　 3050 psi Concrete

4000 psi Concrete

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ACCIDENT #10: IH-20 OVER RABBIT CREEK

Texas Transportation Institute

LRFD Section 5

Project #: __424977__

Sponsor: __TxDOT__

Subject: __Circular Concrete Pier Shear Capacity - Accident #10 IH20 over Rabbit Creek__

1.) Given the following Design Data:

$$f_c := \begin{pmatrix} 3.05 \\ 4.00 \end{pmatrix} \cdot ksi$$

Compressive Strength of Concrete (ksi)

$$f_{yt} := 40ksi$$

yield strength of spiral reinforcement (ksi)

Diameter of Column Impacted:

$$D := 24in$$

Shear Reinforcement size:

$$Stirrup_{size} := \frac{2}{8}in$$

$$A_v := \pi \cdot \left(Stirrup_{size}^2\right) \cdot 0.25 \cdot 2$$

$$A_v = 0.098 \cdot in^2 \qquad \text{Area 2 Legs}$$

Size of Vertical Steel (8~#7's)

$$Vertical_{size} := 0.875in$$

Dia. of Longitudinal Steel (in.)

$$X := 2.25in$$

Distance to Stirrup Center (in.)
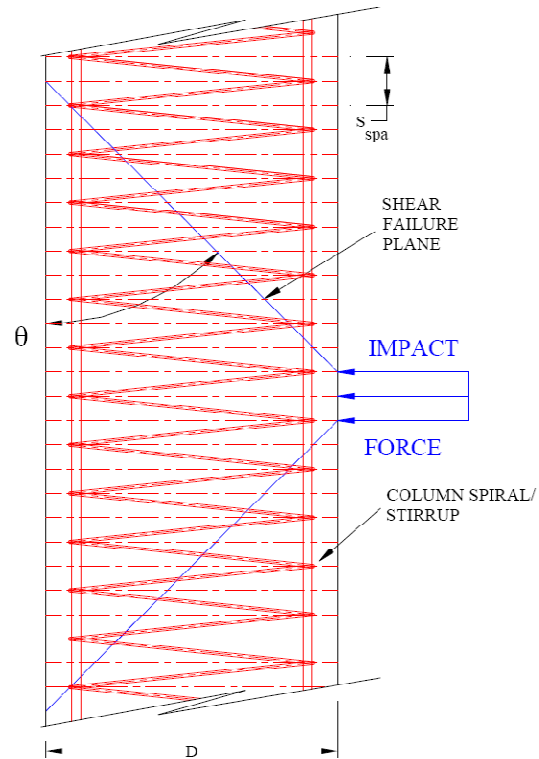
$$d_c := D - 2 \cdot X$$

Diameter of Spiral Steel

$$d_c = 19.5 \cdot in \qquad \text{Diameter of Spiral Steel (in.)}$$
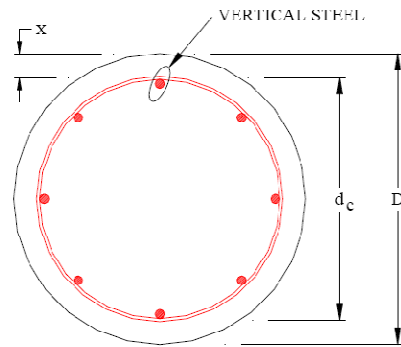
$$psi \equiv \frac{lbf}{in^2} \qquad ksi \equiv \frac{kip}{in^2} \qquad kips \equiv 1000lbf$$

COLUMN SECTION

COLUMN SIDE VIEW

Subject: __Circular Concrete Pier Shear Capacity - Accident #10 IH20 over Rabbit Creek__

$s_{spa} := 6\text{in}$ Pitch in Spiral Stirrup

$\gamma_{con} := 150\text{pcf}$ Unit weight of concrete (lbf/ft^3)

$b_v := D$ Width of section (see Figure C5.8.2.9-3 page 5-57)

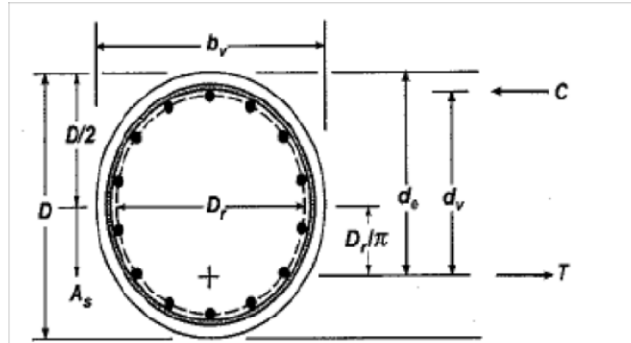$D_r := D - X \cdot 2 - \text{Stirrup}_{size} - \text{Vertical}_{size}$

$D_r = 18.375 \cdot \text{in}$

Figure C5.8.2.9-2 Illustration of Terms b_v, d_v, and d_e for Circular Sections.

$d_e := \dfrac{D}{2} + \dfrac{D_r}{\pi}$ $d_e = 17.849 \cdot \text{in}$ LRFD C5.8.2.9-2, pg. 5-67 Interim 2008

$d_v := 0.90 \cdot d_e$ see section commentary pag. 5-67   (LRFD C5.8.2.9)

$d_v = 16.064 \cdot \text{in}$

$a_g := 2\text{in}$ Maximum Aggregate size in Concrete

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Subject: __Circular Concrete Pier Shear Capacity - Accident #10 III20 over Rabbit Creek__

2.) Calculate the minimum transverse reinforcement as per Section 5.8.2.5-1 pg. 5-63 Interim 2008

$$A_{transneeded} := 0.0316 \sqrt{\frac{f_c}{ksi}} \cdot ksi \cdot b_v \cdot \frac{s_{spa}}{f_{yt}}$$

$$A_{transneeded} = \begin{pmatrix} 0.1987 \\ 0.2275 \end{pmatrix} \cdot in^2 \qquad A_v = 0.098 \cdot in^2 \qquad \text{Actual Area of Transverse Steel Provided (in}^2\text{)}$$

3.) Determine β & θ as per LRFD Section 5.8.3.4.2, Interim 2008, page 5-72

Calculate Crack Spacing Parameter sxe as per LRFD Eq. 5.8.3.4.2-5, Interim 2008, page 5-72

$$s_x := d_v \qquad \text{LRFD 5.8.3.4.2, Interim 2008, pg. 5-73}$$

$$s_{xe} := s_x \cdot \left(\frac{1.38in}{a_g + 0.63in} \right) \qquad s_{xe} = 8.429 \cdot in$$

$$\varepsilon_s := 0.006 \qquad \text{LRFD Section 5.8.3.4.2, pg. 5-74 Interim 2008, based on maximum strain from severe impact condition}$$

Therefore, Calculate β:

$$\beta := \frac{4.8}{(1 + 750 \cdot \varepsilon_s)} \cdot \frac{51}{\left(39 + \frac{s_{xe}}{in} \right)} \qquad \beta = 0.938 \qquad \text{LRFD Eq. 5.8.3.4.2-2, Interim 2008, pg. 5-72}$$

$$\theta := (29 + 3500 \cdot \varepsilon_s) deg \qquad \theta = 50 \cdot deg \qquad \text{LRFD Eq. 5.8.3.4.2-3, Interim 2008, pg 5-72}$$

Texas
Transportation
Institute

LRFD Section 5

Project #: __424977__

Sponsor: __TxDOT__

Subject: __Circular Concrete Pier Shear Capacity - Accident #10 IH20 over Rabbit Creek__

4.) Calculate V_c ~ Strength Attributable to Concrete as per Section 5.8.3.3 pg. 5-61:

with:

$$f_c = \begin{pmatrix} 3.05 \\ 4 \end{pmatrix} \cdot ksi \qquad \text{Compressive strength of column concrete (psi)}$$

$$D = 24 \cdot in \qquad b_v = 24 \cdot in$$

$$d_v = 16.064 \cdot in$$

$$V_c := \left[(0.0316 \cdot \beta) \cdot \sqrt{\frac{f_c}{ksi}} \cdot ksi \cdot b_v \cdot d_v \right] \qquad \begin{array}{l} \text{See Equation 5.8.3.3-3, pg. 5-70, Interim 2008.. for 1} \\ \text{shear plane} \end{array}$$

$$D - 24 \cdot in \qquad f_c = \begin{pmatrix} 3.05 \\ 4 \end{pmatrix} \cdot ksi \qquad V_c = \begin{pmatrix} 19.967 \\ 22.866 \end{pmatrix} \cdot kips \qquad \begin{array}{l} \text{Nominal shear strength of concrete alone} \\ \text{for corresponding column Dia.} \end{array}$$

5.) Calculate V_s ~ Strength Attributable to Shear Reinforcement:

$$Stirrup_{size} - 0.25 \cdot in \qquad \text{Dia. of Stirrup Steel (in.)}$$

$$A_v = 0.098 \cdot in^2 \qquad \text{Area of Stirrup Steel (in}^2) \dots 2 \text{ Legs}$$

$$s_{spa} = 6 \cdot in \qquad \text{Spiral spacing / pitch (inches)}$$

Calculate the Stirrup Angle in degrees:

Stirrup Angles (degrees)

$$\alpha := atan\left(\left(\frac{s_{spa}}{\pi \cdot d_c}\right)\right) \qquad \alpha = 5.594 \cdot deg$$

Subject: **Circular Concrete Pier Shear Capacity - Accident #10 IH20 over Rabbit Creek**

$$V_s := \frac{A_v \cdot f_{yt} \cdot d_v \cdot (\cot(\theta) + \cot(\alpha)) \cdot \sin(\alpha)}{s_{spa}}$$

LFRD 5.8.3.3-4

$V_s = 11.324 \cdot kips$ $D = 24 \cdot in$

$V_s = 11.324 \cdot kips$ Nominal shear strength as per Section 5.8.3.3-4 pg. 5-61

6.) Calculate the Nominal Shear Capacity of Column for two Failure Plane Mechanism considering the strength of the concrete and the spiral reinforcing steel:

Concrete Shear Strength   Spiral Reinforcment Strength
(single shear plane)   (single shear plane)

$$V_c = \begin{pmatrix} 19.967 \\ 22.866 \end{pmatrix} \cdot kips$$ $V_s = 11.324 \cdot kips$

$\phi_v := 0.9$ (LRFD 5.5.4.2.1, Interm 2008, pg. 5-25

$V_r := (V_c + V_s) \cdot \phi_v \cdot 2$ LRFD Eq. 5.8.3.3-1, Interim 2008, pg. 5-70, Shear Resistance 2 Shear planes

Column Dia.   Nominal Shear Capacity of Column (kips)

$D = 24 \cdot in$

$$V_r = \begin{pmatrix} 56.323 \\ 61.541 \end{pmatrix} \cdot kips$$ 3050 psi Concrete

4000 psi Concrete

ACCIDENT #17: IH-90 BRIDGE #53812, MN

Subject: **Circular Concrete Pier Shear Capacity - Accident #17 IH90 Bridge, #53812, Minn.**

1.) Given the following Design Data:

$$f_c := \begin{pmatrix} 4.30 \\ 5.50 \end{pmatrix} \cdot ksi$$

Compressive Strength of Concrete (ksi)

$f_{yt} := 60ksi$ yield strength of spiral reinforcement (ksi)

Diameter of Column Impacted:

$D := 32in$

Shear Reinforcement size:

$$Stirrup_{size} := \frac{4}{8} in$$

$$A_v := \pi \cdot \left(Stirrup_{size}^2 \right) \cdot 0.25 \cdot 2$$

$A_v = 0.393 \cdot in^2$ Area 2 Legs

Size of Vertical Steel (9~#9's)

$Vertical_{size} := 1.128in$ Dia. of Longitudinal Steel (in.)

$X := 4.75in$ Distance to Stirrup Center (in.)

$d_c := D - 2 \cdot X$ Diameter of Spiral Steel

$d_c = 22.5 \cdot in$ Diameter of Spiral Steel (in.)
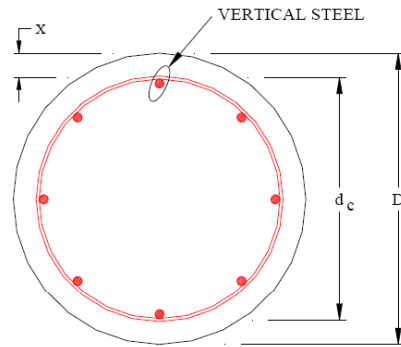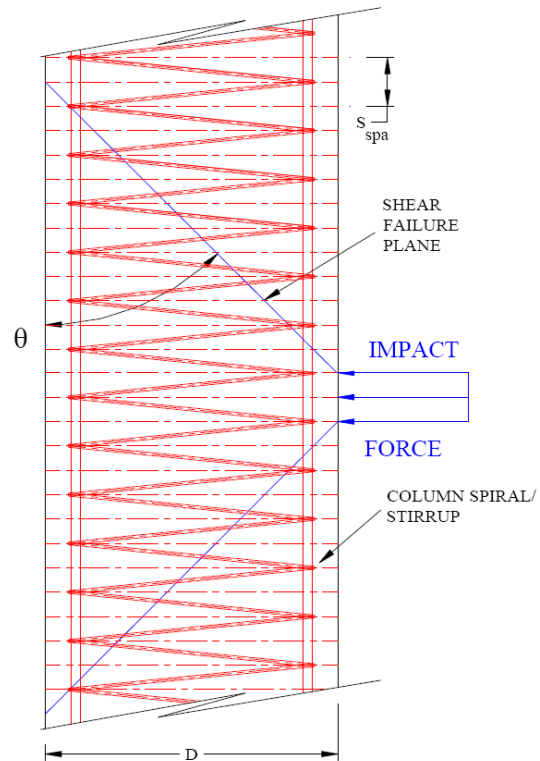
$$psi \equiv \frac{lbf}{in^2} \qquad ksi \equiv \frac{kip}{in^2} \qquad kips \equiv 1000lbf$$

VERTICAL STEEL

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Subject: __Circular Concrete Pier Shear Capacity - Accident #17 IH90 Bridge, #53812, Minn.__

$s_{spa} := 6in$ Pitch in Spiral Stirrup

$\gamma_{con} := 150pcf$ Unit weight of concrete (lbf/ft³)

$b_v := D$ Width of section (see Figure C5.8.2.9-3 page 5-57)

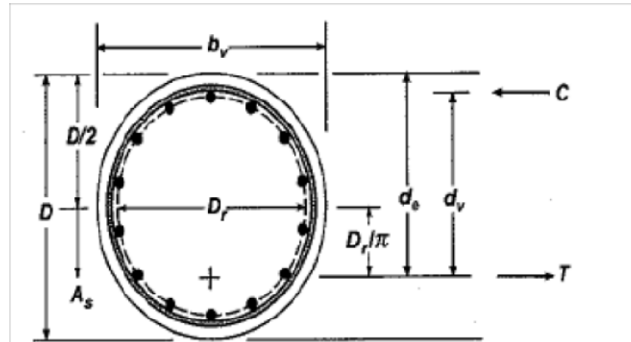$D_r := D - X \cdot 2 - Stirrup_{size} - Vertical_{size}$

$D_r = 20.872 \cdot in$

Figure C5.8.2.9-2 Illustration of Terms b_v, d_r, and d_e for Circular Sections.

$d_e :- \dfrac{D}{2} + \dfrac{D_r}{\pi}$ $d_e - 22.644 \cdot in$ LRFD C5.8.2.9-2, pg. 5-67 Interim 2008

$d_v := 0.90 \cdot d_e$ see section commentary pag. 5-67   (LRFD C5.8.2.9)

$d_v = 20.379 \cdot in$

$a_g := 2in$ Maximum Aggregate size in Concrete

Subject: __Circular Concrete Pier Shear Capacity - Accident #17 IH90 Bridge, #53812, Minn.__

2.) Calculate the minimum transverse reinforcement as per Section 5.8.2.5-1 pg. 5-63 Interim 2008

$$A_{transneeded} := 0.0316 \sqrt{\frac{f_c}{ksi}} \cdot ksi \cdot b_v \cdot \frac{s_{spa}}{f_{yt}}$$

$$A_{transneeded} = \begin{pmatrix} 0.2097 \\ 0.2371 \end{pmatrix} \cdot in^2 \qquad A_v = 0.393 \cdot in^2 \qquad \text{Actual Area of Transverse Steel Provided (in}^2\text{)}$$

3.) Determine β & θ as per LRFD Section 5.8.3.4.2, Interim 2008, page 5-72

Calculate Crack Spacing Parameter sxe as per LRFD Eq. 5.8.3.4.2-5, Interim 2008, page 5-72

$$\beta := 2.0 \qquad \text{See LRFD 5.8.3.4.1, Interim 2008, pg 5-72}$$

$$\theta := 45deg$$

Subject: __Circular Concrete Pier Shear Capacity - Accident #17 IH90 Bridge, #53812, Minn.__

4.) Calculate V_c ~ Strength Attributable to Concrete as per Section 5.8.3.3 pg. 5-61:

with:

$$f_c = \begin{pmatrix} 4.3 \\ 5.5 \end{pmatrix} \cdot ksi \qquad \text{Compressive strength of column concrete (psi)}$$

$$D = 32 \cdot in \qquad b_V = 32 \cdot in$$

$$d_V = 20.379 \cdot in$$

$$V_c := \left[(0.0316 \cdot \beta) \cdot \sqrt{\frac{f_c}{ksi}} \cdot ksi \cdot b_V \cdot d_V \right] \qquad \begin{array}{l} \text{See Equation 5.8.3.3-3, pg. 5-70, Interim 2008.. for 1} \\ \text{shear plane} \end{array}$$

$$D = 32 \cdot in \qquad f_c = \begin{pmatrix} 4.3 \\ 5.5 \end{pmatrix} \cdot ksi \qquad V_c = \begin{pmatrix} 85.466 \\ 96.658 \end{pmatrix} \cdot kips \qquad \begin{array}{l} \text{Nominal shear strength of concrete alone} \\ \text{for corresponding column Dia.} \end{array}$$

5.) Calculate V_s ~ Strength Attributable to Shear Reinforcement:

$Stirrup_{size} = 0.5 \cdot in \qquad$ Dia. of Stirrup Steel (in.)

$A_V = 0.393 \cdot in^2 \qquad$ Area of Stirrup Steel (in^2) .... 2 Legs

$s_{spa} = 6 \cdot in \qquad$ Spiral spacing / pitch (inches)

Calculate the Stirrup Angle in degrees:

$$\alpha := atan\left(\left(\frac{s_{spa}}{\pi \cdot d_c} \right) \right) \qquad \begin{array}{l} \text{Stirrup Angles (degrees)} \\ \alpha = 4.852 \cdot deg \end{array}$$

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LRFD Section 5

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Sponsor: __TxDOT__

Subject: __Circular Concrete Pier Shear Capacity - Accident #17 IH90 Bridge, #53812, Minn.__

$$V_s := \frac{A_v \cdot f_{yt} \cdot d_v \cdot (\cot(\theta) + \cot(\alpha)) \cdot \sin(\alpha)}{s_{spa}}$$ LFRD 5.8.3.3-4

$V_s = 86.512 \cdot kips$ $D = 32 \cdot in$

$V_s = 86.512 \cdot kips$ Nominal shear strength as per Section 5.8.3.3-4 pg. 5-61

6.) Calculate the Nominal Shear Capacity of Column for two Failure Plane Mechanism considering the strength of the concrete and the spiral reinforcing steel:

Concrete Shear Strength   Spiral Reinforcment Strength
(single shear plane)   (single shear plane)

$V_c = \begin{pmatrix} 85.466 \\ 96.658 \end{pmatrix} \cdot kips$ $V_s = 86.512 \cdot kips$

$\phi_v := 0.9$ (LRFD 5.5.4.2.1, Interm 2008, pg. 5-25

$V_r := (V_c + V_s) \cdot \phi_v \cdot 2$ LRFD Eq. 5.8.3.3-1, Interim 2008, pg. 5-70,
                                        Shear Resistance 2 Shear planes

Column Dia.   Nominal Shear Capacity of
                     Column (kips)

$D = 32 \cdot in$

$V_r = \begin{pmatrix} 309.56 \\ 329.706 \end{pmatrix} \cdot kips$ 3050 psi Concrete

                                        4000 psi Concrete

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ACCIDENT #18: FM 1402 OVER IH-30

Texas Transportation Institute

LRFD Section 5

Project #: __424977__

Sponsor: __TxDOT__

Subject: __Circular Concrete Pier Shear Capacity - Accident #18 FM #1402 over IH-30__

1.) Given the following Design Data:

$$f_c := \begin{pmatrix} 3.60 \\ 4.00 \end{pmatrix} \cdot ksi \qquad \text{Compressive Strength of Concrete (ksi)}$$

$$f_{yt} := 60ksi \qquad \text{yield strength of spiral reinforcement (ksi)}$$

Diameter of Column Impacted:

$$D := 30in$$

Shear Reinforcement size:

$$Stirrup_{size} := \frac{3}{8}in$$

$$A_v := \pi \cdot \left(Stirrup_{size}^2\right) \cdot 0.25 \cdot 2$$

$$A_v = 0.221 \cdot in^2 \qquad \text{Area 2 Legs}$$

Size of Vertical Steel (8~#9's)

$$Vertical_{size} := 1.128in \qquad \text{Dia. of Longitudinal Steel (in.)}$$

$$X := 3.0in \qquad \text{Distance to Stirrup Center (in.)}$$

$$d_c := D - 2 \cdot X \qquad \text{Diameter of Spiral Steel}$$

$$d_c = 24 \cdot in \qquad \text{Diameter of Spiral Steel (in.)}$$
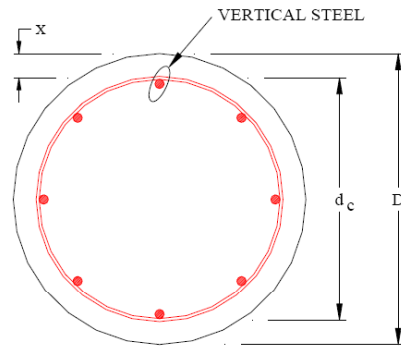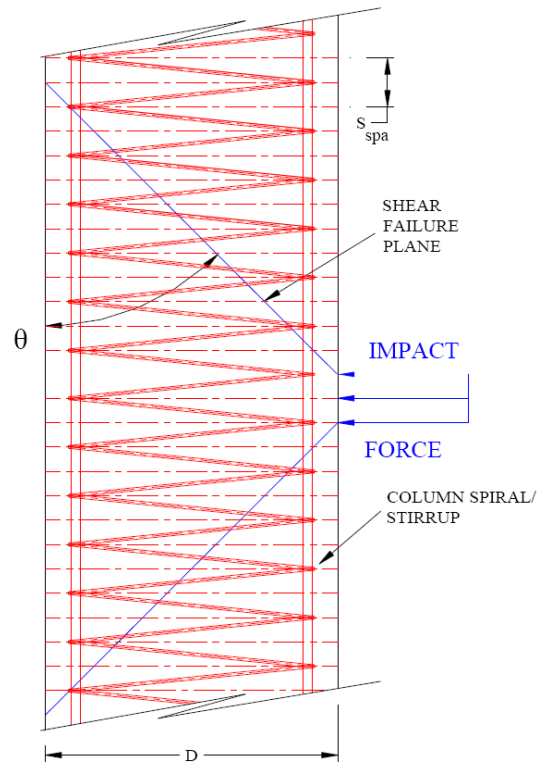
$$psi \equiv \frac{lbf}{in^2} \qquad ksi \equiv \frac{kip}{in^2} \qquad kips \equiv 1000lbf$$

VERTICAL STEEL

COLUMN SECTION

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Sponsor: __TxDOT__

Subject: __Circular Concrete Pier Shear Capacity - Accident #18 FM #1402 over IH-30__

$s_{spa} := 6in$ Pitch in Spiral Stirrup

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

$b_v := D$ Width of section (see Figure C5.8.2.9-3 page 5-57)

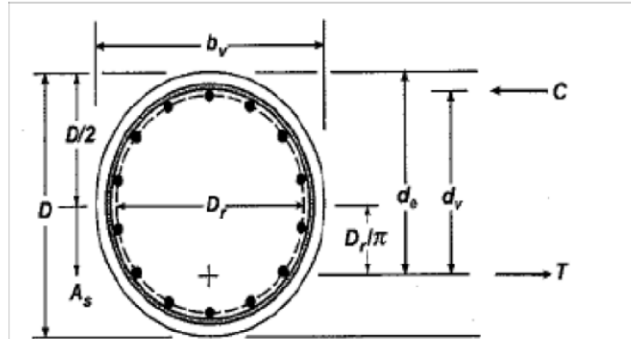$D_r := D - X \cdot 2 - Stirrup_{size} - Vertical_{size}$

$D_r = 22.497 \cdot in$

Figure C5.8.2.9-2 Illustration of Terms b_v, d_v and d_e for Circular Sections.

$d_e := \dfrac{D}{2} + \dfrac{D_r}{\pi}$ $d_e = 22.161 \cdot in$ LRFD C5.8.2.9-2, pg. 5-67 Interim 2008

$d_v := 0.90 \cdot d_e$ see section commentary pag. 5-67 (LRFD C5.8.2.9)

$d_v = 19.945 \cdot in$

$a_g := 2in$ Maximum Aggregate size in Concrete

Subject: **Circular Concrete Pier Shear Capacity - Accident #18 FM #1402 over IH-30**

2.) Calculate the minimum transverse reinforcement as per Section 5.8.2.5-1 pg. 5-63 Interim 2008

$$A_{transneeded} := 0.0316 \sqrt{\frac{f_c}{ksi}} \cdot ksi \cdot b_v \cdot \frac{s_{spa}}{f_{yt}}$$

$$A_{transneeded} = \begin{pmatrix} 0.1799 \\ 0.1896 \end{pmatrix} \cdot in^2 \qquad A_v = 0.221 \cdot in^2 \qquad \text{Actual Area of Transverse Steel Provided} \\ (in^2)$$

3.) Determine β & θ as per LRFD Section 5.8.3.4.1, Interim 2008, page 5-72

$\beta := 2.0$ 		Based on LRFD 5.8.3.4.1, Interim 2008, page 5-72

$\theta := 45 deg$

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Subject: __Circular Concrete Pier Shear Capacity - Accident #18 FM #1402 over IH-30__

4.) Calculate V_c ~ Strength Attributable to Concrete as per Section 5.8.3.3 pg. 5-61:

with:

$f'_c = \begin{pmatrix} 3.6 \\ 4 \end{pmatrix} \cdot ksi$ Compressive strength of column concrete (psi)

$D = 30 \cdot in$ $b_v = 30 \cdot in$

$d_v = 19.945 \cdot in$

$$V_c := \left[(0.0316 \cdot \beta) \cdot \sqrt{\frac{f'_c}{ksi}} \cdot ksi \cdot b_v \cdot d_v \right]$$

See Equation 5.8.3.3-3, pg. 5-70, Interim 2008.. for 1 shear plane

$D = 30 \cdot in$ $f'_c = \begin{pmatrix} 3.6 \\ 4 \end{pmatrix} \cdot ksi$ $V_c = \begin{pmatrix} 71.75 \\ 75.631 \end{pmatrix} \cdot kips$

Nominal shear strength of concrete alone for corresponding column Dia.

5.) Calculate V_s ~ Strength Attributable to Shear Reinforcement:

$Stirrup_{size} = 0.375 \cdot in$ Dia. of Stirrup Steel (in.)

$A_v = 0.221 \cdot in^2$ Area of Stirrup Steel (in^2) .... 2 Legs

$s_{spa} = 6 \cdot in$ Spiral spacing / pitch (inches)

Calculate the Stirrup Angle in degrees:

Stirrup Angles (degrees)

$\alpha := atan\left(\left(\frac{s_{spa}}{\pi \cdot d_c} \right) \right)$ $\alpha = 4.55 \cdot deg$

Subject: __Circular Concrete Pier Shear Capacity - Accident #18 FM #1402 over IH-30__

$$V_s := \frac{A_v \cdot f_{yt} \cdot d_v \cdot (\cot(\theta) + \cot(\alpha)) \cdot \sin(\alpha)}{s_{spa}}$$ LFRD 5.8.3.3-4

$V_s = 47.413 \cdot kips$ $D = 30 \cdot in$

$V_s = 47.413 \cdot kips$ Nominal shear strength as per Section 5.8.3.3-4 pg. 5-61

6.) Calculate the Nominal Shear Capacity of Column for two Failure Plane Mechanism considering the strength of the concrete and the spiral reinforcing steel:

Concrete Shear Strength   Spiral Reinforcment Strength
(single shear plane)   (single shear plane)

$$V_c = \begin{pmatrix} 71.75 \\ 75.631 \end{pmatrix} \cdot kips$$ $V_s = 47.413 \cdot kips$

$\phi_v := 0.9$ (LRFD 5.5.4.2.1, Interm 2008, pg. 5-25

$V_r := (V_c + V_s) \cdot \phi_v \cdot 2$ LRFD Eq. 5.8.3.3-1, Interim 2008, pg. 5-70, Shear Resistance 2 Shear planes

Column Dia.   Nominal Shear Capacity of Column (kips)

$D = 30 \cdot in$

$$V_r = \begin{pmatrix} 214.493 \\ 221.479 \end{pmatrix} \cdot kips$$ 3050 psi Concrete

4000 psi Concrete

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ACCIDENT #19: BRIDGE OVER IH-20 AT MILE POST 519

Texas Transportation Institute   **LRFD Section 5**

Project #: __424977__

Sponsor: __TxDOT__

Subject: **Circular Concrete Pier Shear Capacity - Accident #19 Bridge over IH-20 @ MP 519, Canton, TX**

1.) Given the following Design Data:

$$f_c := \begin{pmatrix} 3.05 \\ 4.00 \end{pmatrix} \cdot ksi$$ Compressive Strength of Concrete (ksi)

$f_{yt} := 40 ksi$ yield strength of spiral reinforcement (ksi)

Diameter of Column Impacted:

$D := 30 in$

Shear Reinforcement size:

$Stirrup_{size} := \dfrac{2}{8} in$

$A_v := \pi \cdot \left(Stirrup_{size}^2 \right) \cdot 0.25 \cdot 2$

$A_v = 0.098 \cdot in^2$ Area 2 Legs

Size of Vertical Steel (8~#9's)

$Vertical_{size} := 1.128 in$ Dia. of Longitudinal Steel (in.)

$X := 2.25 in$ Distance to Stirrup Center (in.)

$d_c := D - 2 \cdot X$ Diameter of Spiral Steel

$d_c = 25.5 \cdot in$ Diameter of Spiral Steel (in.)

$psi \equiv \dfrac{lbf}{in^2}$ $ksi \equiv \dfrac{kip}{in^2}$ $kips \equiv 1000 lbf$

VERTICAL STEEL

COLUMN SECTION

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LRFD Section 5

Project #: __424977__

Sponsor: __TxDOT__

Subject: __Circular Concrete Pier Shear Capacity - Accident #19 Bridge over IH-20 @ MP 519, Canton, TX__

$s_{spa} := 6in$ Pitch in Spiral Stirrup

$\gamma_{con} := 150pcf$ Unit weight of concrete (lbf/ft³)

$b_v := D$ Width of section
(see Figure C5.8.2.9-3 page 5-57)

$D_r := D - X \cdot 2 - Stirrup_{size} - Vertical_{size}$

$D_r = 24.122 \cdot in$
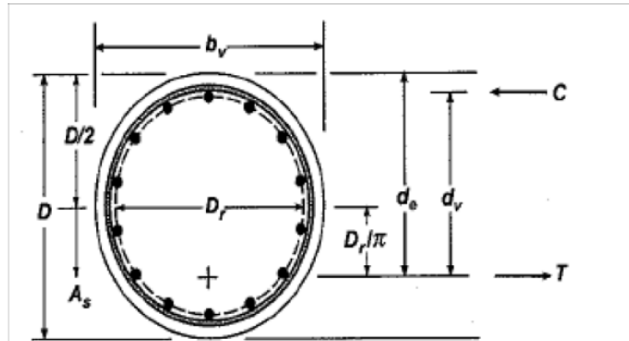
Figure C5.8.2.9-2 Illustration of Terms b_v, d_v and d_e for Circular Sections.

$d_e := \dfrac{D}{2} + \dfrac{D_r}{\pi}$ $d_e = 22.678 \cdot in$ LRFD C5.8.2.9-2, pg. 5-67 Interim 2008

$d_v := 0.90 \cdot d_e$ see section commentary pag. 5-67   (LRFD C5.8.2.9)

$d_v = 20.41 \cdot in$

$a_g := 2in$ Maximum Aggregate size in Concrete

Subject: **Circular Concrete Pier Shear Capacity - Accident #19 Bridge over IH-20 @ MP 519, Canton, TX**

2.) Calculate the minimum transverse reinforcement as per Section 5.8.2.5-1 pg. 5-63 Interim 2008

$$A_{transneeded} := 0.0316 \sqrt{\frac{f_c}{ksi}} \cdot ksi \cdot b_v \cdot \frac{s_{spa}}{f_{yt}}$$

$$A_{transneeded} = \begin{pmatrix} 0.2483 \\ 0.2844 \end{pmatrix} \cdot in^2 \qquad A_v = 0.098 \cdot in^2 \qquad \text{Actual Area of Transverse Steel Provided} \\ (in^2)$$

3.) Determine β & θ as per LRFD Section 5.8.3.4.2, Interim 2008, page 5-72

Calculate Crack Spacing Parameter sxe as per LRFD Eq. 5.8.3.4.2-5, Interim 2008, page 5-72

$$s_x := d_v \qquad \text{LRFD 5.8.3.4.2, Interim 2008, pg. 5-73}$$

$$s_{xe} := s_x \cdot \left(\frac{1.38in}{a_g + 0.63in} \right) \qquad s_{xe} = 10.71 \cdot in$$

$$\varepsilon_s := 0.006 \qquad \text{LRFD Section 5.8.3.4.2, pg. 5-74 Interim 2008, based on maximum strain from severe impact condition}$$

Therefore, Calculate β:

$$\beta := \frac{4.8}{(1 + 750 \cdot \varepsilon_s)} \cdot \frac{51}{\left(39 + \frac{s_{xe}}{in} \right)} \qquad \beta = 0.895 \qquad \text{LRFD Eq. 5.8.3.4.2-2, Interim 2008, pg. 5-72}$$

$$\theta := (29 + 3500 \cdot \varepsilon_s) deg \qquad \theta = 50 \cdot deg \qquad \text{LRFD Eq. 5.8.3.4.2-3, Interim 2008, pg 5-72}$$

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LRFD Section 5

Project #: __424977__

Sponsor: __TxDOT__

Subject: __Circular Concrete Pier Shear Capacity - Accident #19 Bridge over IH-20 @ MP 519, Canton, TX__

4.) Calculate V_c ~ Strength Attributable to Concrete as per Section 5.8.3.3 pg. 5-61:

with:

$$f_c = \begin{pmatrix} 3.05 \\ 4 \end{pmatrix} \cdot ksi \qquad \text{Compressive strength of column concrete (psi)}$$

$$D = 30 \cdot in \qquad b_v = 30 \cdot in$$

$$d_v = 20.41 \cdot in$$

$$V_c := \left[(0.0316 \cdot \beta) \cdot \sqrt{\frac{f_c}{ksi}} \cdot ksi \cdot b_v \cdot d_v \right] \qquad \begin{array}{l} \text{See Equation 5.8.3.3-3, pg. 5-70, Interim 2008.. for 1} \\ \text{shear plane} \end{array}$$

$$D = 30 \cdot in \qquad f_c = \begin{pmatrix} 3.05 \\ 4 \end{pmatrix} \cdot ksi \qquad V_c = \begin{pmatrix} 30.256 \\ 34.65 \end{pmatrix} \cdot kips \qquad \begin{array}{l} \text{Nominal shear strength of concrete alone} \\ \text{for corresponding column Dia.} \end{array}$$

5.) Calculate V_s ~ Strength Attributable to Shear Reinforcement:

$$Stirrup_{size} = 0.25 \cdot in \qquad \text{Dia. of Stirrup Steel (in.)}$$

$$A_v = 0.098 \cdot in^2 \qquad \text{Area of Stirrup Steel (in}^2) \dots 2 \text{ Legs}$$

$$s_{spa} = 6 \cdot in \qquad \text{Spiral spacing / pitch (inches)}$$

Calculate the Stirrup Angle in degrees:

Stirrup Angles (degrees)

$$\alpha := atan\left(\left(\frac{s_{spa}}{\pi \cdot d_c} \right) \right) \qquad \alpha = 4.283 \cdot deg$$

Subject: __Circular Concrete Pier Shear Capacity - Accident #19 Bridge over IH-20 @ MP 519, Canton, TX__

$$V_s := \frac{A_v \cdot f_{yt} \cdot d_v \cdot (\cot(\theta) + \cot(\alpha)) \cdot \sin(\alpha)}{s_{spa}}$$ LFRD 5.8.3.3-4

$V_s = 14.158 \cdot kips$ $D = 30 \cdot in$

$V_s = 14.158 \cdot kips$ Nominal shear strength as per Section 5.8.3.3-4 pg. 5-61

6.) Calculate the Nominal Shear Capacity of Column for two Failure Plane Mechanism considering the strength of the concrete and the spiral reinforcing steel:

Concrete Shear Strength (single shear plane)   Spiral Reinforcment Strength (single shear plane)

$$V_c = \begin{pmatrix} 30.256 \\ 34.65 \end{pmatrix} \cdot kips$$ $V_s = 14.158 \cdot kips$

$\phi_v := 0.9$ (LRFD 5.5.4.2.1, Interm 2008, pg. 5-25

$$V_r := (V_c + V_s) \cdot \phi_v \cdot 2$$ LRFD Eq. 5.8.3.3-1, Interim 2008, pg. 5-70, Shear Resistance 2 Shear planes

Column Dia.   Nominal Shear Capacity of Column (kips)

$D = 30 \cdot in$

$$V_r = \begin{pmatrix} 79.947 \\ 87.855 \end{pmatrix} \cdot kips$$ 3050 psi Concrete

4000 psi Concrete

VARIABLE SIZE PIER SHEAR CAPACITY (LRFD)

Texas Transportation Institute

LRFD Section 5

Subject: __Variable Size Pier Shear Capacity (LRFD)__

1.) Given the following Design Data:

$f'_c := 3600 psi$ Compressive Strength of Concrete (ksi)

$f_{yt} := 60 ksi$ yield strength of spiral reinforcement (ksi)

Variable Diameters of Pier Impacted:

$$D := \begin{pmatrix} 24 \\ 30 \\ 36 \\ 42 \\ 48 \\ 54 \\ 60 \\ 66 \\ 72 \end{pmatrix} in$$

Shear Reinforcement (Spiral Stirrup) size Used in Analysis for Diameter "D":

$$Stirrup_{size} := \begin{pmatrix} 0.375 \\ 0.375 \\ 0.375 \\ 0.500 \\ 0.500 \\ 0.500 \\ 0.500 \\ 0.500 \\ 0.625 \end{pmatrix} in$$

VERTICAL STEEL

COLUMN SECTION

SHEAR FAILURE PLANE

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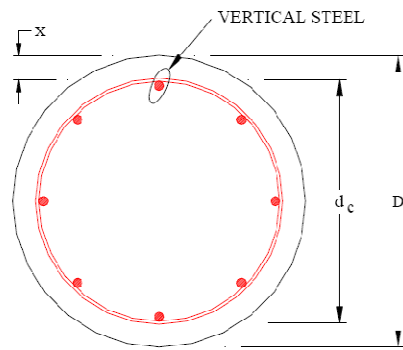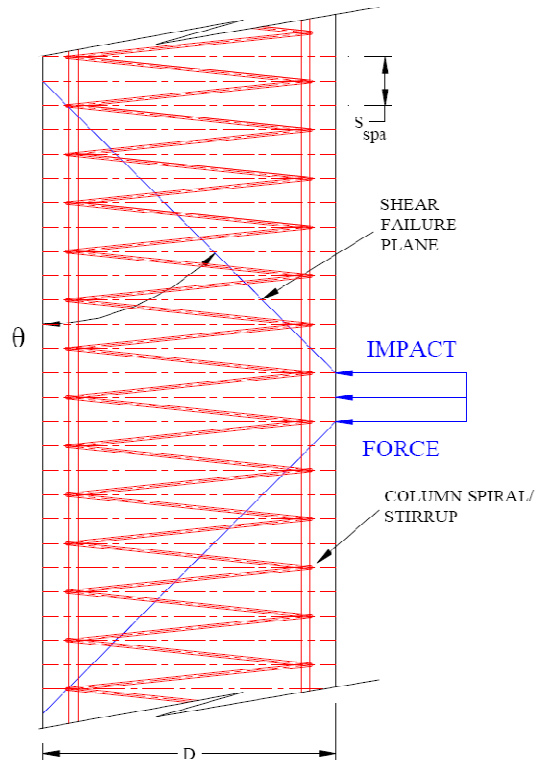
COLUMN SIDE VIEW

Subject: __Variable Size Pier Shear Capacity (LRFD)__

Area on Shear Reinfrorcement (in^2)

$$A_v := \pi \cdot \left(\text{Stirrup}_{size}^{2} \right) \cdot 0.25 \cdot 2$$

$$A_v = \begin{pmatrix} 0.221 \\ 0.221 \\ 0.221 \\ 0.393 \\ 0.393 \\ 0.393 \\ 0.393 \\ 0.393 \\ 0.614 \end{pmatrix} \cdot in^2 \qquad \text{Area 2 Legs}$$

Global Units & Setting:

ORIGIN := 1

$$psi \equiv \frac{lbf}{in^2} \qquad ksi \equiv \frac{kip}{in^2} \qquad kips \equiv 1000lbf$$
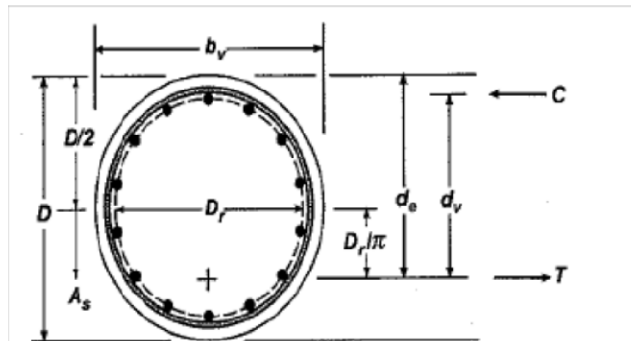
Figure C5.8.2.9-2 Illustration of Terms b_v, d_v and d_e for Circular Sections.

Size of Vertical Steel (8~#9's)

$\text{Vertical}_{size} := 1.128in$ Dia. of Longitudinal Steel (in.)

$X := 3.0in$ Distance to Stirrup Center (in.)

$$d_c := D - 2 \cdot X \qquad d_c = \begin{pmatrix} 18 \\ 24 \\ 30 \\ 36 \\ 42 \\ 48 \\ 54 \\ 60 \\ 66 \end{pmatrix} \cdot in \qquad \text{Diameter of Spiral Steel (in.)}$$

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Subject: __Variable Size Pier Shear Capacity (LRFD)__

$s_{spa} := 6in$ Pitch in Spiral Stirrup (in.)

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

$b_v := D$ Width of section
(see Figure C5.8.2.9-3 page 5-57)

$D_r := D - X \cdot 2 - Stirrup_{size} - Vertical_{size}$

$$D_r = \begin{pmatrix} 16.497 \\ 22.497 \\ 28.497 \\ 34.372 \\ 40.372 \\ 46.372 \\ 52.372 \\ 58.372 \\ 64.247 \end{pmatrix} \cdot in$$

$$d_e := \frac{D}{2} + \frac{D_r}{\pi}$$ LRFD C5.8.2.9-2, pg. 5-67 Interim 2008

$$d_e - \begin{pmatrix} 17.251 \\ 22.161 \\ 27.071 \\ 31.941 \\ 36.851 \\ 41.761 \\ 46.671 \\ 51.58 \\ 56.45 \end{pmatrix} \cdot in$$

Subject: __Variable Size Pier Shear Capacity (LRFD)__

$d_v := 0.90 \cdot d_e$ see section commentary pag. 5-67   (LRFD C5.8.2.9)

$a_g := 2\text{in}$ Maximum Aggregate size in Concrete

2.) Calculate the minimum transverse reinforcement as per Section 5.8.2.5-1 pg. 5-63 Interim 2008:

$$A_{transneeded} := 0.0316 \sqrt{\frac{f_c}{ksi}} \cdot ksi \cdot b_v \cdot \frac{s_{spa}}{f_{yt}} \qquad \text{Section 5.8.2.5-1, Interim 2008}$$

$$A_{transneeded} = \begin{pmatrix} 0.144 \\ 0.18 \\ 0.216 \\ 0.252 \\ 0.288 \\ 0.324 \\ 0.36 \\ 0.396 \\ 0.432 \end{pmatrix} \cdot in^2 \qquad A_v = \begin{pmatrix} 0.221 \\ 0.221 \\ 0.221 \\ 0.393 \\ 0.393 \\ 0.393 \\ 0.393 \\ 0.393 \\ 0.614 \end{pmatrix} \cdot in^2 \qquad \begin{array}{l}\text{Actual Area of Transverse} \\ \text{Steel Provided} \\ (in^2)\end{array}$$

3.) Determine β & θ as per LRFD Section 5.8.3.4.2, Interim 2008, page 5-72

Calculate Crack Spacing Parameter sxe as per LRFD Eq. 5.8.3.4.2-5, Interim 2008, page 5-72

$s_x := d_v$ LRFD 5.8.3.4.2, Interim 2008, pg. 5-73

$$s_{xe} := s_x \cdot \left(\frac{1.38\text{in}}{a_g + 0.63\text{in}} \right)$$

Subject: __Variable Size Pier Shear Capacity (LRFD)__

$\varepsilon_s := 0.006$ LRFD Section 5.8.3.4.2, pg. 5-74 Interim 2008, based on maximum strain from severe impact condition

Therefore, Calculate β:

$$\beta_1 := \frac{4.8}{\left(1 + 750 \cdot \varepsilon_s\right)} \cdot \frac{51}{\left(39 + \dfrac{s_{xe}}{in}\right)}$$

$$\beta_1 = \begin{pmatrix} 0.944 \\ 0.9 \\ 0.86 \\ 0.823 \\ 0.789 \\ 0.758 \\ 0.729 \\ 0.702 \\ 0.678 \end{pmatrix}$$ LRFD Eq. 5.8.3.4.2-2, Interim 2008, pg. 5-72

$i := 1..9$

$\theta_1 := \left(29 + 3500 \cdot \varepsilon_s\right)\deg$ $\theta_1 = 50 \cdot \deg$ LRFD Eq. 5.8.3.4.2-3, Interim 2008, pg 5-72

Check β & θ considering the amount of transverse steel provided versus the amount required:

$$\beta_i := \left| \begin{array}{l} 2.0 \quad \text{if } \overrightarrow{A_{v_i} \geq \left(0.95 \cdot A_{transneeded_i}\right)} \\[2mm] \beta_{1_i} \quad \text{otherwise} \end{array} \right.$$

$$\theta_i := \left| \begin{array}{l} 45\deg \quad \text{if } \overrightarrow{\left[A_{v_i} \geq \left(0.95 \cdot A_{transneeded_i}\right)\right]} \\[2mm] \theta_1 \quad \text{otherwise} \end{array} \right.$$

Tabulated β & θ Values Used in the Analyses for the different Pier Sizes:

$$\theta = \begin{pmatrix} 45 \\ 45 \\ 45 \\ 45 \\ 45 \\ 45 \\ 45 \\ 45 \\ 45 \end{pmatrix} \deg \qquad \beta = \begin{pmatrix} 2 \\ 2 \\ 2 \\ 2 \\ 2 \\ 2 \\ 2 \\ 2 \\ 2 \end{pmatrix}$$

Subject: **Variable Size Pier Shear Capacity (LRFD)**

4.) Calculate V_c ~ Strength Attributable to Concrete as per Section 5.8.3.3 pg. 5-61:

with: $f_c = 3600 \cdot psi$

$$D = \begin{pmatrix} 24 \\ 30 \\ 36 \\ 42 \\ 48 \\ 54 \\ 60 \\ 66 \\ 72 \end{pmatrix} \cdot in \qquad d_v = \begin{pmatrix} 15.526 \\ 19.945 \\ 24.364 \\ 28.747 \\ 33.166 \\ 37.585 \\ 42.003 \\ 46.422 \\ 50.805 \end{pmatrix} \cdot in \qquad b_v = \begin{pmatrix} 24 \\ 30 \\ 36 \\ 42 \\ 48 \\ 54 \\ 60 \\ 66 \\ 72 \end{pmatrix} \cdot in$$

...sive strength of column concrete (psi)

$$V_c := \left[(0.0316 \cdot \beta) \cdot \sqrt{\frac{f_c}{ksi}} \cdot ksi \cdot b_v \cdot d_v \right]$$

See Equation 5.8.3.3-3, pg. 5-70, Interim 2008.. for 1 shear plane

$$f_c = 3.6 \cdot ksi \qquad D = \begin{pmatrix} 24 \\ 30 \\ 36 \\ 42 \\ 48 \\ 54 \\ 60 \\ 66 \\ 72 \end{pmatrix} \cdot in \qquad V_c = \begin{pmatrix} 44.683 \\ 71.75 \\ 105.176 \\ 144.78 \\ 190.897 \\ 243.373 \\ 302.207 \\ 367.4 \\ 438.643 \end{pmatrix} \cdot kips$$

Nominal shear strength of concrete alone for corresponding column Dia.

Subject: __Variable Size Pier Shear Capacity (LRFD)__

$f_{yt} = 60\,ksi$ Yield strength of stirrup steel (ksi)

Calculate the Stirrup Angle in degrees:

$$\alpha := atan\left(\left(\frac{s_{spa}}{\pi \cdot d_c}\right)\right)$$

$$\alpha = \begin{pmatrix} 6.057 \\ 4.55 \\ 3.643 \\ 3.037 \\ 2.604 \\ 2.279 \\ 2.026 \\ 1.823 \\ 1.658 \end{pmatrix} \cdot deg$$ Stirrup Angles (degrees)

$s_{spa} = 6 \cdot in$ Spiral spacing / pitch (inches)

$$V_s := \frac{\overrightarrow{A_v \cdot f_{yt} \cdot d_v \cdot (cot(\theta) + cot(\alpha)) \cdot sin(\alpha)}}{s_{spa}}$$ LFRD 5.8.3.3-4

$$V_s = \begin{pmatrix} 37.723 \\ 47.413 \\ 57.128 \\ 118.711 \\ 136.023 \\ 153.346 \\ 170.674 \\ 188.008 \\ 320.625 \end{pmatrix} \cdot kips$$ Nominal shear strength as per Section 5.8.3.3-4 pg. 5-61

$$D = \begin{pmatrix} 24 \\ 30 \\ 36 \\ 42 \\ 48 \\ 54 \\ 60 \\ 66 \\ 72 \end{pmatrix} \cdot in$$

Subject: __Variable Size Pier Shear Capacity (LRFD)__

5.) Tabulate the shear capacity of piers sizes for two failure plane mechanism considering the strength of the concrete and the spiral reinforcing steel:

Concrete Shear Strength
(single shear plane)

Spiral Reinforcment Strength
(single shear plane)

$$V_c = \begin{pmatrix} 44.683 \\ 71.75 \\ 105.176 \\ 144.78 \\ 190.897 \\ 243.373 \\ 302.207 \\ 367.4 \\ 438.643 \end{pmatrix} \cdot kips$$

$$V_s = \begin{pmatrix} 37.723 \\ 47.413 \\ 57.128 \\ 118.711 \\ 136.023 \\ 153.346 \\ 170.674 \\ 188.008 \\ 320.625 \end{pmatrix} \cdot kips$$

$\phi_v := 0.9$ (LRFD 5.5.4.2.1, Interm 2008, pg. 5-25

$V_r := \left(V_c + V_s \right) \cdot \phi_v \cdot 2$ LRFD Eq. 5.8.3.3-1, Interim 2008, pg. 5-70, Shear Resistance 2 Shear planes

Pier Dia. (in.)

Calculated Shear Capacity of Pier (kips)

3600 psi Concrete

$$D = \begin{pmatrix} 24 \\ 30 \\ 36 \\ 42 \\ 48 \\ 54 \\ 60 \\ 66 \\ 72 \end{pmatrix} \cdot in$$

$$V_r = \begin{pmatrix} 148 \\ 214 \\ 292 \\ 474 \\ 588 \\ 714 \\ 851 \\ 1000 \\ 1367 \end{pmatrix} \cdot kips$$

LRFD Section 5

Subject: **Variable Size Pier Shear Capacity (LRFD)**

6.) Plot shear capacity versus pier size for two failure plane mechanism:

Circular Pier Shear Capacity (Kips) vs. Pier Diameter (inches)

LRFD Pier Shear Capacity (kips)

$\dfrac{V_r}{kips}$

$\dfrac{D}{in}$

Pier Diameter (inches)
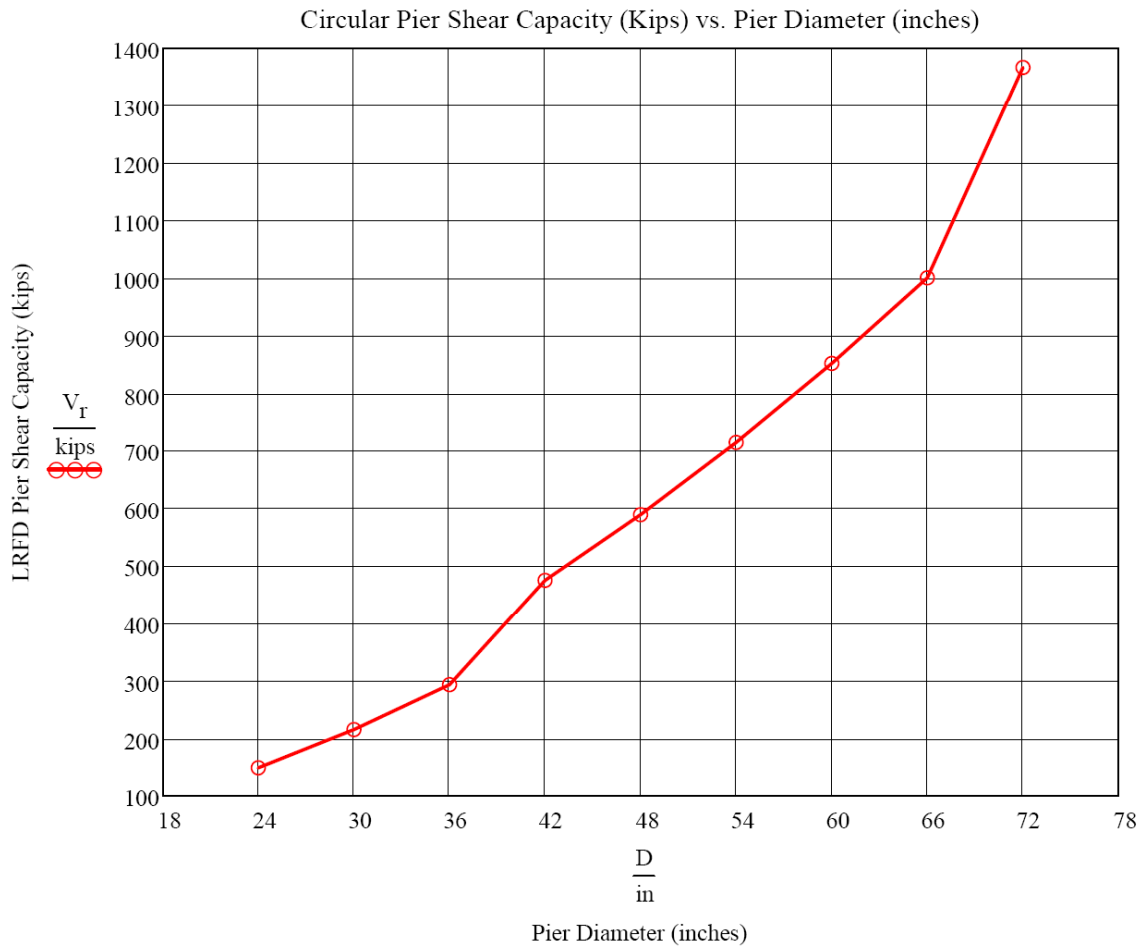
APPENDIX B. MODEL DEVELOPMENT

SINGLE UNIT TRUCK (SUT) MODEL DEVELOPMENT

No public domain SUT model is readily available; an SUT model was developed for the purpose of simulation using the closest related tractor-trailer model developed by National Crash Analysis Center (NCAC) for FHWA. The differences between the tractor model and the actual SUT were determined. Modifications were made to the tractor model to convert to an SUT vehicle.

Figure B1. Modeled 1982 Mack Econodyne Truck Model R688ST.
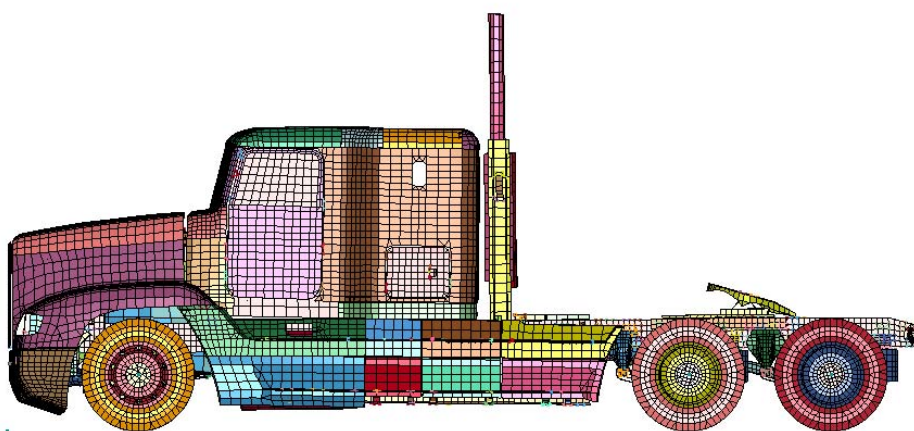
Figure B2. Original Tractor Model NCAC V01b.
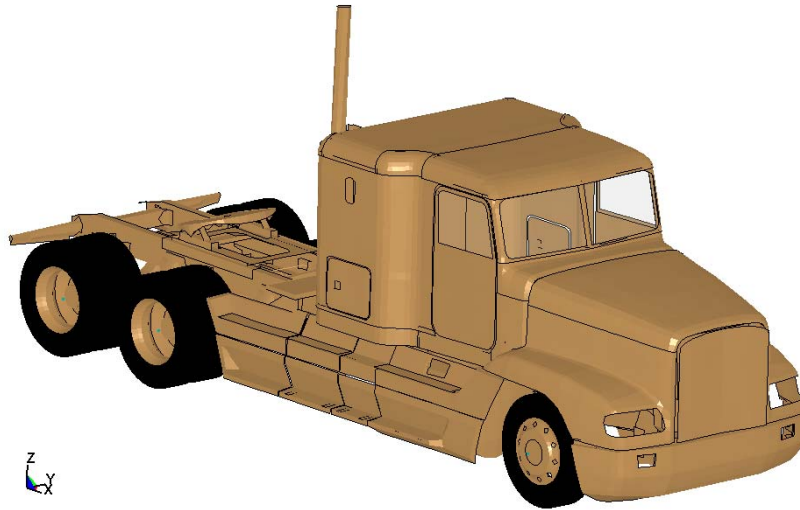
Figure B3. Original Tractor Trailer Model.
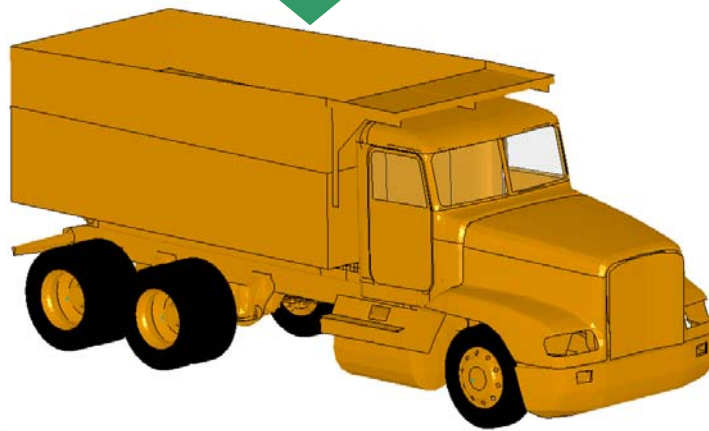
Figure B4. Modified SUT with Rigid Container.
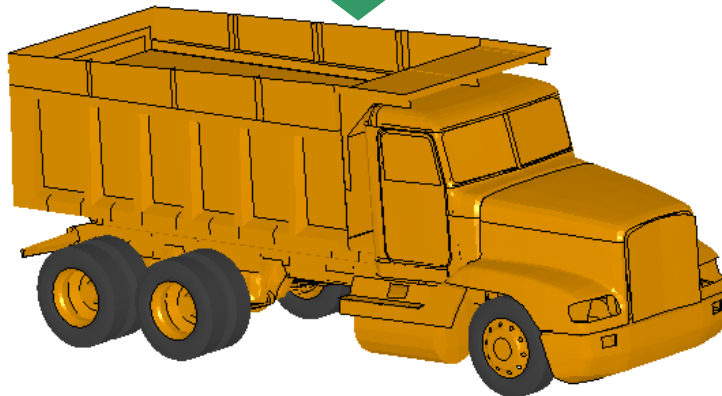
Figure B5. Modified SUT with Deformable Container.

TRACTOR-TRAILER MODEL DEVELOPMENT

NCAC is currently developing a public domain tractor-trailer model for the FHWA. The model has a completed tractor, whereas the trailer is still under development. The original model trailer was comprised of a single rigid component. For the purposes of this study a trailer model was developed from measurements and data taken from an actual trailer as seen in Figure B6.

Figure B6. Modeled Trailer.
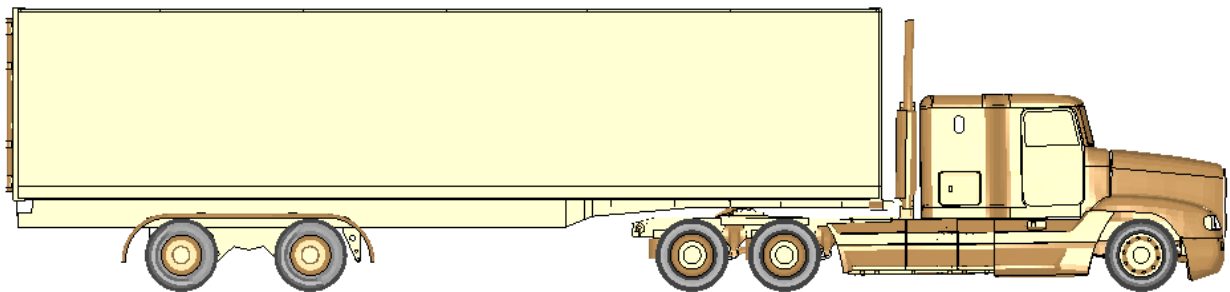
Figure B7. Original Tractor-Trailer Model NCAC V01b.
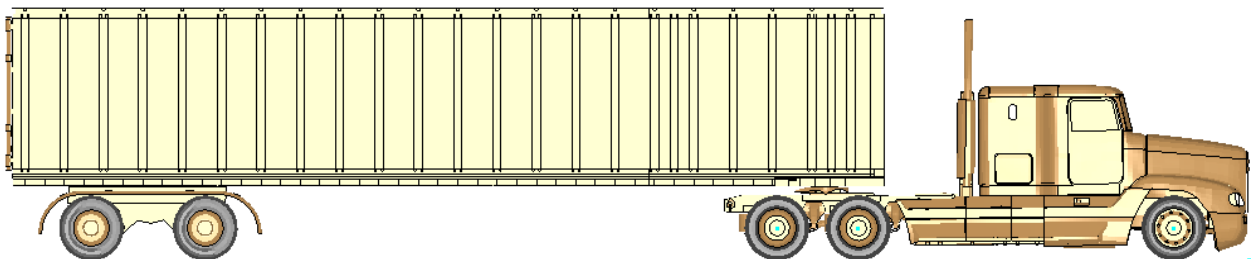
Figure B8. Modified Tractor-Trailer Model.
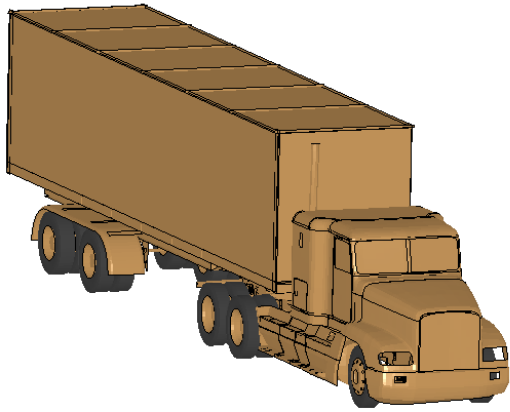
Figure B9. Original Tractor-Trailer.

Figure B10. Modified Tractor-Trailer.
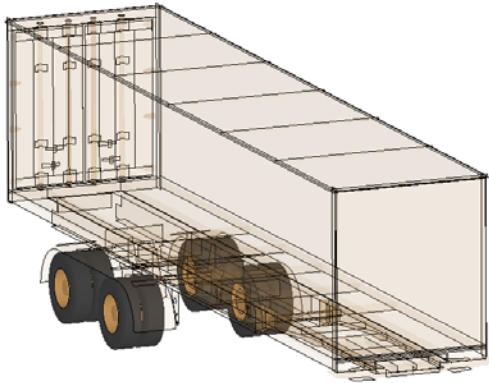
Figure B11. Original Trailer.

Figure B12. Modified Trailer Structure.

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A pin model was developed for the tractor-trailer connection. The model allows for the trailer to articulate upon impact, as well as capture the shearing effects of the pin.
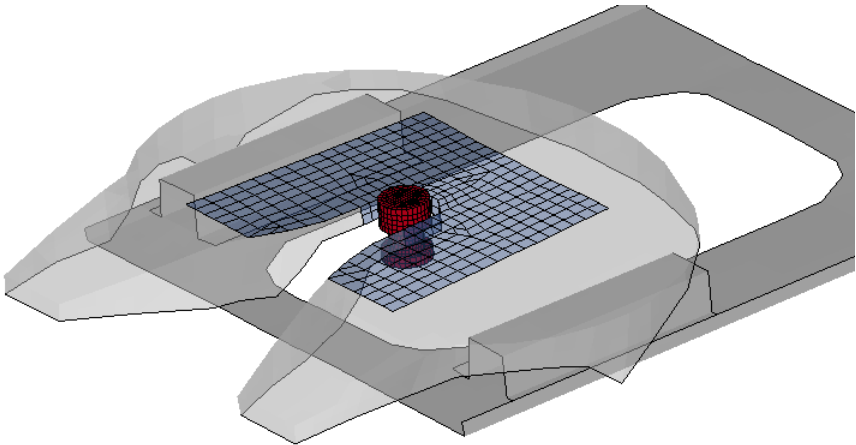
Figure B13. Trailer Pin Connection Model.
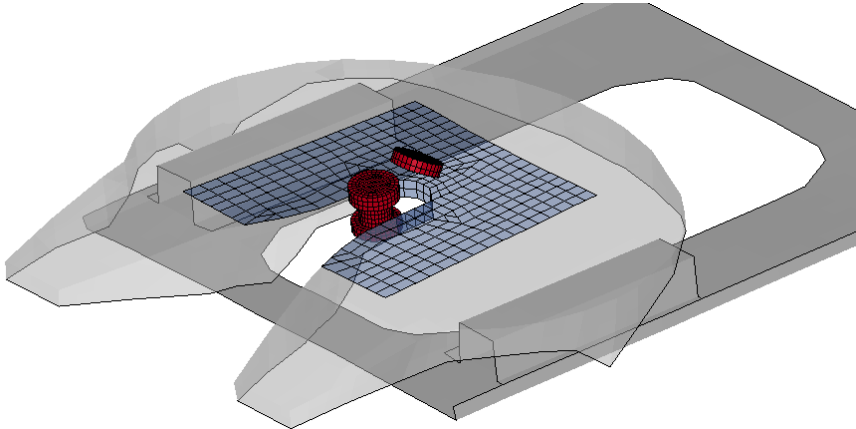
Figure B14. Trailer Pin Connection Shearing Action.

APPENDIX C. TEXAS 6 VEHICLE CLASSIFICATIONS

(http://onlinemanuals.txdot.gov/txdotmanuals/tda/texas_6_classification_figures.htm)

Figure C1. Texas 6 Class 1 — Motorcycles and Passenger Vehicles.
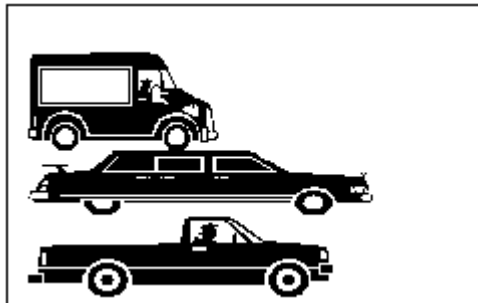
Figure C2. Texas 6 Class 2 — 2 Axles, 4-Tire Single Units.
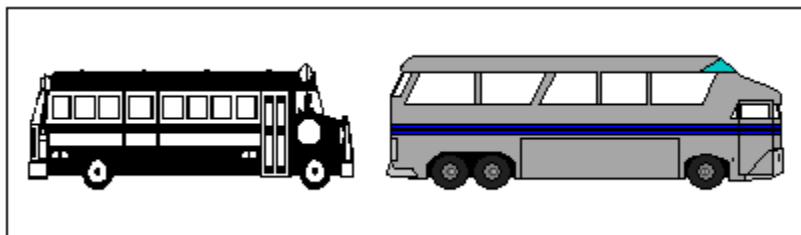
Figure C3. Texas 6 Class 3 — Buses.
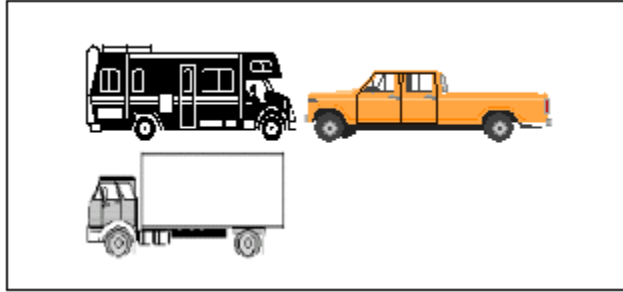
Figure C4. Texas 6 Class 4 — 2D, 6-Tire Single Unit (Includes Handicapped-Equipped and Mini School Buses).
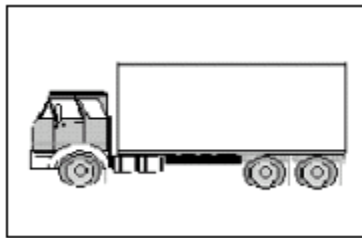
Figure C5. Texas 6 Class 5 — 3 Axles, Single Unit.
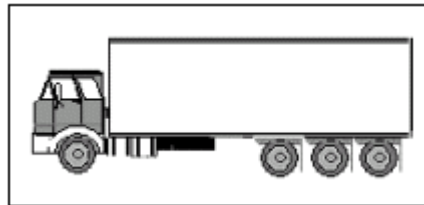
Figure C6. Texas 6 Class 6 — 4 or More Axles, Single Unit.
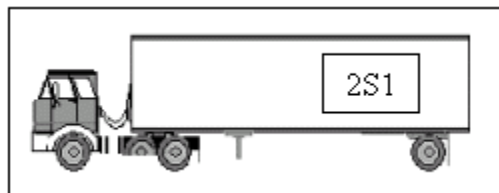
Figure C7. Texas 6 Class 7 — 3 Axles, Single Trailer.
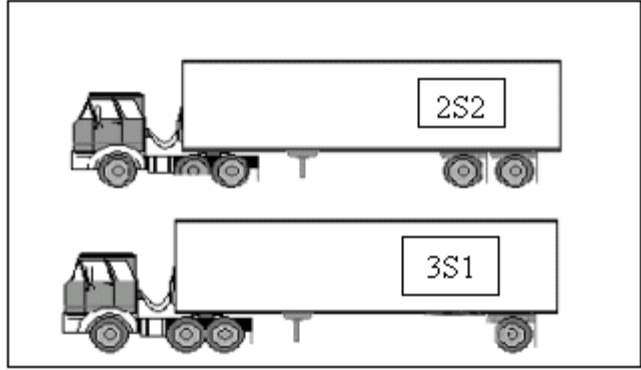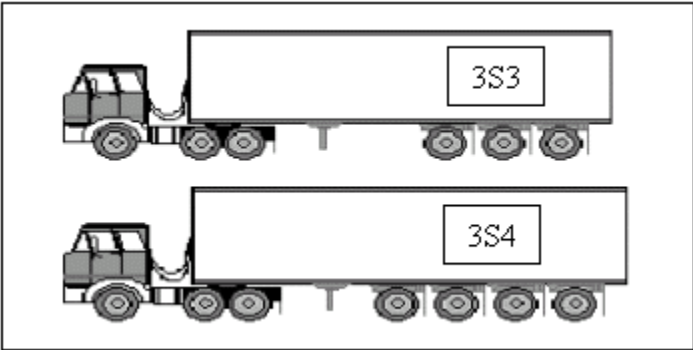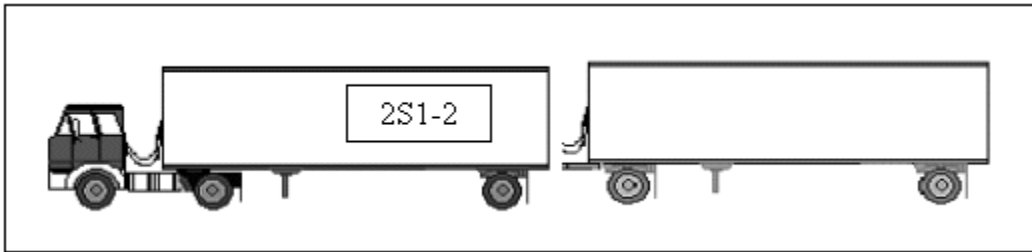
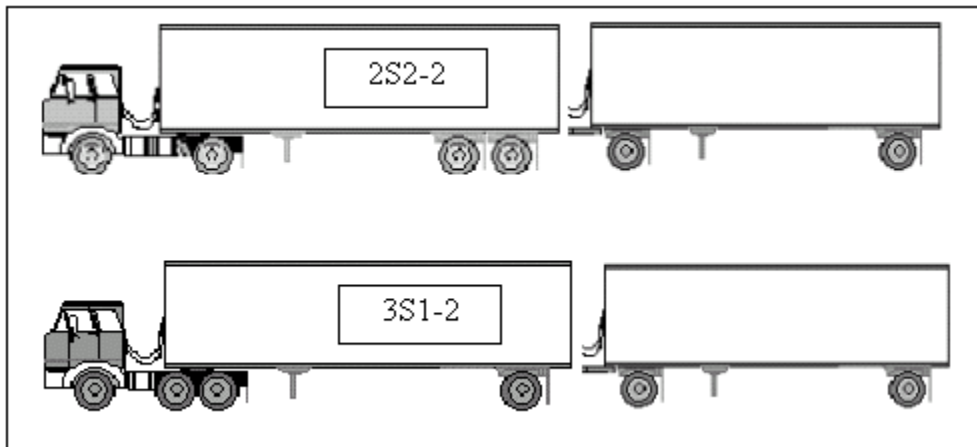**Figure C8. Texas 6 Class 8 — 4 Axles, Single Trailer.**
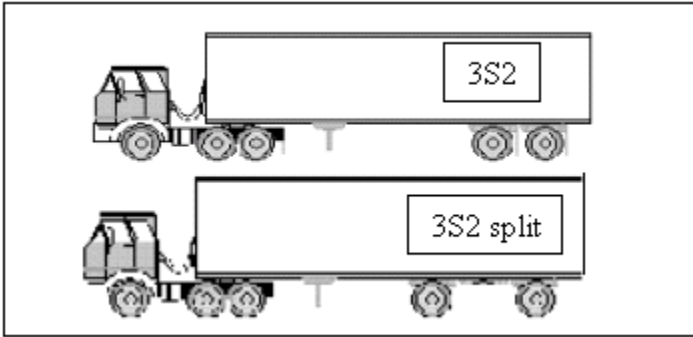
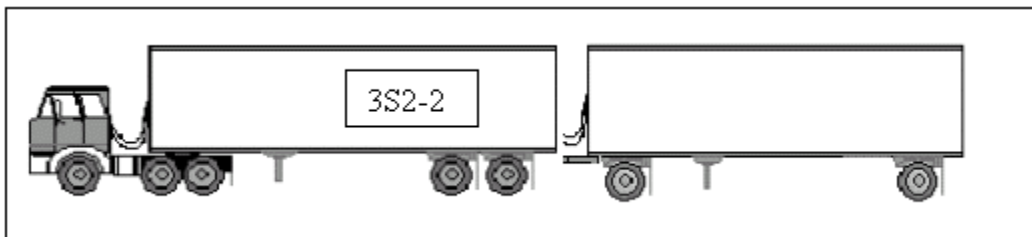**Figure C9. Texas 6 Class 9 — 5 Axles, Single Trailer.**

Figure C10. Texas 6 Class 10 — 6 or More Axles, Single Trailer.

Figure C11. Texas 6 Class 11 — 5 or Less Axles, Multi-Trailers.

Figure C12. Texas 6 Class 12 — 6 Axles, Multi-Trailers.

Figure C13. Texas 6 Class 13 — 7 or More Axles, Multi-Trailers.