

**Experimental and Analytical Study of Dynamic Behavior of Bridge
Superstructures Subjected to Overheight Vehicle Collisions**

A Dissertation Presented for the
Doctor of Philosophy
Degree
The University of Tennessee, Knoxville

Yuan Jing
May 2017

Dedication

I dedicate this dissertation to my parents and my little brother, for their unconditional love and support. Without their encouragement, this dissertation would not have been possible.

Acknowledgement

I would like to express my sincere appreciation to my advisor, Dr. Zhongguo John Ma. He is one of the best mentors who gave me the opportunity to pursue my doctorate degree, advised me on my study and research, and encouraged me when I went through difficult times.

I would like to express my appreciation to Dr. Edwin G. Burdette, Dr. Richard M. Bennett, and Dr. Stephanie TerMaath for serving on my committee and reading my dissertation. Thanks for their great effort in teaching my graduate level classes. Thank Dr. Bennett for offering help with the modeling of the PC girder in Chapter 4 of this dissertation.

Special thanks go to Mr. Cody A. Mitchell who assisted with designing and building the impact testing facility and Mr. Larry Roberts for offering so much assistance than I ever thought. Thank them for making the lateral impact testing possible.

I would also like to thank my colleagues for their help and friendship, Dr. Wenchao Song, Dr. Xin Jiang, Dr. John Cabage, Mr. Qiang Gui, Dr. Yulin Zhan, Dr. Lin Xiao, Dr. Changwang Yan.

Finally, I want to give my thanks to the following that have provided financial support and help to make this research possible: National University Rail (NURail) Center, Mr. John Hillman from HC Bridge Company, LLC, Britton Bridge, LLC, Southern Shore Development, LLC.

Abstract

The increasing occurrence of over-height vehicle collisions with bridges in the United States leads to concern about the damage due to lateral impact to bridge superstructures by over-height vehicles. However, this issue is not fully addressed in current bridge specifications. Previous researchers have conducted a number of small-scale tests to study the impact process. Also, finite element method (FEM) has largely been used to analyze the complicated collision mechanism.

A full-scale lateral impact testing facility was designed and built on a construction site in Knoxville, Tennessee, United States. An AASHTO Type-I prestressed concrete (PC) girder and a Hybrid Composite Beam (HCB) bridge were tested using this facility, which led to a realistic level of damage and mechanism analysis of bridge superstructures under lateral impact loading as described in this dissertation. The failure of the PC girder was first introduced by punching shear around the impact zone. With the penetration of the impactor, the damaged impact zone behaved as a “hinge” which moved upward due to the heavy weight of both overhangs. HCB bridge experienced no global failure but only local damages of the FRP shell around the impact zone. Impact energy was mostly absorbed through strain energy of the tension reinforcement and the low-density foam and dissipated through local damage of the FRP shell.

Commercial software ABAQUS/Explicit was used to develop FE model of the PC girder and the FE results were compared with the experimental results. Parametric study was also performed to evaluate the behavior of the PC girder under different impact conditions.

Keywords: Bridge, over-height vehicles, full-scale, lateral impact, prestressed concrete, hybrid composite beam, ABAQUS/Explicit, parametric study.

Table of Contents

1. INTRODUCTION	1
1.1 Statement of the Problem	1
1.2 Literature Review on Impact	6
1.2.1 Definition	6
1.2.2 Current Code Provisions	7
1.2.3 Research on Beams under Impact Loading.....	10
1.3 Over-Height Vehicle Collisions Literature Review.....	20
2. DESIGN OF FULL-SCALE LATERAL IMPACT TESTING FACILITY AND DATA	
ACQUISITION SYSTEM.....	27
2.1 Impact Testing Facility Options	27
2.2 Design Requirements	28
2.3 Construction of Impact Testing Facility	29
2.3.1 Impact Cart	29
2.3.2 Track System	30
2.3.3 Backstop and Support.....	30
2.4 Data Acquisition System.....	33
3. FULL-SCALE LATERAL IMPACT TESTS OF PC GIRDER AND HYBRID COMPOSITE	
BEAM BRIDGE	39
3.1 Impact Test of PC Girder	39
3.1.1 Specimen	39
3.1.2 Instrumentation.....	39
3.1.3 Dynamic Behavior of Prestressed Concrete Girder.....	41
3.2 Impact Test of HCB Bridge	46
3.2.1 Introduction	46
3.2.2 Specimen	48
3.2.3 Instrumentation.....	53
3.2.4 Lateral Impact Testing.....	53
3.2.5 Damage of HCB Bridge during Collision	56
3.2.6 Testing Results and Discussion.....	59
3.3 Comparison Between PC Girder and HCB Bridge under Lateral Impact Loading	73
4. FINITE ELEMENT SIMULATION OF PC GIRDER UNDER IMPACT LOADING.....	75
4.1 FE Simulation of PC Girder Impact Process	75
4.1.1 Concrete Damaged Plasticity	75
4.1.2 Material Constitutive Relation for Concrete and Prestressing Strand	80
4.1.3 Strain Rate Effect	82
4.1.4 FE Simulation of AASHTO Type-I PC Girder Impact Process.....	86
4.1.5 FE Results and Data Calibration	87

4.1.6	Parametric Study	91
4.1.7	Results Discussion.....	101
5.	CONCLUSIONS	104
5.1	Conclusions.....	104
5.2	Future Work.....	106
REFERENCES		107
VITA		116

List of Tables

Table 3.1 Beam Properties	40
Table 3.2 HCB Material Properties	52
Table 4.1 Elastic Material Properties	83

List of Figures

Figure 1.1 Perspective of Hybrid Composite Beam (Hillman, 2012).....	3
Figure 1.2 Dynamic Force (EN 1991-1-7, 2006).....	6
Figure 1.3 Analytical Model in High Speed Loading Analysis (Ishikawa et al., 2000).....	15
Figure 1.4 Impact Testing Setup (Fujukaka et al., 2009)	18
Figure 1.5 Two-Degree-of-Freedom Mass-Spring-Damper System (Fujukaka et al., 2009).....	18
Figure 1.6 Two Types of Damage (Shi et al., 2014).....	20
Figure 1.7 Three-Dimensional View of Experimental Layout (Xu et al., 2012)	24
Figure 1.8 FE Models of Container Truck and PC T-Girder Bridge (Xu et al., 2013)	25
Figure 1.9 Simplified Model (Xu et al., 2013)	26
Figure 2.1 Truck Before and After Impact (Zaouk et al., 1996).....	28
Figure 2.2 Impact Cart	30
Figure 2.3 Track System and Excavator	31
Figure 2.4 Backstop System	31
Figure 2.5 Support System.....	32
Figure 2.6 Full-Scale Lateral Impact Testing Facility.....	33
Figure 2.7 NI-SCXI-1001 DAQ Measurement Device	34
Figure 2.8 Example Code of Strain Measurement (National Instruments, 2016).....	35
Figure 2.9 Virtual Channels of Strain Gages	35
Figure 2.10 Front Panel	36
Figure 2.11 Sample Clock	37
Figure 2.12 Data Split and Merge	37
Figure 2.13 Data Monitoring Plots	38
Figure 3.1 Prestressed Concrete Girder Cross Section View (cm); 1 cm=0.39 inch	40
Figure 3.2 Prestressed Concrete Girder Setup	41
Figure 3.3 Positions of Sensors (cm); 1 cm=0.39 inch	42
Figure 3.4 Collision Process from Side View	43
Figure 3.5 Collision Process from Rear View	44
Figure 3.6 Cross Section View of HCB Bridge (mm); 1 mm=0.039 inch	49
Figure 3.7 HCB Bridge Setup.....	49
Figure 3.8 Cutting of Hardwire® Tape (Hillman, 2005).....	51
Figure 3.9 Hardwire® from Bottom View	51
Figure 3.10 Small Window Cutout of FRP Shell.....	53
Figure 3.11 Sensor Positions on HCB Bridge and Impact Cart	54
Figure 3.12 Impact Cart and Second Excavator	55
Figure 3.13 Damage at Front Face of the Beam	57
Figure 3.14 Damage at Bottom Face of the Beam.....	57
Figure 3.15 Distance between Specimen and Steel Tube after Impact	58
Figure 3.16 Filtered Acceleration Time History; 1 g =9.81 m/s ² =386.4 in./s ²	60
Figure 3.17 Displacement Time History; 1 cm =0.39 inch	61

Figure 3.18 Maximum Horizontal Displacement Distribution along Span; 1 cm =0.39 inch	63
Figure 3.19 Backstop Strain Time History.....	64
Figure 3.20 Lateral Boundary Condition of HCB Bridge.....	64
Figure 3.21 Reaction Force Time History; 1 kN =0.225 kip	65
Figure 3.22 HCB Bridge Strain Time History	66
Figure 3.23 Load Displacement Curve; 1 kN =0.225 kip, 1 cm =0.39 inch	68
Figure 3.24 Calibration Stick and Coordinate System.....	69
Figure 3.25 Footprints of Point A.....	70
Figure 3.26 Displacement of Point A; 1 cm=0.39 inch.....	71
Figure 3.27 Displacements of the Impact Cart and the HCB Bridge; 1 cm=0.39 inch.....	71
Figure 3.28 Indentation of the HCB Bridge during Impact; 1 cm=0.39 inch	72
Figure 3.29 Internal Configuration of HCB.....	74
Figure 4.1 Typical Yield Surfaces on the Deviatoric Plane (Abaqus, 6.14)	77
Figure 4.2 Concrete Uniaxial Response in Compression (Abaqus, 6.14)	78
Figure 4.3 Concrete Uniaxial Response in Tension (Abaqus, 6.14)	78
Figure 4.4 Stress-Strain Curve of High Strength Concrete in Compression; 1 MPa=145 psi.....	82
Figure 4.5 Stress-Strain Curve of High Strength Concrete in Tension; 1 MPa=145 psi	83
Figure 4.6 Stress-Strain Curve for Prestressing Strand; 1 MPa=145 psi.....	84
Figure 4.7 Finite Element Model of Prestressed Concrete Girder	87
Figure 4.8 Finite Element Model of Impact Cart	88
Figure 4.9 Horizontal Displacement at Midspan from FE Analysis; 1 cm=0.39 inch	89
Figure 4.10 Horizontal Displacement Contour at Midspan	89
Figure 4.11 Vertical Displacement from SP1 vs. FE Results; 1 cm=0.39 inch.....	90
Figure 4.12 Damage Contour of the PC Girder at t=0.1 s	91
Figure 4.13 PC Girder at t=0.1 s from Side View.....	92
Figure 4.14 Damage Contour at the Bottom Face	92
Figure 4.15 Horizontal Displacement under Different Velocities; 1 cm=0.39 inch	93
Figure 4.16 Impact Force under Different Velocities; 1 kN=0.225 kip	94
Figure 4.17 Damage Contours under Different Velocities at t=0.1 s	95
Figure 4.18 Horizontal Displacement under Different Compressive Strengths; 1 cm=0.39 inch.....	98
Figure 4.19 Impact Force under Different Compressive Strengths; 1 kN=0.225 kip.....	99
Figure 4.20 Damage Contours under Different Compressive Strengths at t=0.1 s	100
Figure 4.21 Horizontal Displacement under Different Impact Areas; 1 cm=0.39 inch	101
Figure 4.22 Damage Contours under Different Impact Areas at t=0.1 s.....	102

1. INTRODUCTION

1.1 Statement of the Problem

The number of automobiles in use in the United States has dramatically increased year by year, leading to their growing interaction with bridges. Injuries and fatalities frequently occur in vehicle crashes with bridges, and the collisions can cause damage to the bridges as well. Based on the Federal Highway Administration (FHWA), more than 600,000 bridges are registered in the National Bridge Inventory (NBI), and the third leading cause of bridge failure is collision damage when a bridge is hit by a vehicle or a vessel. The vehicles with height exceeding the vertical clearance permit will impact the bridge superstructures and lead to the damage of the beam or the collapse of the whole bridge. In the meantime, highway and railroad bridge collisions with over-height vehicles have been a common occurrence in recent years. For example, a project conducted by the Texas Department of Transportation reported an increased number of prestressed concrete bridge incidents over the five-year survey period of 1987 to 1992 due to overheight vehicles and loads (Feldman et al. 1998). Three levels of damage were classified in their study: 61% of impact damaged girders were reported having minor damage, which was defined as independent concrete cracks or small spalls; girders with large enough concrete cracks and spalls that can expose undamaged prestressing strands were assessed as moderately damaged, and 25% girders suffered moderate damage; 14% girders had severe damage, which includes damaged strands, loss of large concrete cross section, and lateral girder distortion. A

survey was carried out by the New York State Department of Transportation (NYSDOT) to collect information on difference aspects of problems caused by bridge hits across the country (Agrawal et al., 2011). Forty-four state DOTs and two local authorities responded, and the NYSDOT bridge hits database indicated that a majority of bridge hits were caused by overheight vehicles. The frequency of overheight accidents with highway bridges reported in Maryland increased by 81% between 1995 and 2000 (Fu et al., 2004). A survey that was sent to collect national statistics showed that of the 29 states responding, 62% indicated that they consider overheight collisions to be a significant problem according to Fu et al. (2004). In their study, the percentage of vehicles involved to total overheight vehicle accidents was also studied: 36% for enclosed box trailers, 31% for flatbed trailers with oversized loads, 16% for dump trucks, and 17% for others. Based on the investigation of vehicular collisions to railway bridges in the United Kingdom (Agrawal et al., 2011), the drivers not aware of the height of their vehicles, and the unclear signing of the clearance limit were the main reasons that led to vehicular collisions at bridges. Although measures such as driver education, detection systems, and improvement in signing have been studied by other researchers. The bridge itself is studied here to evaluate whether it has enough capacity to resist the impact to some extent in order to reduce the time and the inconvenience of bridge closure for repair or replacement. The study of the bridge overheight vehicle collision process is of significant importance to understand the impact mechanism, which can provide data to evaluate existing bridges and offer guidance to the

design of new bridges, especially newly developed bridge systems.

For example, a new bridge technology, Hybrid Composite Beam (HCB), originally conceived by John Hillman in 1996 (Hillman, 2012), was developed as a sustainable solution for the construction of new and replacement bridges in rail infrastructure. The hybrid composite beam combines advanced composite materials with conventional concrete and steel to create a bridge that is stronger and more resistant to corrosion than conventional bridges. In general, the HCB, as shown in **Fig. 1.1**, is composed of three main sub-components.

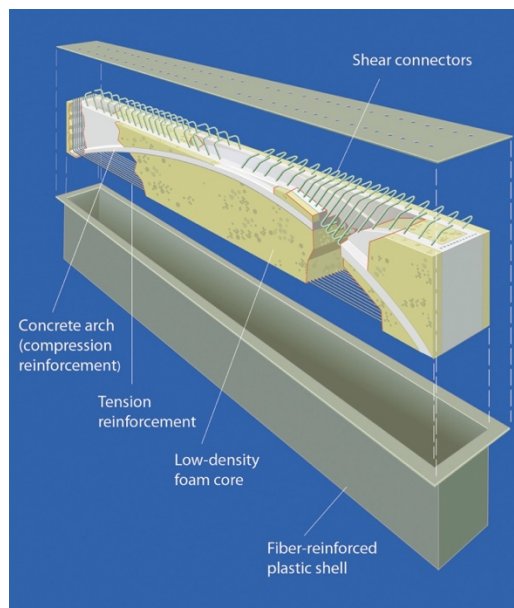


Figure 1.1 Perspective of Hybrid Composite Beam (Hillman, 2012)

Resistance of compression forces can be achieved by using a concrete arch. Tension

reinforcement anchored at the ends of concrete arch could consist of carbon, glass, steel fibers or prestressing strands. A fiber reinforced polymer (FRP) shell is comprised of vinyl ester resin reinforced by glass fibers and encapsulates both the concrete arch and the tension reinforcement. The tension reinforcement is fabricated at the same time the FRP shell is constructed, and subsequently infused in the same resin as the glass fibers.

FRP has been used in construction industry and transportation infrastructure applications due to its light weight, fast and simplified construction process, as well as corrosion and fatigue resistance. However, the high initial cost prevents FRP from being cost competitive with conventional concrete and steel materials. FRP is strong in tension when the applied load is parallel to the orientation of its fibers, but less strong when the load is perpendicular to the fibers' orientation. Other limitations result from the low shear strength capacity, low compression capacity combined with local buckling phenomena, and the behavior being almost linear up to failure with little ductility. Bridges with pure FRP material are limited to short span lengths. HCB can overcome the shortcomings of the composite materials. The use of conventional materials, combined with FRP, distinguish HCB from earlier FRP structures. The combination of conventional materials with FRP takes advantage of the inherent benefits of each material and optimizes the overall performance of the structure. However, an issue might result in HCB bridge damages when an HCB bridge is subjected to lateral impact from over-height vehicles passing under

the bridge, as mentioned in Hillman's (2012) report. This fact has raised concern, especially when a growing number of bridge collisions caused by over-height vehicles occurred in the United States (Fu et al., 2004) and the lateral impact from vehicles to bridge superstructures is not addressed in the AASHTO bridge design specifications. Though the behavior of this novel system has been studied during the last few years through limited tests, impact testing has not yet been conducted.

The main objective of this dissertation study was to evaluate the performance of an PC girder and an HCB bridge when subjected to lateral impact loading, which will help provide data to satisfy concerns regarding safety of these beams. The following tasks were performed in this research:

1. Designed and constructed a full-scale lateral impact testing facility with another student's assistance;
2. Developed a data acquisition system to collect dynamic data through available sensors;
3. Conducted lateral impact testing of an AASHTO Type-I prestressed concrete girder;
4. Conducted lateral impact testing of an HCB bridge;
5. Developed a finite element model of the prestressed concrete girder and simulated the lateral impact process, calibrated the FE results with testing results, and performed parametric study.

1.2 Literature Review on Impact

1.2.1 Definition

“In mechanics, an impact is defined as a high force or shock applied over a short time period when two or more bodies collide.”

Eurocode EN 1991-1-7 (2006) Section 1.5.5 defines the dynamic force as a force that varies in time as shown in **Fig. 1.2**. The dynamic force is also associated with a contact area under the circumstance of impact.

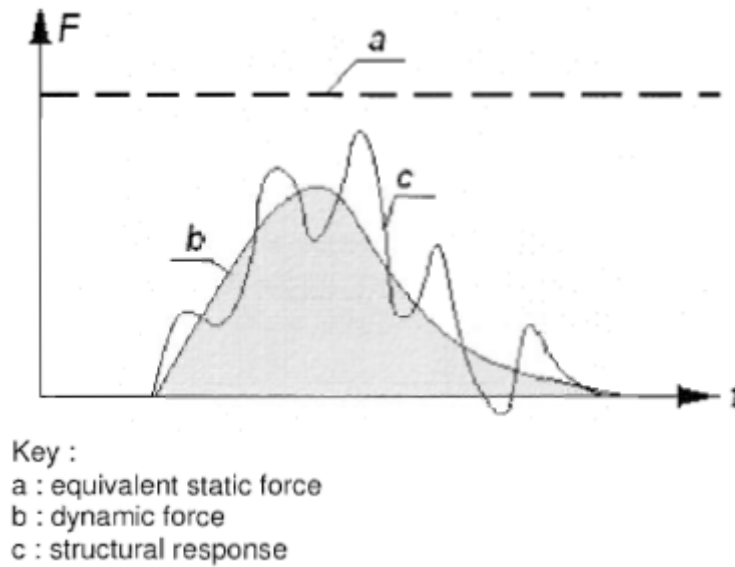


Figure 1.2 Dynamic Force (EN 1991-1-7, 2006)

1.2.2 Current Code Provisions

The present provisions for vehicle impact are contained in Section 3.4.1 and 3.6.5 of American Association of State Highway and Transportation Officials (AASHTO) Load and Resistance Factor Design (LRFD) Bridge Design Specifications (2016) and Section 4.3.2 in Eurocode EN 1991-1-7 (2006).

1.2.2.1 AASHTO LRFD Bridge Design Specifications

AASHTO LRFD Bridge Design Specifications (2016) Section 3.4.1 specify that Extreme Event II includes vehicular collision force (CT). The provision in AASHTO reads as follows:

“Extreme Event II-Load combination relating to ice load, collision by vessels and vehicles, check floods, and certain hydraulic events with a reduced live load other than that which is part of the vehicular collision load, CT. The cases of check floods shall not be combined with BL, CV, CT, or IC”.

In Section 3.6.5, an equivalent static force of 2670 kN (600 kips) is suggested for pier or abutment impacted by vehicles, which is assumed to act in a direction of zero to 15 degrees with the edge of the pavement in a horizontal plane, at a distance of 1.52 m (5.0 ft) above ground. The equivalent static force of 2670 kN (600 kips) is based on the information from full-scale crash tests (Buth et al., 2010) of rigid columns impacted by 356 kN (80-kip) tractor trailers at 80 km/h (50 mph). AASHTO adopted the recommended

equivalent static force as a rigid pier collision force, but the results are only applicable to the type of truck and cargo used in the tests and the effect of vehicle speed was not considered in Buth et al. (2010) study.

AASHTO (2016) also specifies the ship collision force on superstructure in Section 3.14.10.

However, there is no provision for vehicular collision of bridge superstructure.

1.2.2.2 Eurocode EN 1991-1-7

Eurocode 1 - Actions on structures - Part 1-7: General actions – Accidental actions specifies the vehicle collision force on bridge superstructures.

The provisions in Section 4.3.2 are as follows:

- (1) “Design values for actions due to impact from lorries and/or loads carried by the lorries on members of the superstructure should be defined unless adequate clearances or suitable protection measures to avoid impact are provided.
- (2) Where appropriate, forces perpendicular to the direction of normal travel, F_{dy} , should also be taken into account.
- (3) The applicable area of the impact force “F” on the members of the superstructure should be specified. The recommended area of impact is a square with the sides of 0.25 m (10 in.) length.”

Table 4.2 of EN 1991-1-7 lists the recommended equivalent static design forces for the vehicular impact on bridge superstructures. For motorways and country national and main roads, the lateral equivalent static design force is suggested as 500 kN (112 kips). However, the suggested forces cannot be directly adopted without considering the volume and type of traffic in different regions or countries.

Annex C of EN 1991-1-7 presents the dynamic design for impact. Impact is defined as either hard impact, where the energy is mostly dissipated by the impacting object, or soft impact, where the structure is designed to absorb the energy through deforming.

The maximum dynamic force for hard impact with the impacting object deforming linearly during the impact process is given by Eq. (1.1)

$$F = v_r \sqrt{km} \quad (1.1)$$

Where:

v_r is the impacting object velocity at impact;

k is the equivalent elastic stiffness of the impacting object;

m is the mass of the impacting object.

If the structure is assumed elastic and the impacting object rigid, Eq. (1.1) still applies and should be used with k being the stiffness of the structure.

1.2.3 Research on Beams under Impact Loading

Previous research has studied the performance of structures due to impact and impulsive load by experiments, analysis, and numerical simulations.

Simms (1945) studied the impact resistance of reinforced concrete (RC) members failing in bending by doing mass-falling impact tests. Reinforced concrete beams and slabs were tested in his study. The dimension of the beams was 10.2 cm wide x 20.3 cm deep x 313 cm long (4 in. x 8 in. x 7 ft). In the beams, stirrups were provided to avoid shear failure. Different amounts of mild steel bars were used in the reinforced concrete beams, and high-strength steel was also employed in a few beams. The reinforced concrete slabs were 50.8 cm wide x 15.2 cm deep x 183 cm long (20 in. x 6 in. x 6 ft) with different types of steels.

The reinforced concrete members were simply supported and the load was transmitted to the concrete through a steel bearing plate to prevent local failure. The test showed that some concrete members had the same form of damage under impact as under static loading. Therefore, the estimation of the damage of reinforced concrete members under impact can be determined from Eq. (1.2).

$$\alpha WH = E \quad (1.2)$$

where: α is a reduction factor allowing for the member's inertia;

W is the weight of the falling mass;

H is the height of drop;

E is the energy of deformation of the member.

The energy of deformation E is equal to the area of load-deflection curve of the members under static loading A_s if energy absorbed under static loading is the same as that under impact. The proposed reduction factor calculation in his study is a function of the ratio of the weight of the member and the weight of the falling mass as:

$$\alpha = \frac{1}{1 + \frac{4w}{5W}} \quad (1.3)$$

where: w is the weight of the member;

W is the weight of the falling mass.

Since the weight of falling mass was equal to that of the member (90.7 kg/200 lb.), α was equal to 0.56 in this study. Then $\alpha WH = A_s$, so that the deflection and damage under a known impact can be determined from a knowledge of the complete static load deflection curve. Estimated and actual degrees of damage under impacts were compared and damage is represented by the ratio of deflection to span. The results showed that they were in good agreement.

Bate (1961) carried out a large number of tests as an extension of the investigation by

Simms (1945). The test specimens consisted of prestressed concrete (PC) beams and reinforced concrete beams. The results were not only evaluated for beams having same failure modes under impact and under static loading but also for beams having different modes of failure. Bate (1961) concluded that for both prestressed and reinforced concrete beams, the energy of deformation from static tests generally gave an approximation to the impact resistance for a single impact, provided that failures under static and impact loading were similar. But the effects of potential slip of pretensioned wires and shear failures played an important role in the performance of prestressed concrete beams under impact, and these effects might lead to the differences between failure modes under static and under impact loading. For example, a beam which fails in flexure under static loading could actually fail in shear or by strand slip under impact loading. The slip of strands most likely occurring when the supports are close to the beam ends and the strength of concrete is relatively low. Shear failure may happen when the concrete strength is high and the steel is so placed that fracture is induced along a horizontal plane in the beam. The testing results also showed that the largest differences between measured and calculated α appeared when the failure mode was shear under impact loading.

Mindess et al. (1986) measured the velocity with which cracks propagated in cementitious materials under high strain rates. Their study found that crack velocities in hardened

cement paste, fiber reinforced concrete, and reinforced concrete varied from 75 m/s (168 mph) to 115 m/s (257 mph) under impact loading. The reinforcement of either fibers or steel bars, reduced the crack velocity compared to that of hardened cement paste.

Banthia et al. (1987) studied the impact behavior of normal strength, high strength, and fiber reinforced concrete beams. Simply supported specimens of 100 mm (3.94 in.) x 125 mm (4.92 in.) x 1400 mm (55.1 in.) were impacted at midspan by a 345 kg (761 lb.) mass impact hammer. Variation in the stress rate was achieved by varying the drop height of the hammer. An equation of generalized inertial load $P_i(t)$ was proposed in this paper as follows,

$$P_i(t) = \rho A \ddot{u}_o(t) [l/3 + (8/3)/(h^3/l^2)] \quad (1.4)$$

where

$P_i(t)$: Generalized inertial load;

ρ : Mass density of concrete;

A : Area of cross section of the beam;

$\ddot{u}_o(t)$: Acceleration at the center;

l : Span of the test beam;

h : Length of the overhang.

Once the generalized inertial load $P_i(t)$ is known, the actual bending load equals the

recorded impact load minus the generalized inertial load. Data from accelerometers were used to predict the inertial load. According to the study, care must be taken in the use of high strength concrete, with $f'_c = 82$ MPa (11900 psi), in dynamic situations due to its brittle property though it has a higher impact strength than normal strength concrete. Fiber reinforced concrete was better than plain concrete because of its ductility and higher impact resistance shown in this study.

Erki (1999) presented the test results of four concrete beams under impact loading, two of which were externally strengthened for flexure with carbon fiber reinforced polymer (CFRP) laminates and two strengthened with steel plates. Each beam was simply supported and one end of the beam was raised to and then dropped from a specified height. An impact loading was introduced in the beam when the end of the beam got contact with the support. The impact testing results indicated that beams strengthened with CFRP laminates performed well but couldn't provide the same energy absorption capacity as beams with steel plates. It was suggested that additional epoxy bonding of CFRP laminates to the beam would improve their impact resistance. An equation of motion was also proposed to compare the midspan deflections of CFRP strengthened beams and steel plate strengthened beams from the experimental results.

Ishikawa et al. (2000) presented a dynamic analysis method based on the beam elements

to evaluate the dynamic behavior of prestressed concrete beams under impact and high speed loadings. The analytical model is shown in **Fig. 1.3**.

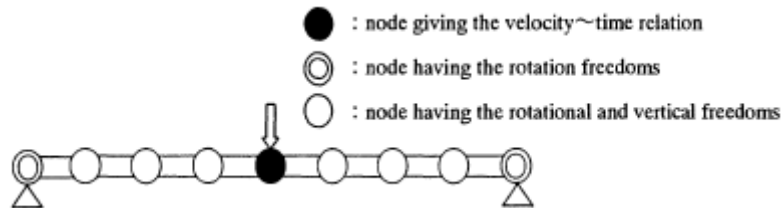


Figure 1.3 Analytical Model in High Speed Loading Analysis (Ishikawa et al., 2000)

During the analysis, the moment-curvature curve of the beam was obtained from compressive and tensile resultant forces by adopting stress strain curve for the concrete and the prestressed tendon. The stiffness matrix of beam element was developed and dynamic load displacement relations of PC beams were found by using Newmark β method and the unknown external force was determined from the equation of motion. Finally, the breaking of prestressed tendon or crushing of concrete terminates the computation. The dropping limit height was estimated from the analysis and provides guidance for impact testing.

After the analysis, a one-blow impact test was performed to confirm the failure behavior of the bonded and unbonded prestressed beams. A shock absorber was used and a steel plate was placed in order to prevent local failure. The study found that dynamic energy

absorption capacity of an unbonded PC beam was 2.4 times bigger than the bonded PC beam. The failure drop height of an unbonded PC beam was about 1.7 times larger than that of a bonded beam during the impact test, and the unbonded PC beam had a larger ductility.

Ando et al. (2000) performed weight falling impact tests on shear-failure type reinforced concrete (RC) beams. 27 RC beams without stirrups were simply supported with the same cross section of 150 mm (5.91 in.) x 250 mm (9.84 in.). Rebar and shear-span were taken as variables. This study concluded that when the static shear bending capacity ratio was less than 1.0 and when the impact velocity was relatively high, RC beams failed resulting from diagonal cracks from loading point to supports, which is shear failure type. Reaction force was also observed to linearly increase up to the maximum value and then gradually decrease. The maximum value of reaction force was similar to static shear capacity.

Rokugo et al. (2001) presented the results of repeated drop weight tests with increasing drop height on RC beams, PC beams, RC beams with short steel fibers, and PC beams with short steel fibers. When the residual displacement exceeded 20 mm (0.79 in.), the impact was terminated. A buffer layer, micro-fiber-reinforced mortar, was developed for improving the impact resistance. The mortar had strain hardening behavior and polyethylene fiber was used, with fiber content of 1.5% by volume. The results showed

that the addition of steel fibers reduced the damage of concrete members by slowing down the propagation of cracks. The buffer layer reduced the concrete spalling. The prestressing improved the restoration of deflection of the members under impact load while too much prestressing raised the local damage of concrete.

Fujukaka et al. (2009) examined the impact response of reinforced concrete (RC) beams through an experimental study and an analytical model that was used to predict maximum deflection and maximum impact force. Twelve specimens of RC beams were impacted by dropping a weight. The testing setup is shown in **Fig. 1.4**. Sufficient stirrups were provided for all the specimens in order to allow for flexure failure. The study found that RC beams with relatively smaller number of longitudinal steel bars exhibited only overall flexural failure while the RC beam with higher amount of longitudinal steel reinforcement experienced not only global flexural failure but also local failure around the impact point. However, the local failure was significantly reduced when large amount of longitudinal compression reinforcement was provided. A two-degree-of-freedom mass-spring-damper system was used to model the drop weight test, of which both overall response and local response at the contact area can be captured, as illustrated in **Fig. 1.5**.

Deng and Tuan (2013) proposed an energy design procedure of concrete-filled steel tube (CFT) beams under lateral impact loading. The finite element code LS-DYNA was used to

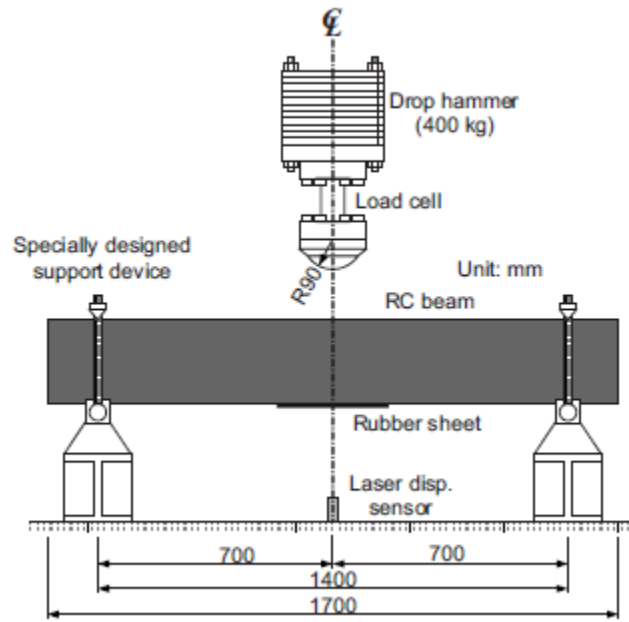


Figure 1.4 Impact Testing Setup (Fujukaka et al., 2009)

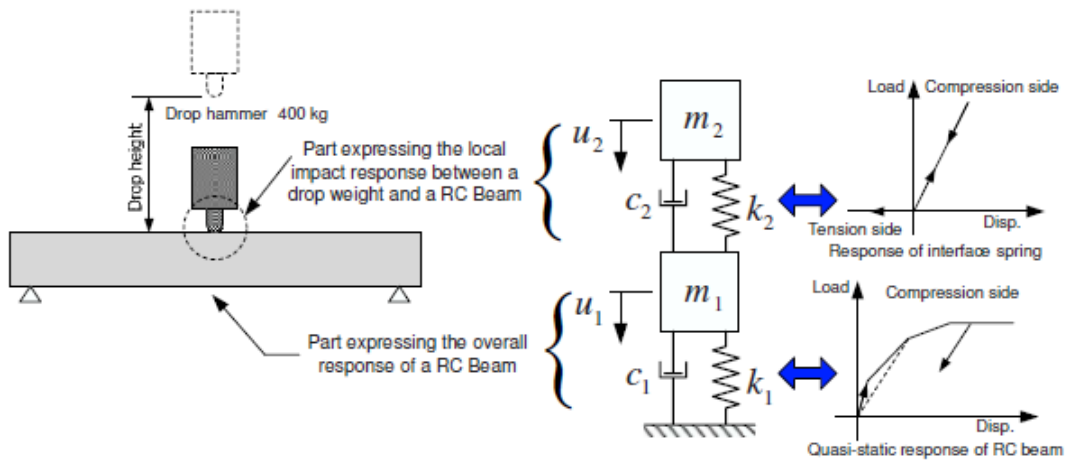


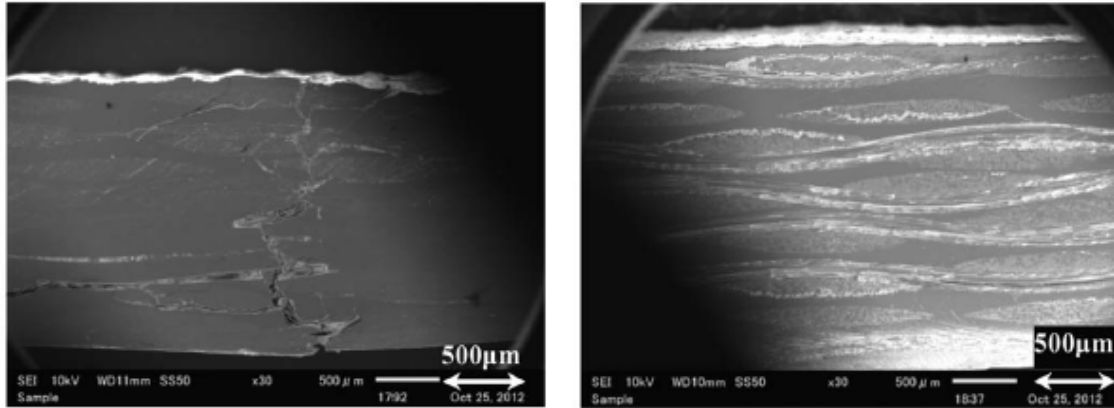
Figure 1.5 Two-Degree-of-Freedom Mass-Spring-Damper System (Fujukaka et al., 2009)

investigate the dynamic response of the CFT beam subjected to drop weight impact.

Theoretical sectional analysis (TSA) was introduced to evaluate the dynamic plastic moment capacity. A linear relationship between initial impact energy E_i and absorbed plastic energy E_a was found as $E_a = 0.634 E_i$ in their study. The rest of the energy was elastic energy. The design procedure was proposed as follows:

1. Given: clear span L_o , the initial impact energy E_i , support rotation limit θ ;
2. Obtain plastic energy E_a of CFT beam from the linear relationship $E_a = 0.634 E_i$;
3. Calculate required dynamic moment capacity M_r by using $M_r = E_a / 2\theta$;
4. Determine CFT dimensions and parameters of materials;
5. Determine dynamic plastic moment capacity of a design beam M_p by using TSA;
6. Check the limit state requirement, $M_r \leq \phi M_p$.

Shi et al. (2014) presented the low-velocity response and compression after impact (CAI) assessment of recycled carbon fiber-reinforced polymer (CFRP) composites. Three types of composite laminate were used: virgin CFRP (V-CFRP), recycled CFRP (R-CFRP) and treated recycled CFRP (TR-CFRP). After the impact testing, the major damage differed for three laminates due to fiber surface state: fiber failure for V-CFRP, fiber failure and some delamination for TR-CFRP, and delamination for R-CFRP. These two types of the damage are shown in **Fig. 1.6**. Damage resistance of TR-CFRP was improved up to 80% of V-CFRP



(a) Fiber Failure

(b) Delamination

Figure 1.6 Two Types of Damage (Shi et al., 2014)

by surface cleaning compared to 50% of V-CFRP for R-CFRP. The surface cleaning was done by soaking the recycled woven fabric in NMP (Kanto Chemical, Japan) for three days at 200°C, and then the carbon fibers were cleaned by an ultrasonic washing machine for one hour.

1.3 Over-Height Vehicle Collisions Literature Review

Collisions between highway or railroad bridges and over-height vehicles have been a common occurrence in recent years. The vehicles with height exceeding the vertical clearance permit will impact some bridge superstructures and lead to the damage of the beam or the collapse of the whole bridge. Fu et al. (2004) reported that the frequency of overheight accidents with highway bridges in Maryland increased by 81% between 1995 and 2000. A survey that was sent by Fu et al. (2004) to collect national statistics showed

that of the 29 states responding, 62% indicated that they consider overheight collisions to be a significant problem. In their study, the percentage of vehicles involved in total overheight vehicle accidents was also studied, 36% for enclosed box trailers, 31% for flatbed trailers with oversized loads, 16% for dump trucks and 17% for others. Agrawal and Chen (2008) indicated that from the analysis of the New York State Department of Transportation (NYSDOT) bridge hits database that a majority of bridge hits are by overheight vehicles.

Abendroth et al. (2004) examined the steel diaphragm in prestressed concrete girder bridges under impact load. Finite element (FE) models for PC-girder bridges were developed and the FE technique was validated by experimental results from previous research. The lateral impact loads were applied to the bottom flange of the exterior girders at diaphragm locations and away from diaphragms. A reinforced concrete diaphragm and two types of steel intermediate diaphragms were analyzed. A constant impact load over a short period of time was applied to all the bridge models in the simulation. The value of the maximum impact load was selected such that the induced maximum principle tensile stress would not exceed the modulus of rupture of concrete in the girder. A 534 kN (120-kip) load was applied to the girder location with an intermediate diaphragm, and a 267 kN (60-kip) load was selected at the location without intermediate diaphragm. The duration time of 0.1 second was adopted since the collision times were in

the range of 0.05 to 0.15 seconds based on the literature search. The results showed that the intermediate diaphragms could reduce the impact damage to the PC girders. When the load was applied at the diaphragm location, the reinforced-concrete diaphragm provided more protection than that of the two types of steel diaphragms.

Yang et al. (2010) performed dynamic finite element (FE) analysis by using software Abaqus to study the effects of overheight truck impacts on intermediate diaphragms in prestressed concrete bridge girders. The FE models were validated by the existing testing data from Abendroth et al. (2004). Parametric studies were conducted to evaluate the key factors including location of intermediate diaphragms, size of intermediate diaphragms, girder types, truck speed and impact force. The FE model considered the elastic-plastic behavior of concrete by using the Abaqus built-in concrete plasticity damage model. Plastic strains under different stresses for concrete compression were imported and tensile strengths after cracking were included to simulate concrete tensile softening behavior. The results demonstrated the importance of intermediate diaphragms on impact protection of the bridges under impact. A full-depth intermediate diaphragm with minimum allowed thickness was suggested in the design guidance in order to improve the impact resistance of PC girders. Intermediate diaphragm spacing and location were recommended as follows: intermediate diaphragms spacing of 6.10 m (20 ft) to 12.2 m (40 ft) for bridges with a span of 30.5 m (100 ft) or longer; either two intermediate

diaphragms at the $1/3$ span points or three intermediate diaphragms at the $1/4$ span points for bridges with a span between 15.2 m (50 ft) and 30.5 m (100 ft); one intermediate diaphragm at the center for bridges with a span less than 15.2 m (50 ft).

Xu et al. (2012) conducted a series of scaled model tests to simulate the collision between over-height truck and bridge superstructure. The purpose of this experimental study was to observe the response and failure modes of the girder during collision. A steel box girder, a steel plate girder and a reinforced concrete (RC) T-beam girder were adopted as the testing models. A cylindrical tank was selected to represent a typical over-height truck. A pendulum trajectory was followed by the tank to impact the girders. Similarity scale ratios in geometry, material properties, load, and dynamic properties were calculated from the Buckingham π theory in their study. The geometry scale ratio 0.2 was used, the material properties ratio was set as 1.0, the similarity ratios of load, time, and speed were calculated as 0.04, 0.2 and 1.0. The experimental layout is presented in **Fig. 1.7**.

There was no residual deformation on the steel box girder while permanent deformation on the tank observed. Large local deformation was found in the collision region of steel plate girder, and deformation were observed for both tank and the girder. However, the damage of the tank was much smaller than that of the steel box girder. For RC T-beam, numerous cracks and serious concrete damage were observed after collision, and

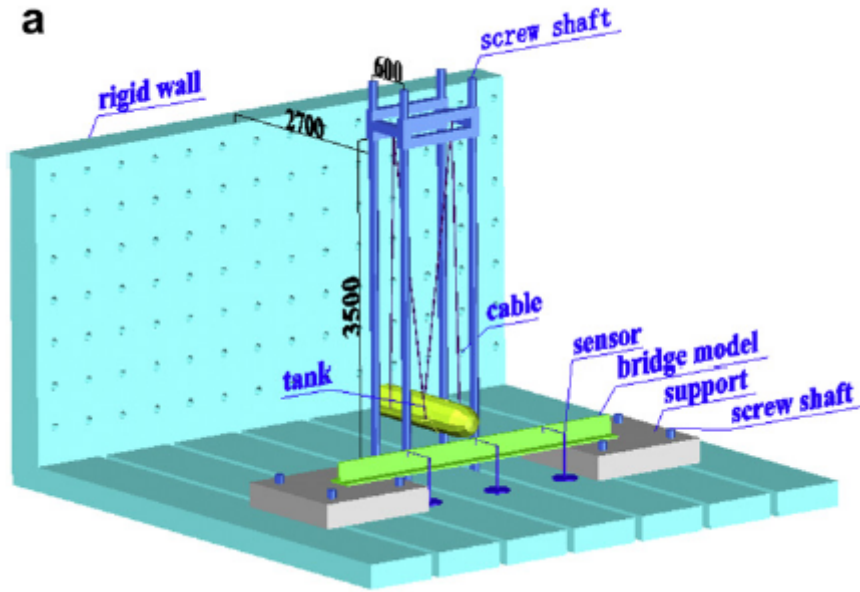


Figure 1.7 Three-Dimensional View of Experimental Layout (Xu et al., 2012)

permanent indentation was left on the tank. Finite element simulation was conducted using MSC. MARC to validate the testing data. Both results indicated that local failure was found to be the main failure mode for the steel plate and the RC T-beam while global failure for the steel box girder.

Xu et al. (2013) used finite element (FE) software MSC. MARC to simulate the collision between overheight trucks and bridge superstructures, and a simplified model used to estimate the design impact force was also proposed. Three types of bridge superstructures were considered in their study: a prestressed concrete (PC) T-girder bridge, a steel box-concrete slab composite bridge, and a three-span PC box girder bridge. A standard double-axle truck, container truck, tipper truck and tank truck were modeled.

Three different collision speeds were adopted: 30 km/h (19 mph), 60 km/h (37 mph) and 90 km/h (56 mph). **Fig. 1.8** shows the FE models of the container truck and the PC T-girder bridge.

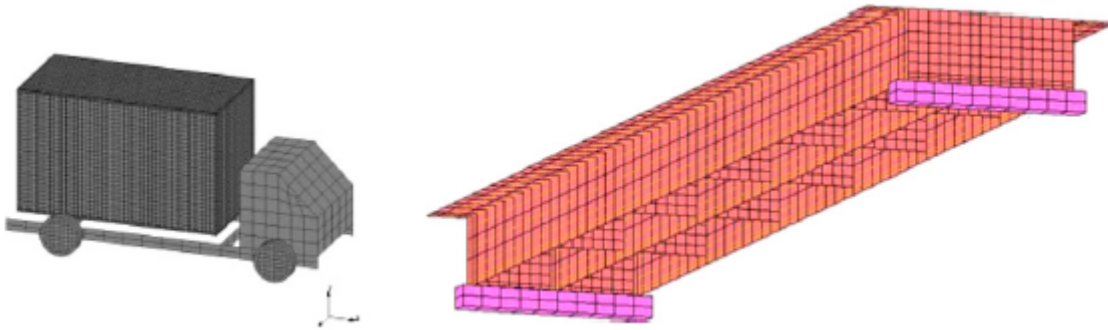


Figure 1.8 FE Models of Container Truck and PC T-Girder Bridge (Xu et al., 2013)

Damage to the bridge superstructure from collisions mainly resulted from global deformations and local punching forces. Longitudinal and diagonal concrete cracking, and yielding of steel reinforcement bars were observed for PC T-girder bridge. Severe yielding of the steel bridges' web occurred. Since the overall weight and stiffness of the bridges are much larger than those of the overheight trucks, the bridge was modeled as a rigid wall in the simplified model, which is presented in **Fig. 1.9**.

In the simplified model, the displacement response of the overheight truck was a combination of the horizontal and vertical translations and rotation around the rear axle.

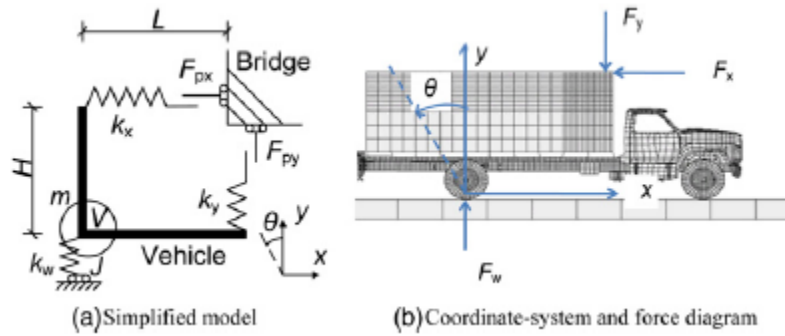


Figure 1.9 Simplified Model (Xu et al., 2013)

Finally, the design collision forces were suggested in the range of $F_x = 700\text{-}950\text{ kN}$ (157-214 kips) and $F_y = 650\text{-}850\text{ kN}$ (146-191 kips) for $V = 30\text{ km/h}$ (19 mph); $F_x = 800\text{-}1900\text{ kN}$ (180-427 kips) and $F_y = 700\text{-}1650\text{ kN}$ (157-371 kips) for $V = 60\text{ km/h}$ (37 mph); and $F_x = 2000\text{-}2600\text{ kN}$ (450-585 kips) and $F_y = 2000\text{-}2500\text{ kN}$ (450-562 kips) for $V = 90\text{ km/h}$ (56 mph).

2. DESIGN OF FULL-SCALE LATERAL IMPACT TESTING FACILITY AND DATA ACQUISITION SYSTEM

2.1 Impact Testing Facility Options

In this study, a decision was made to build an outdoor full-scale impact testing facility in order to understand the behavior of bridge superstructures under lateral impact loading. Several options were considered during the design of the full-scale lateral impact testing facility. Although drop weight tests had been adopted by many researchers, the direction of impact in drop weight test was the same as the direction of gravity not like the actual lateral impact. Drop weight would cause more damage to bridges than lateral impact. Also in the over-height vehicle impact situation, both horizontal and vertical forces are applied to the bridge superstructures, but drop weight test is a one directional impact. For the pendulum test, the control of pendulum arm from impacting the specimen more than once would be a difficult technical issue and the cost was determined to be beyond the project budget limit. After considering safety, cost, and construction time involved, an impact cart with an elevated track were selected as the final testing setup. The initial speed of the impact cart was provided by its potential energy when the cart rolled down the track.

2.2 Design Requirements

Collision process between over-height vehicle and bridge superstructure is very complicated since vehicle mass, vehicle velocity, impact area, impact angle, and energy absorbed by bridge or vehicle are all variables. In a real traffic situation, truck moves at 72 km/h (45 mph), 97 km/h (60 mph), or higher speed. It is quite difficult to quantify the exact impact energy going into the structure when collision occurs because the truck may not completely stop and would also experience large plastic deformation during collision.

The testing facility described in this dissertation was designed based on a car crash study. Zaouk et al. (1996) demonstrated the results of a computer simulation of a frontal impact of a Chevrolet C-1500 pick-up truck with a rigid wall at an initial velocity of 56 km/h (35 mph) with 0-degree impact angle. The truck before and after impact is illustrated in **Fig.**

2.1. The non-linear finite element vehicle model was calibrated by the data obtained from the impact testing. The energy absorption was analyzed in the simulation by computing the material internal energies. The energy absorbed by the vehicle at the complete stop

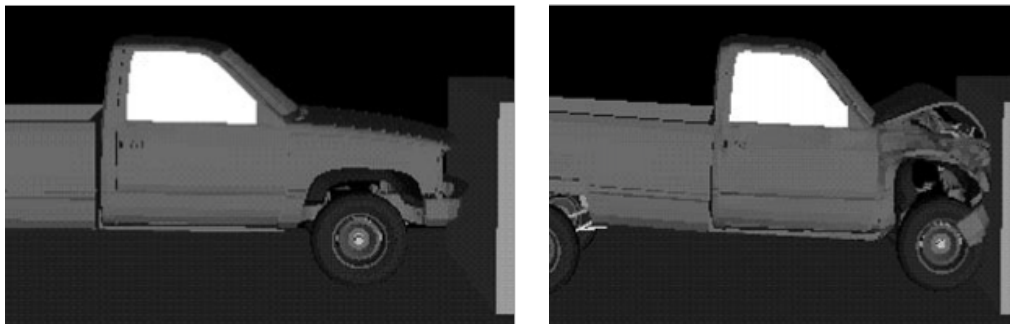


Figure 2.1 Truck Before and After Impact (Zaouk et al., 1996)

was determined to be 214 kilojoules (158 kip-ft). Based on their study, a total initial impact energy of 100 kilojoules (74 kip-ft) was chosen to design our full-scale impact testing facility. Almost half of the absorbed energy was chosen to determine the height of the track and the mass of the impact cart. The track system was required to support the weight of the cart and also to make no permanent change to the testing site. This amount of initial energy was proved reasonable to meet the construction requirements, and at the same time to simulate a relative severe situation during vehicle impact since the specimen in this study was expected to absorb almost all the energy.

2.3 Construction of Impact Testing Facility

The full-scale lateral impact testing apparatus consists of an impact cart, a track system, a backstop system and vertical supports.

2.3.1 Impact Cart

The weight of the impact cart was measured as 4080 kg (9000 lb.) after construction. The impact cart is a 1.42 m³ cubic foot (50 ft³) concrete block with a 25.4 cm (10 in.) x 25.4 cm (10 in.) x 25.4 cm (10 in.) impactor confined with steel plates on four sides, as shown in **Fig. 2.2**. Eight casters are also attached to the steel frame, four of which are used to support the weight of impact cart, and the other four act as side wheels to provide straight tracking.



Figure 2.2 Impact Cart

2.3.2 Track System

The track system consists of posts, bracing, work platforms and a rail system. The bottom of the track is set at a height of 0.31 m (1 ft), and the height at the top of the track is 3.35 m (11 ft). The impact cart needs to roll down a smooth surface to create an impact with the specimen and the surface is provided by rails that are placed on top of the post lines. **Fig. 2.3** illustrates the track system used in the impact testing, as well as an excavator with a chain that will be explained later.

2.3.3 Backstop and Support

The backstop was designed to provide lateral support that prevents the specimen from sliding horizontally during the impact. As shown in **Fig. 2.4**, the backstop consists of two wide-flange steel beams that are set in a deep concrete foundation, which was designed to prevent the wide-flange steel beams from rotation (Mitchell, 2014). The steel tubes are



Figure 2.3 Track System and Excavator



Figure 2.4 Backstop System

removable so that multiple specimens with different widths can be tested using this facility. The center of the impactor is designed to impact the specimen near its bottom and the vertical support is illustrated in **Fig. 2.5**.



Figure 2.5 Support System

Fig. 2.6 shows the full-scale lateral impact testing facility. Before the impact, the cart was connected to an excavator by a chain and a shackle that was connected to the pull hitch on the cart (**Fig. 2.3**). Once the cart was pulled up the track, the bucket of the second excavator was placed on the front side of the impact cart to prevent it from rolling down. Once the second excavator had secured the cart, the chain was slacked and shackle was unfastened. The bucket of the second excavator was raised to release the cart. The impact cart rolls down the track and the center of the impactor impacts the bottom of the specimen.

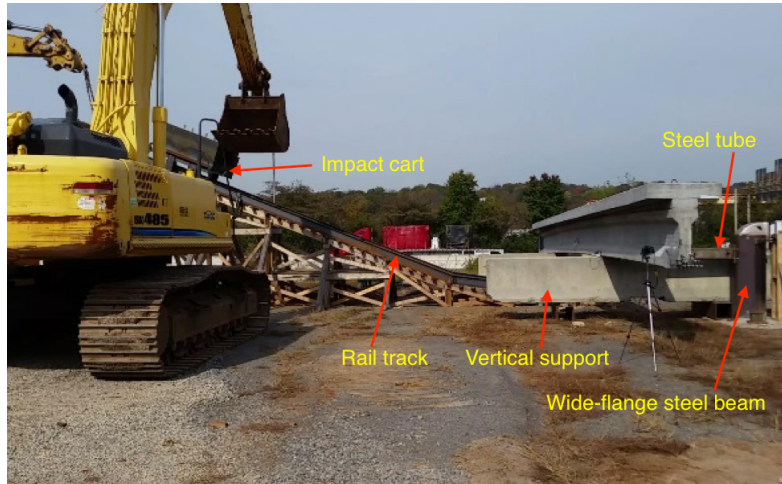


Figure 2.6 Full-Scale Lateral Impact Testing Facility

2.4 Data Acquisition System

The data collection during our full-scale lateral impact tests was accomplished by using different types of sensors and National Instruments (NI) data acquisition (DAQ) hardware and software. Accelerometer, strain gage and string potentiometer were used in this study to obtain acceleration, strain and displacement. DAQ hardware acts as a connection between signal and computer, and it is a device that digitizes analog signals so that computer can interpret them. NI SCXI-1001 and NI cDAQ-9172 DAQ measurement devices were used. As presented in **Fig. 2.7**, the wires are connected to the physical channels on the terminal blocks of the measurement device, and the other sides of the wires are connected to the sensors. A programmable software LabVIEW was installed on the computer. This software can control the operation of the DAQ measurement device and can provide communication between the user and the computer for acquiring and storing data.

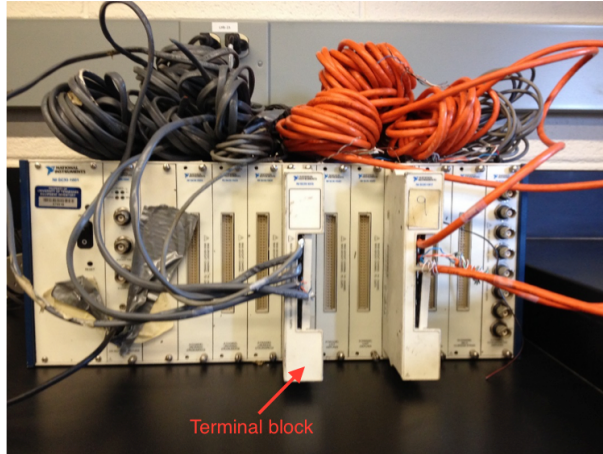


Figure 2.7 NI-SCXI-1001 DAQ Measurement Device

The source code for data acquisition during our impact testing was developed in LabVIEW block diagram. The programming language in LabVIEW is a graphic programming language. On block diagram, the DAQmx functions were connected by drawing wires. **Fig. 2.8** shows a simple example of a strain measurement code that controls the data flow.

In our impact testing, four different sensors and two measurement devices were used. When writing code on block diagram, sensors corresponding to the physical channels on device NI SCXI-1001 were connected in series and they had a parallel connection with the sensors on NI cDAQ-9172 DAQ. **Fig. 2.9** illustrates strain gage virtual channels that are corresponding to physical channels on the measurement device. Maximum and minimum values, and gage information etc. were input on front panel. Front panel is a user interface and includes controls and indicators, as shown in **Fig. 2.10**. Sample clock function-node defines a sampling rate of the data, which controls the frequency of data flowing as

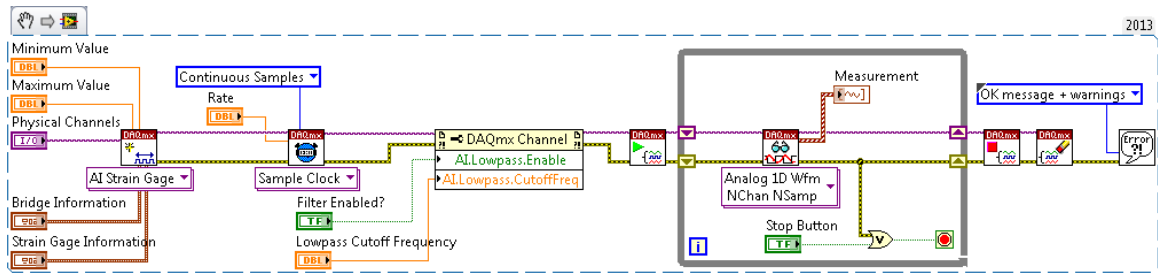


Figure 2.8 Example Code of Strain Measurement (National Instruments, 2016)

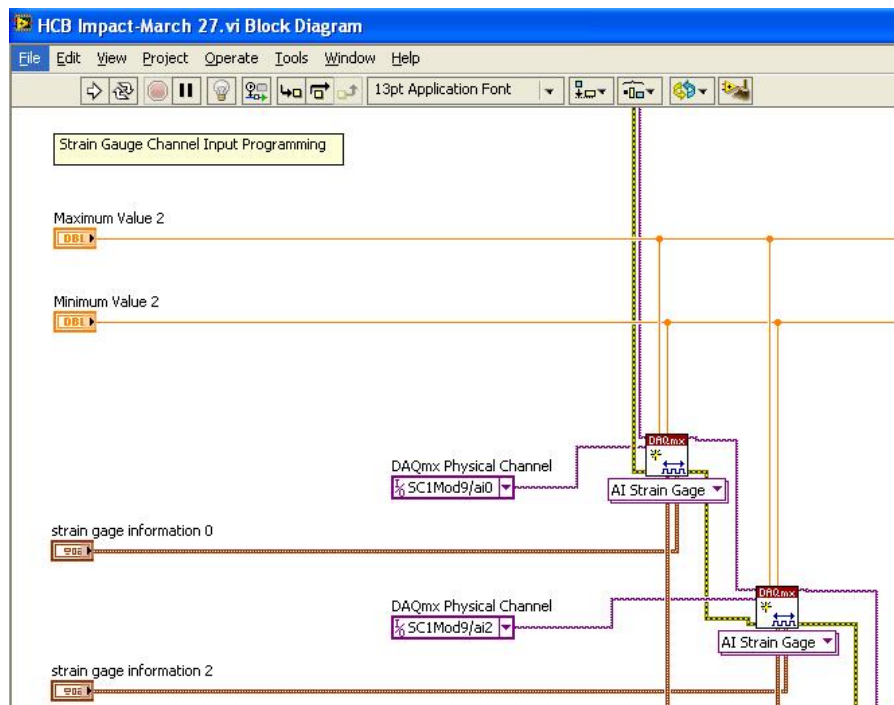


Figure 2.9 Virtual Channels of Strain Gauges

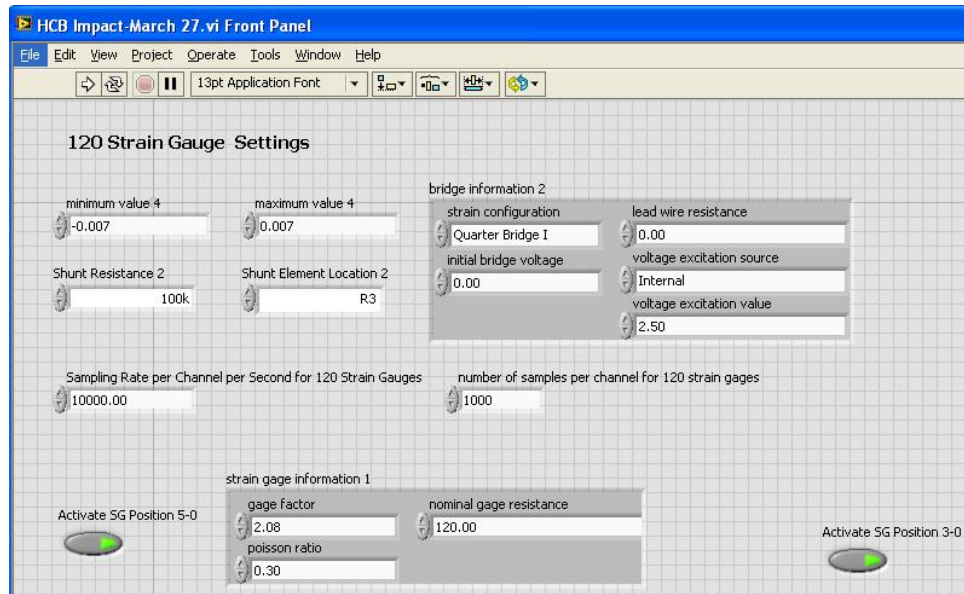


Figure 2.10 Front Panel

presented in **Fig. 2.11**. In our impact testing, a sampling rate of 10 kHz was selected. Data was read continuously when the while loop structure was used (**Fig. 2.12**). Because the testing data were collected from different sensors, all the bundled signals were split to four groups and then merged within each group and written to separate files. **Fig. 2.13** lists all the data monitoring plots on front panel.

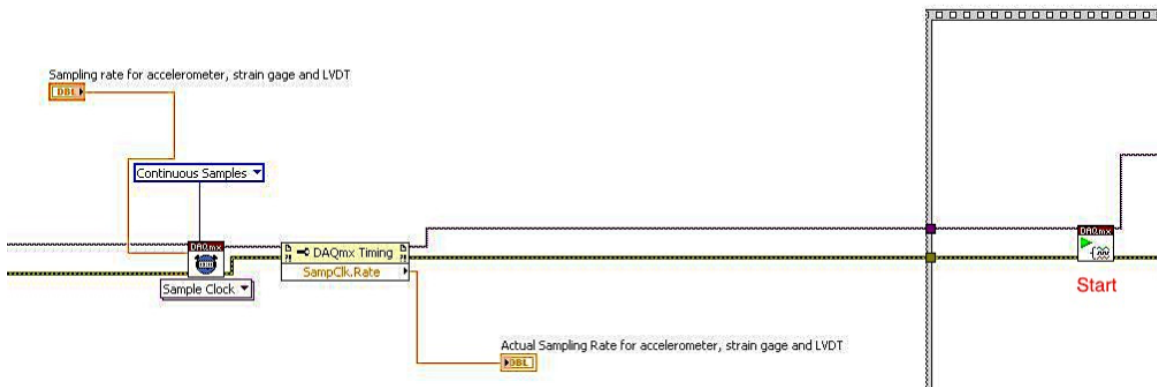


Figure 2.11 Sample Clock

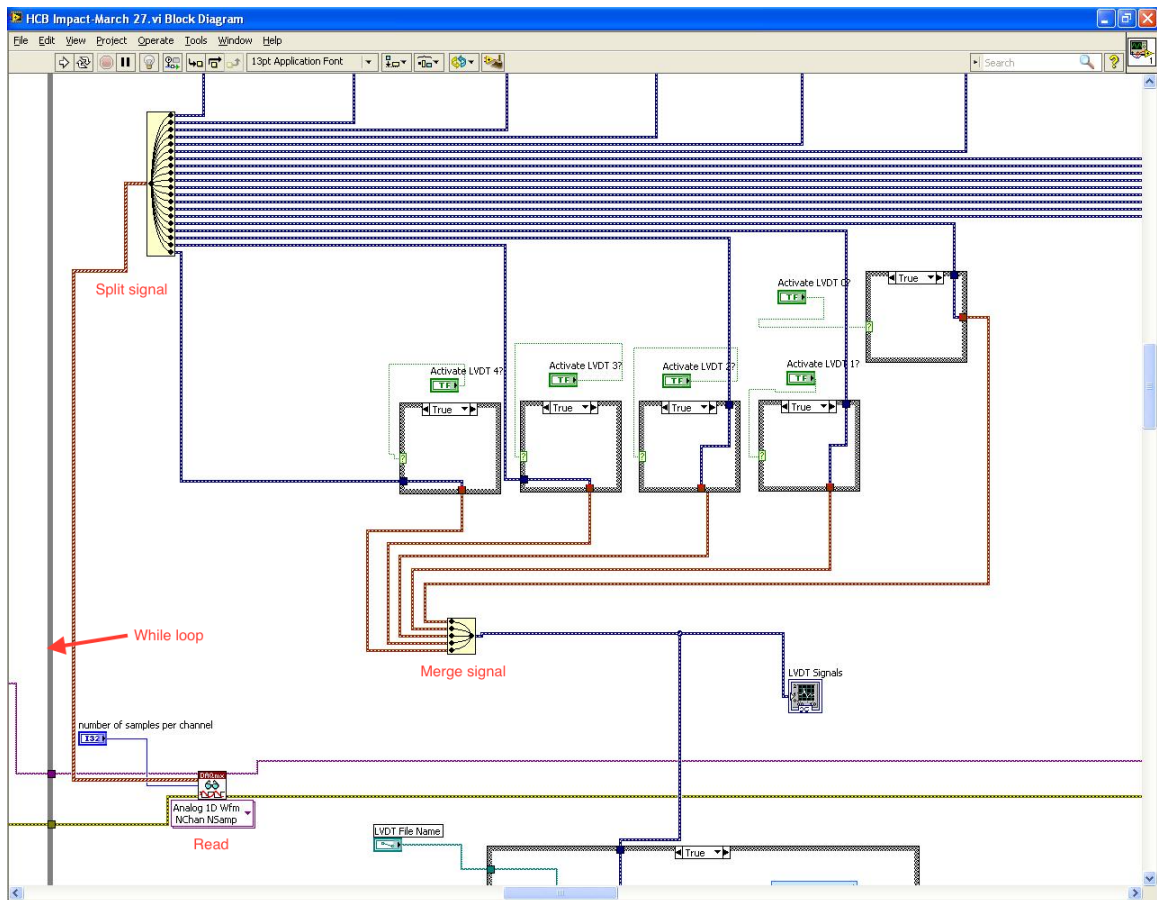


Figure 2.12 Data Split and Merge

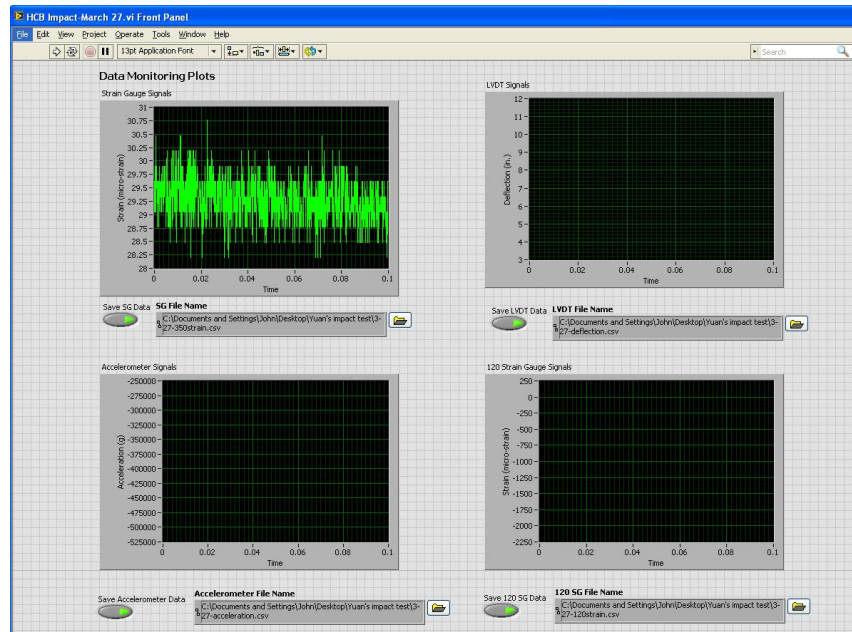


Figure 2.13 Data Monitoring Plots

3. FULL-SCALE LATERAL IMPACT TESTS OF PC GIRDER AND HYBRID COMPOSITE BEAM BRIDGE

3.1 Impact Test of PC Girder

3.1.1 Specimen

The first lateral impact was conducted in November, 2014. The specimen used in the impact test was an AASHTO Type I prestressed concrete girder with 18-mm (0.7-in.) strands. The compressive strength of the concrete girder was 97 MPa (14100 psi), and the compressive strength of the concrete deck was 73 MPa (10600 psi), which is higher than that of a typical deck in order to use a much narrower width of the deck for testing. The length of the girder was 17 m (56 ft). This girder was statically tested for shear at both ends by Cabage (2014). After the static test was completed, the beam was shipped to the impact testing site. This prestressed concrete girder was set up so that the middle portion of the girder could be tested by impact while leaving the failed girder ends from the static testing cantilevered out from the supports. The cross section view and properties of the prestressed concrete girder are shown in **Fig. 3.1** and **Table 3.1**, respectively. The track system, impact cart and PC girder setup are displayed in **Fig. 3.2**.

3.1.2 Instrumentation

National Instruments (NI) data acquisition system was used during the impact test. The acceleration, displacement and strain were recorded by three accelerometers, four string

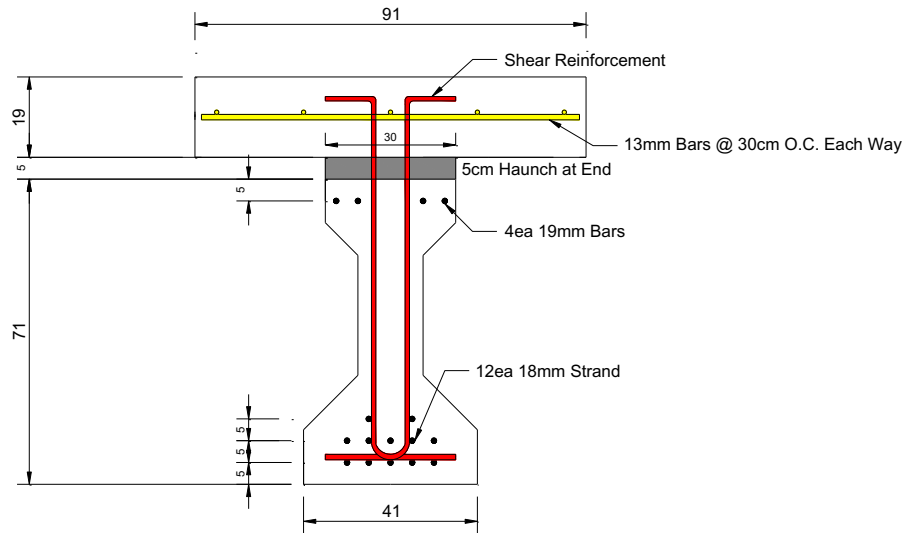


Figure 3.1 Prestressed Concrete Girder Cross Section View (cm); 1 cm=0.39 inch

Table 3.1 Beam Properties

	Beam Properties without Deck	Composite Section Properties
Area (cm ²)	1781	3652
I_x (cm ⁴)	946926	3032246
I_y (cm ⁴)	126867	1320369
Weight (kg/m)	429	878
f'_c (MPa)	97	-----
Strand: 18-mm diameter, low relaxation, $f_{pu}=1862$ MPa		
Area of Strand, $A_p= 1.90$ cm ²		
Strand Stress before Transfer, $f_{pi} = 1255$ MPa		
Strand Stress after Losses, $f_{se} = 1048$ MPa		

Note: 1 MPa=145 psi; 1 cm=0.39 inch; 1 kg/m=0.67 lb./ft.

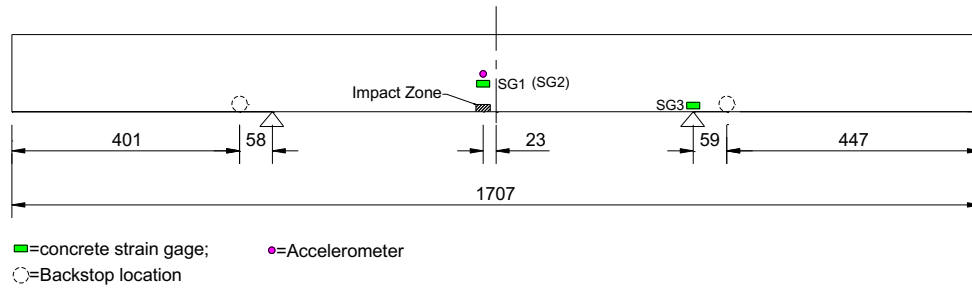


Figure 3.2 Prestressed Concrete Girder Setup

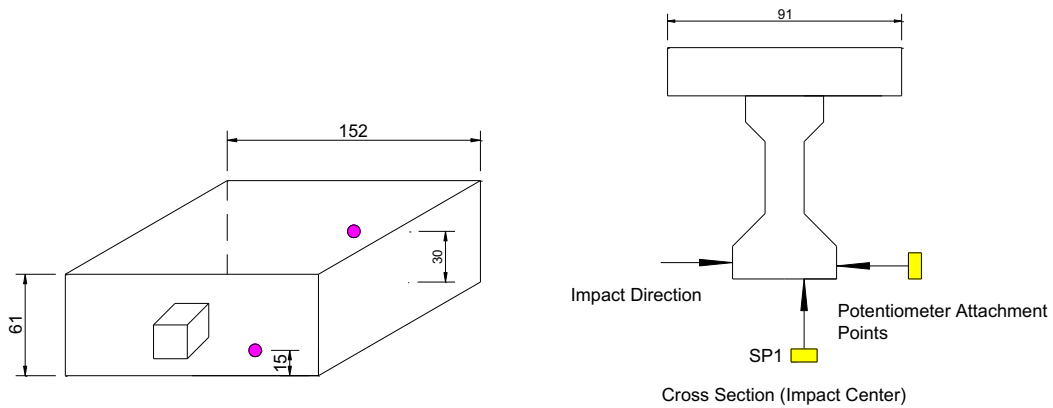
potentiometers and eight strain gages with a sampling rate of 10-kHz. **Fig. 3.3** shows the locations of all the sensors used in the test. Three concrete strain gages were put on the girder. Two accelerometers were attached to the impact cart, and the last one was above the impact zone on the girder. Two string potentiometers were attached to the bottom flange in order to obtain the girder horizontal and vertical movements, and the other two were clamped to the top of wide-flange steel beams. Five strain gages were bounded to two wide-flange steel beams.

3.1.3 Dynamic Behavior of Prestressed Concrete Girder

Videos were taken from several views during the testing. **Fig. 3.4** and **Fig. 3.5** show the collision process from side view and rear view of the girder, respectively. The picture of $t=0$ s is a moment before impact. At $t=0.033$ s, the specimen bends in the lateral direction

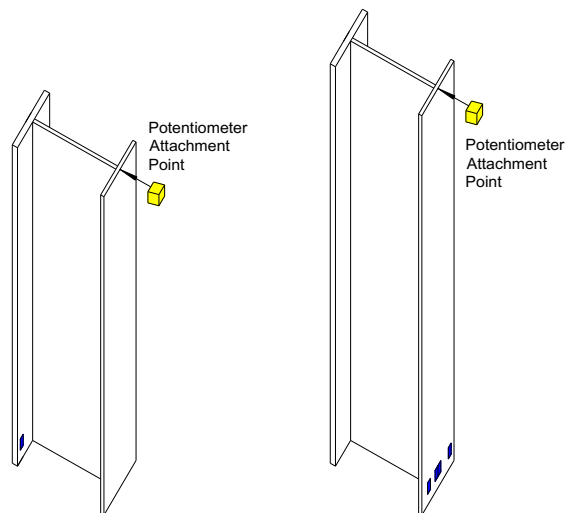


(a) Front View ("SG2" located on the back)



(b) Impact Cart with Two Accelerometers

(c) Two String Potentiometers on the Specimen



(d) String Potentiometers and Strain Gages on Wide-Flange Steel Beams

Figure 3.3 Positions of Sensors (cm); 1 cm=0.39 inch



t=0 s



t=0.033 s



t=0.067 s



t=0.100 s



t=0.133 s



t=0.167 s



t=0.200 s

Figure 3.4 Collision Process from Side View



$t=0\text{ s}$



$t=0.033\text{ s}$



$t=0.067\text{ s}$



$t=0.100\text{ s}$



$t=0.133\text{ s}$



$t=0.167\text{ s}$



$t=0.200\text{ s}$

Figure 3.5 Collision Process from Rear View

and concrete particles fall off the impact zone. Two diagonal cracks can be observed from the back side of the girder, and a horizontal crack between web and bottom flange initiates from the left diagonal crack. At $t=0.067$ s, the girder continues to bend in the impact direction and concrete spalling occurs in a larger area. At the back side of the girder, horizontal cracks between the web and bottom flange tend to extend to the center of the girder. At $t=0.100$ s, besides horizontal bending and additional concrete spalling, the specimen can be observed to move upward with a deeper penetration of the impactor. Small concrete blocks also fall off from the backside of the girder, which is called scabbing. At $t=0.133$ s, the specimen continues to bend horizontally and move upward. Concrete blocks spall off around impact zone along the whole thickness of the PC girder and secondary diagonal cracking extending to top flange can be obviously observed from the rear view. From $t=0.167$ s to $t=0.200$ s, the specimen moves forward and upward with large concrete spalling and scabbing, and all prestressing strands are uncovered. During the whole impact process, the concrete deck bends more than the PC girder in the impact direction. The failure of the specimen began with punching shear around the impact zone due to the relatively high rigidity of the impactor, as well as the high compressive strength of the PC girder. This phenomenon can also be observed from girder backside diagonal cracks and horizontal cracks between web and bottom flange. With the penetration of the impactor, the damaged impact zone behaves as a “hinge” which moves upward due to the heavy weight of both overhangs.

3.2 Impact Test of HCB Bridge

3.2.1 Introduction

Limited experimental and analytical studies of HCB have been performed in the last few years. An initial study of HCB was the High Speed Rail (HSR) program HSR-23, in which a 6.10 m (20 ft) HCB using 3.49 cm (1-3/8 in.) diameter post-tensioning bars as tension reinforcement, with high strength non-shrink grout for the arch, was tested to failure (Hillman, 2012). The HCB beam failed at an ultimate load that was 180% of the factored design load required by code. During project HSR-43, a full-size 9.14 m (30 ft) prototype railroad bridge using HCB was constructed and tested (Hillman, 2012). This HCB bridge consisted of 8 beams. Self-consolidating concrete (SCC) was used for the concrete arch, and tension reinforcement consisted of steel fibers running along the bottom flange of the beam. Live load tests on this prototype HCB railroad bridge were conducted by using a heavy axle freight train. The static loads were applied by positioning of the axle loads of the freight train and the dynamic test was performed under live train operations with speeds ranging from 8 km/h (5 mph) to 72 km/h (45 mph). Stresses and deflections were monitored during the tests and the bridge was found to behave as predicted.

A full-scale HCB unit was fabricated and tested before the replacement of the Knickerbocker Bridge in order to confirm its strength, stiffness, and durability (Snape and Lindyberg, 2009). Static shear, bending and fatigue bending tests were conducted under

a 4-point static loading. Coupon tests for composite skins of the HCB and coupon tests for composite skins subjected to ultra-violet (UV) light exposure were also performed. The analytical model developed by John Hillman was verified by the beam behavior under service loads, and significant improvements to insure the performance and safety of the product during fabrication were also suggested in their study. Ahsan (2012) reported on the testing of HCBs prior to their use for the replacement of a skewed bridge (Tide Mill Bridge). Individual HCBs without the concrete arch, individual HCBs with the concrete arch, and a three-HCB system with a 45-degree skew were tested under different service loading configurations. Testing results showed that components of the HCB behaved linear-elastically under service loads except for the concrete arch. Recommendations were made for the analysis of the concrete arch, and HCB was shown to be adequate for use in the Tide Mill Bridge. Aboelseoud et al. (2014) conducted a field evaluation of one of the HCB bridges constructed in Missouri in order to analyze the behavior of HCB and to examine the current design assumptions. Two trucks were used to apply three different load cases and it was found that the current design method significantly overestimated the beams' deflections and tensile stresses in the different elements. A finite element (FE) model of another HCB bridge in Missouri was developed to provide deeper insight into its structural behavior (Aboelseoud and Myers, 2015). The study showed that FE analysis could provide acceptable accuracy of beam behavior. The study indicated that HCB might have lateral and rotational deformations under vertical loads.

An issue might occur in the event of a HCB bridge being subjected to lateral impact from over-height vehicles passing under the bridge, as mentioned in Hillman's (2012) report. Though the behavior of this novel system has been studied during the last few years through limited tests, impact testing has not yet been conducted.

3.2.2 Specimen

A 9.14 m (30 ft) prototype HCB railroad bridge was constructed and tested on the Facility for Accelerated System Testing (FAST) at the Transportation Technology Center, Inc., (TTCI) in Pueblo, Colorado in 2007 (Hillman, 2008). This prototype bridge consists of two assemblies, of which one assembly contains four HCB units bolted together with tie-rods, a 10 cm (4 in.) concrete deck and a ballast curb. The bridge was subjected to an equivalent Cooper E-60 static load and approximately 0.25 Million Gross Tons of live loading. There was no deterioration of the structural members measured except for some shear cracking of the concrete deck at both ends of the span. A thicker deck was cast after the removal of the original deck. The specimen used in the impact testing described herein is one assembly of this prototype railroad bridge with replaced concrete deck, and the cross section view of the specimen is shown in **Fig. 3.6**. In order to simulate the ballast above the HCB railroad bridge, New Jersey barriers with equivalent weight are placed on top of the concrete deck. **Fig. 3.7** illustrates the HCB bridge set up.

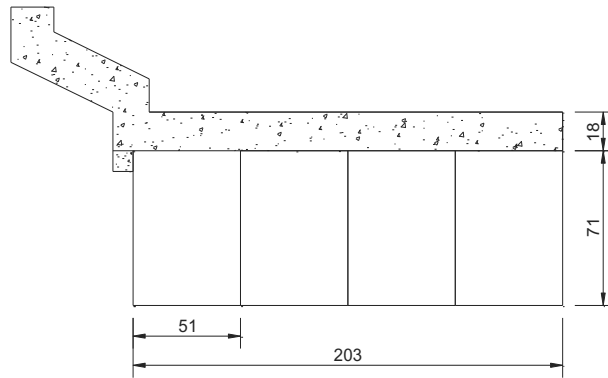


Figure 3.6 Cross Section View of HCB Bridge (mm); 1 mm=0.039 inch



Figure 3.7 HCB Bridge Setup

The prototype HCB railroad bridge had SCC pumped into the arch conduit within the beam shell with the thickness of the concrete arch being 11.4 cm (4.5 in.). The tension reinforcement, named as Hardwire®, is a laminate preform from Hardwire, LLC., comprised of high strength twisted steel wires in parallel cords to form a unidirectional tape. **Fig. 3.8** shows the cutting of Hardwire® tape (Hillman, 2005). In the HCB bridge, the 3x2 cord type was used, which was made by twisting 5 individual wire filaments: 3 straight filaments wrapped by 2 filaments at a high twist angle. The diameters of filament and cord are 0.35 mm (0.014 in.) and 0.89 mm (0.035 in.), respectively. Nine layers of the Hardwire® tape were laid up and infused at the same time as the FRP shell using the same vinyl ester resin and, the total thickness of the tension reinforcement is 1.14 cm (0.45 in.) in the HCB unit. Before HCB bridge lateral impact testing, a small piece of FRP shell was cut off the bottom surface of the bridge in order to apply a strain gage to the tension reinforcement, as illustrated in **Fig. 3.9**. The FRP shell is made of a vinyl ester resin reinforced by glass fibers with a thickness of 0.37 cm (0.14 in.), and it includes a top flange, a bottom flange, two vertical webs and a continuous conduit. Low density foam fills the gap between the concrete arch and the tension reinforcement. The FRP shell encapsulates all the components of the beam. Detailed material properties of each part are summarized in **Table 3.2**, in which the steel fiber laminate properties are for after the tension reinforcement is infused by vinyl ester resin, which is the same matrix for glass fibers to form the FRP shell.



Figure 3.8 Cutting of Hardwire® Tape (Hillman, 2005)



Figure 3.9 Hardwire® from Bottom View

Table 3.2 HCB Material Properties

Property	Type	Elastic Modulus	Density	Poisson Ratio	Yield Strength	Ultimate Strength
		(MPa)	(kg/m ³)		(MPa)	(MPa)
Concrete	SCC	30440	2307	0.2		41
Deck	Concrete	30440	2307	0.2		41
Steel	3x2-23-12	82254	3684	0.3	965	
Fiber	411-350					
Laminate	Vinyl Ester					
FRP Shell	Quad+Mat 4720-50	21374	1922	0.3		
	411-350 Vinyl Ester					
Foam	P200	5(PA) 3(PE)	32			0.186(PA) 0.138(PE) (C)
						0.283(PA) 0.179(PE) (T)

Note: PA-Parallel; PE-Perpendicular; C-Compression; T-Tension;

1 MPa=0.145 ksi; 1 kg/m³=0.062 lb./ft³.

3.2.3 Instrumentation

Small windows (**Fig. 3.10**) were cut out of the FRP shell in order to apply strain gages on the concrete arch. **Fig. 3.11** shows the positions of the sensors bonded to HCB bridge and impact cart. Accelerometer A6 was put on a New Jersey Barrier on top of the deck. Strain gages SG17-SG18 were attached to the top and bottom face of the left HSS steel tube of the backstop, and SG19-SG20 were on the right HSS steel tube.



Figure 3.10 Small Window Cutout of FRP Shell

3.2.4 Lateral Impact Testing

The second lateral impact test was conducted in March 2015. Before impact, the impact cart was connected to an excavator by a chain and a shackle that was connected to the pull hitch on the cart. Once the cart was pulled up the track, the bucket of the second excavator was placed on the front side of the impact cart to prevent it from rolling down, as shown in **Fig. 3.12**. Once the second excavator has secured the cart, the chain was

- 120 Strain Gage
- 350 Strain Gage
- Accelerometer
- ◆ String Potentiometer

- △ Support
- Backstop

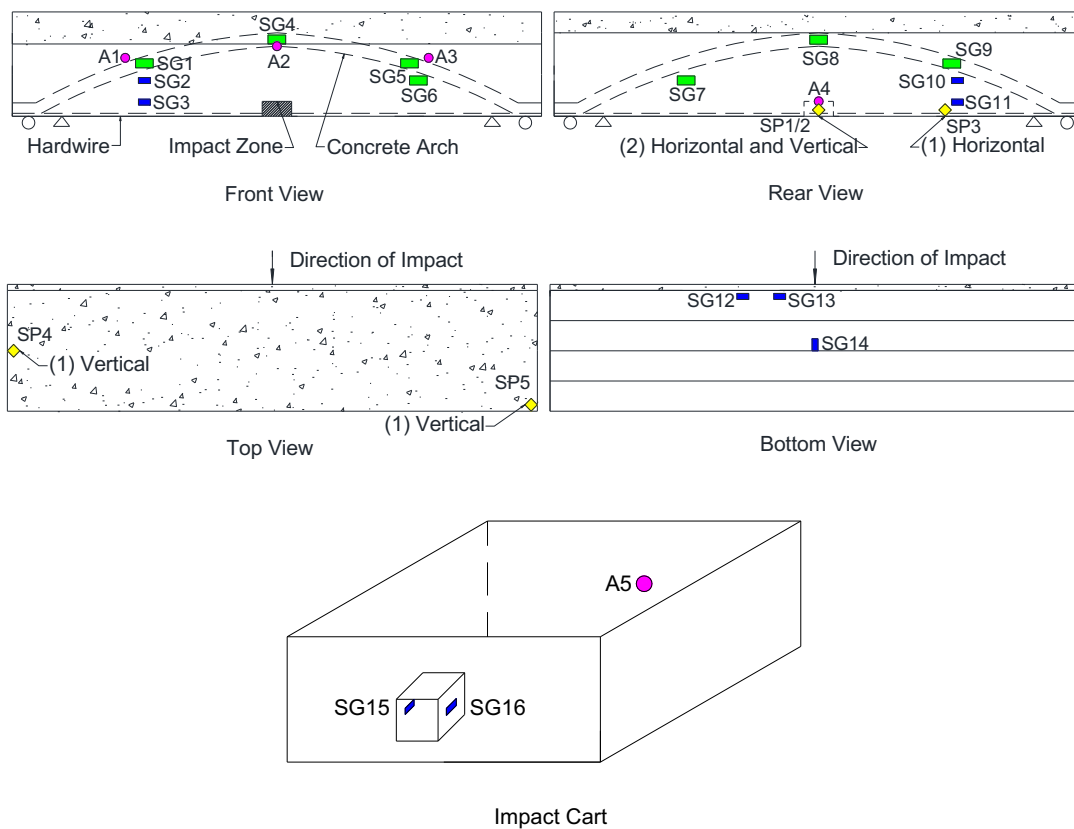


Figure 3.11 Sensor Positions on HCB Bridge and Impact Cart



Figure 3.12 Impact Cart and Second Excavator

slacked and shackle was unfastened. The bucket of the second excavator was raised to release the cart to impact the HCB bridge. Once the bucket of the second excavator was raised, the impact cart rolled down the rail track. A slight upward angle at the end of the track was designed to prevent a second time impact to the specimen from impact cart, because after first impact the speed of the cart was reduced dramatically and would not be able to roll uphill and overcome the gravity to impact the specimen again. However, due to the tremendous vertical impact, as well as a little uphill at the bottom of the track, the two front vertical wheels on the impact cart tried to shear off from the rail at the bottom section of the track. The front wheels rotated under vertical compression, at the same time local buckling occurred over a length of the rail. Because of this damage, the impactor partially missed the bridge and the impact area was 153 cm (60 in.) x 17 cm (6.75 in.) instead.

3.2.5 Damage of HCB Bridge during Collision

During the collision, there was no global failure of the HCB bridge. At the impact zone, since the thickness of the tension reinforcement is only 1.14 cm (0.45 in.) it is easy to deform in the direction of impact. At the same time, underlying the FRP shell, the majority of the material is low-density foam. Therefore, a lot of energy was absorbed through strain energy of the tension reinforcement and the foam. Local damage of the FRP shell at the front face and bottom face were observed after collision, as shown in **Fig. 3.13** and **Fig. 3.14**, respectively. The damage area of FRP at the bridge front face was approximately 56 cm (22 in.) x 2.54 cm (1 in.), and about 198 cm (78 in.) x 11 cm (4.25 in.) at the bottom face. Cracks emanated from the point of impact, and at the severe damage zone, the entire thickness of the FRP laminate piece was separated from the internal material. NCHRP Report 564-Field Inspection of In-Service FRP Bridge Decks (Telang, et al., 2006) provides details with respect to FRP damage types and inspection and evaluation methods. Large cracks around the impact zone, delamination between FRP lamina and the debonding of FRP shell from the interior low-density foam, tearing of FRP shell at the severe impact zone, and discoloration were all observed after the HCB bridge lateral impact testing.

FRP has brittle failure modes and impact can introduce damage to FRP laminate in the form of matrix cracking, fiber breakage and delamination between lamina layers. As

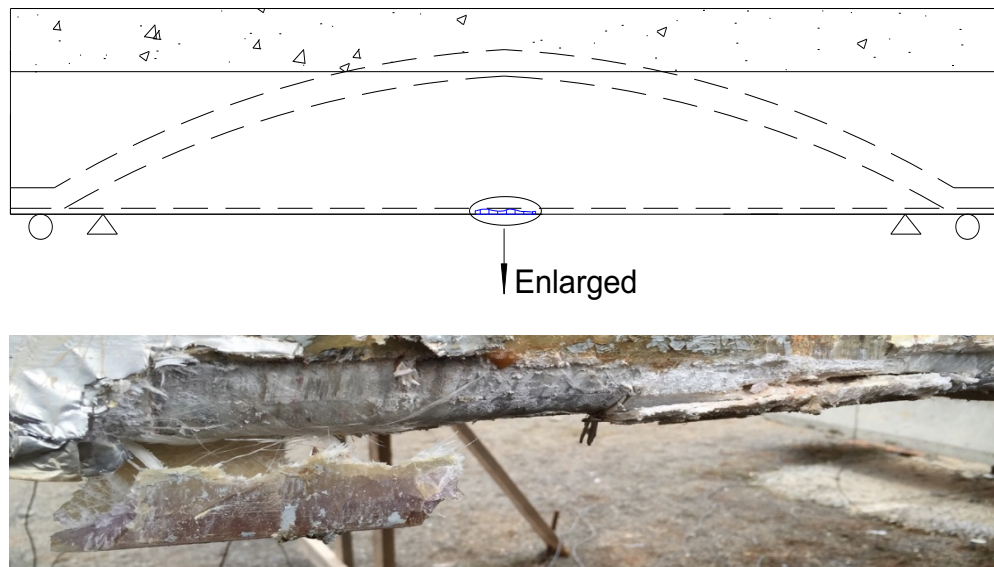


Figure 3.13 Damage at Front Face of the Beam

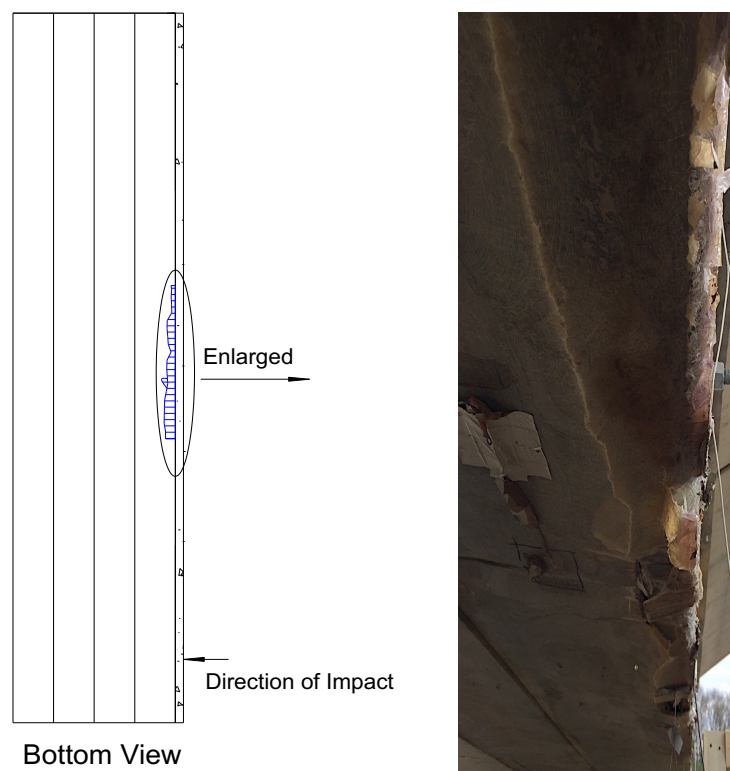


Figure 3.14 Damage at Bottom Face of the Beam

noticed, tearing of FRP laminate and large cracks and delamination could also cause the absorption of moisture, which would then lead to further deterioration. Several repair methods were suggested by Hillman (2012). Vacuum infusion of vinyl ester by drilling a number of holes in the laminate can be adopted to restore the bond between FRP laminas and between FRP shell and interior foam. At most of the damaged areas with obvious cracks, the FRP laminate should be cut off the structure and new FRP strengthening patches should be applied to these areas.

The distance between the specimen and the backstop HSS steel tubes after impact was about 1.59 cm (0.63 in.), as shown in **Fig. 3.15**.



Figure 3.15 Distance between Specimen and Steel Tube after Impact

3.2.6 Testing Results and Discussion

3.2.6.1 Acceleration

Fig. 3.16 shows the filtered acceleration time history from accelerometers. Some corrections were applied to the raw data obtained from the accelerometers. The accelerometers were oscillating about a non-zero acceleration prior to impact, and the average acceleration before impact was subtracted from each accelerometer channel. The acceleration data was then filtered using a built-in filter in a commercial finite element software, Abaqus. The filter used is a low-pass, second-order, Butterworth filter with a cutoff frequency set to one-sixth of the sampling rate (Abaqus 6.14). The noise induced high spikes were filtered out to give a more reasonable response of the structure. Accelerometers A1 and A3 are attached to the front face of the specimen around the quarter spans. Accelerometer A4 is at the back face of the HCB bridge corresponding to the impact zone (**Fig. 3.11**). The initial rise of the accelerations starts at approximately $t=0.022$ s.

3.2.6.2 Horizontal and Vertical Displacements

Displacement time history data obtained from string potentiometers is illustrated in **Fig. 3.17**. The initial rise of the displacement begins at approximately $t=0.025$ s, which is a 0.003 s time lag from the initial rise of the acceleration of the beam. Before conducting the impact test, all the sensors were calibrated through a simple free vibration test in our lab and this time lag is considered as the system error.

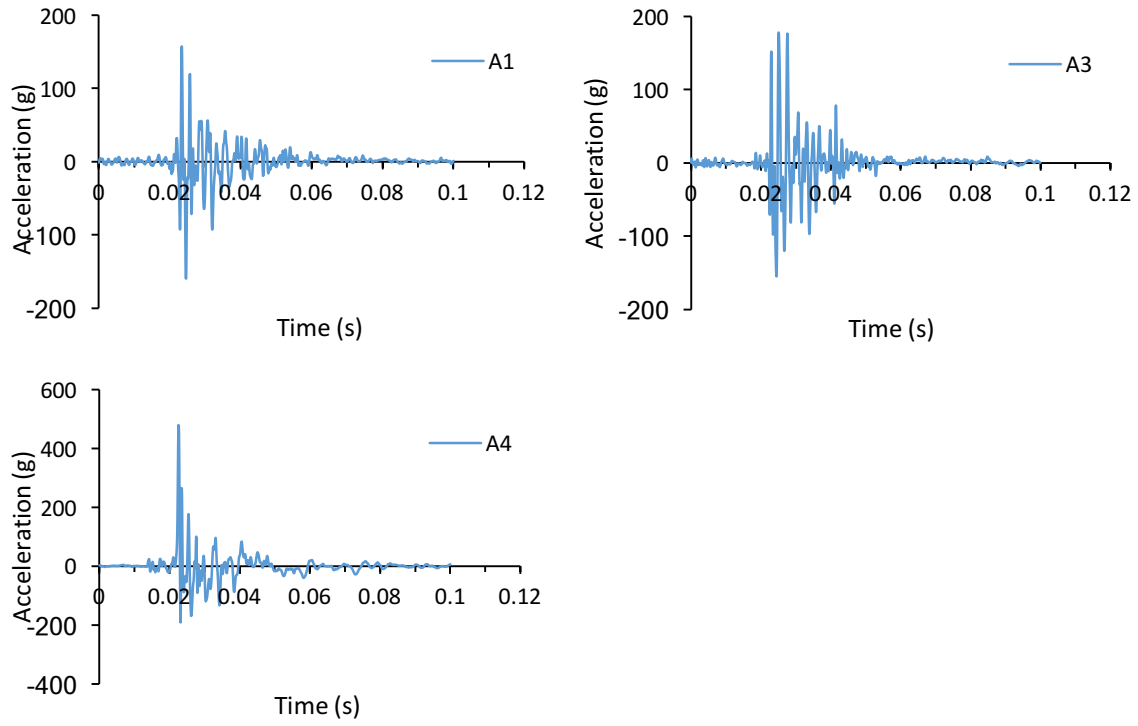


Figure 3.16 Filtered Acceleration Time History; $1\text{ g} = 9.81\text{ m/s}^2 = 386.4\text{ in./s}^2$

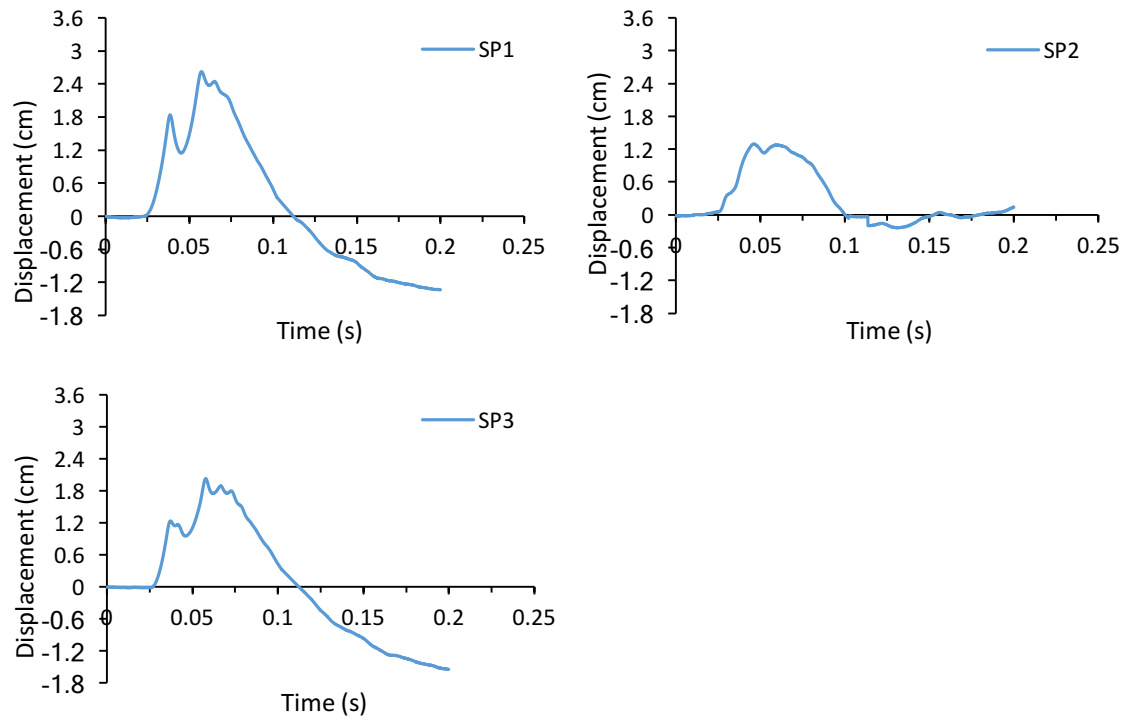


Figure 3.17 Displacement Time History; 1 cm =0.39 inch

Data from SP1 and SP3 are the horizontal displacements at midspan and quarter span of the beam. The two horizontal displacements are almost the same. Taking the midspan horizontal displacement as an example, the displacement starts to increase linearly from $t=0.025$ s, at $t=0.038$ s, the displacement begins to decrease slightly till $t=0.05$ s. The displacement then goes up again in its elastic range, and at $t=0.06$ s, reaches the peak value of 2.62 cm (1.03 in.). At $t=0.0125$ s, the displacement returns to zero. A displacement of about -1.52 cm (-0.60 in.) is observed at $t=0.2$ s due to the overall movement of the specimen, which matches the distance between HCB bridge and steel tube after impact (Fig. 3.15).

Data from SP2 is the vertical displacement at midspan of the HCB bridge. The vertical displacement increases linearly from $t=0.025$ s, reaches the maximum value of 1.29 cm (0.51 in.) at about $t=0.05$ s and then starts to decrease, going to zero at $t=0.1$ s. There are small fluctuation of the data resulting from the vibration during the collision process.

As shown in Fig. 3.17, the maximum displacements for SP1 and SP3 occur almost at the same time. Fig. 3.18 presents the maximum horizontal displacements at support, quarter-span and midspan of the HCB bridge during impact, which indicates horizontal displacements are not linearly distributed along the bridge span.

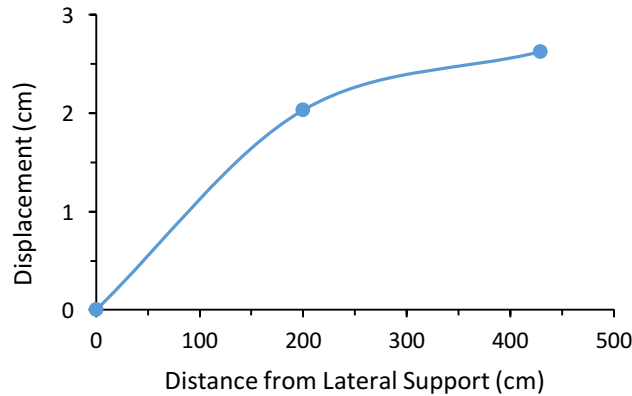


Figure 3.18 Maximum Horizontal Displacement Distribution along Span; 1 cm = 0.39 inch

3.2.6.3 Reaction Force

Fig. 3.19 shows the strain time history obtained from strain gages SG17-SG20, in which tension is positive and compression is negative. Strains from these gages were used to calculate reaction forces. Gages SG17-SG18 were on the top and bottom face of the left HSS steel tube respectively, and gages SG19-SG20 were on the right HSS steel tube. The reason why the strains from top gages are much smaller than those from bottom gages can be explained by the lateral boundary condition of HCB bridge in **Fig. 3.20**. The HCB bridge was put against the backstop in the testing setup. Due to the tie rods at the ends of the specimen, the HCB bridge couldn't fully contact with the backstop and thin steel plates were inserted, but still there existed a gap between the specimen and the steel tube, as shown in the circle in **Fig. 3.20**. Upon impact, the bottom of the HCB bridge was pushed forward, and the bottom part of HSS (arrow in **Fig. 3.20**) acted as almost the only area to transfer forces to backstop.

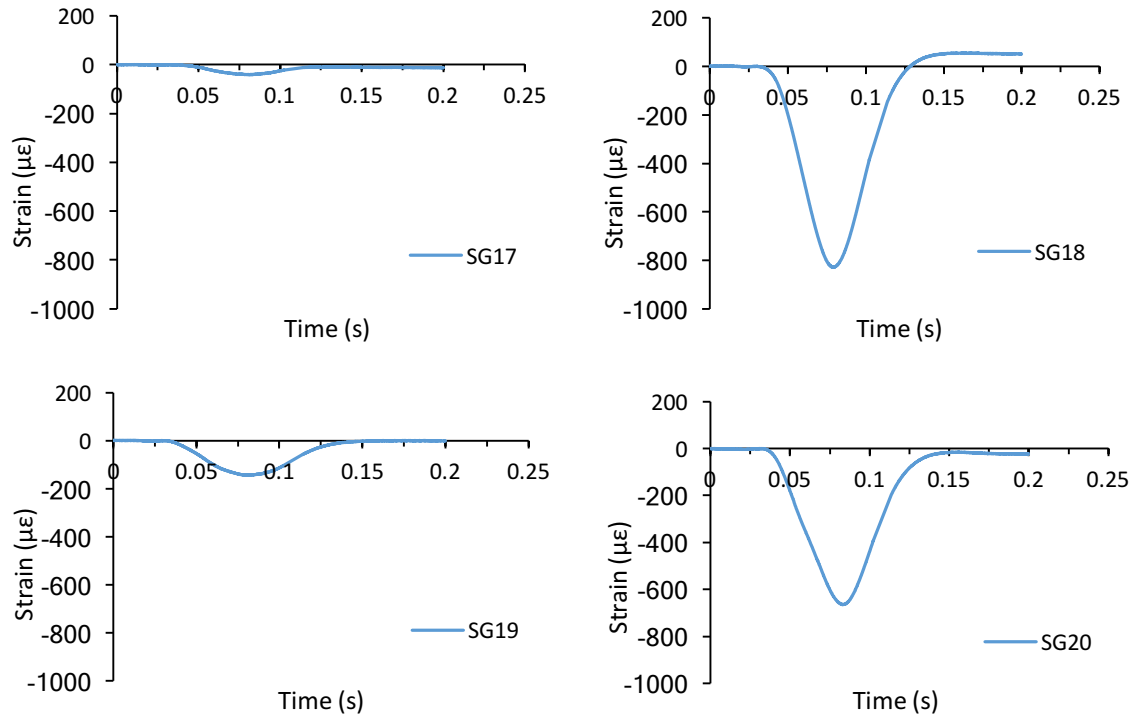


Figure 3.19 Backstop Strain Time History



Figure 3.20 Lateral Boundary Condition of HCB Bridge

The dimension of steel tube is 20.3 cm x 20.3 cm x 1.27 cm (8 in. x 8 in. x 0.5 in.), the area used to calculate the compression force is $A = 20.3 \text{ cm} \times 1.27 \text{ cm} = 25.8 \text{ cm}^2$ (4.0 in.²). The maximum compression forces from gages SG17-SG20 are 21.9 kN (4.92 kips), 427 kN (96.1 kips), 73.8 kN (16.6 kips), and 343 kN (77 kips) respectively. Therefore, the compression forces from top strain gages can be neglected compared to the forces from bottom strain gages. The reaction force was calculated as the sum of compression forces from two bottom strain gages, as shown in **Fig. 3.21**.

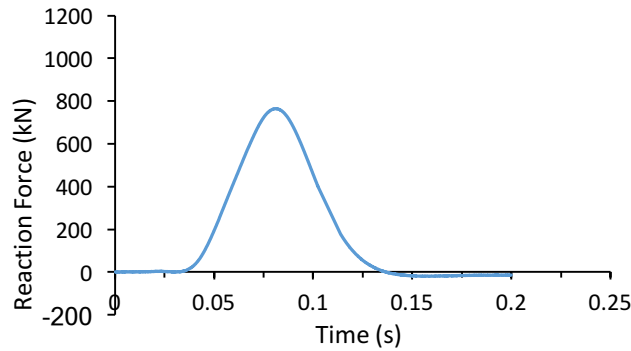


Figure 3.21 Reaction Force Time History; 1 kN = 0.225 kip

3.2.6.4 Strain of HCB Bridge

Strain time history recorded by strain gages on the HCB bridge is illustrated in **Fig. 3.22**, in which tension is positive and compression is negative. Gages SG1 and SG5 were bonded to the concrete arch on the front face of the HCB bridge. Strains from these two gages were generally in compression during impact. Gages SG8 and SG9 were attached to the concrete arch on the back face of the specimen, and they were in tension. Gages SG2, SG3,

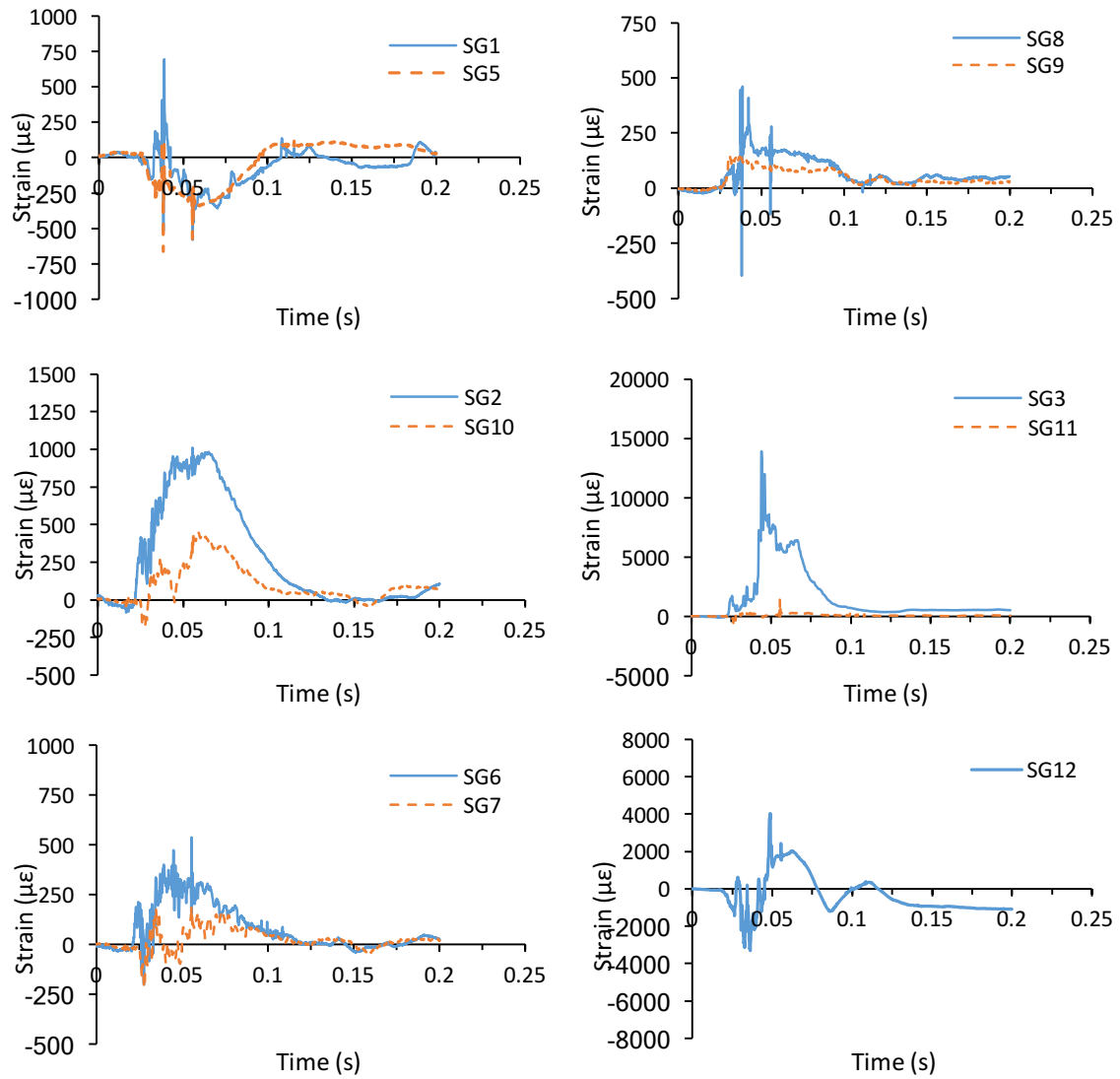


Figure 3.22 HCB Bridge Strain Time History

and SG6 were all attached to the FRP shell on the front face of the specimen. However, they were all under tension during the impact process. Gages SG10, SG11, and SG7 were at the corresponding same locations while on the back face of the specimen. Strains at these three locations were tensile with the values smaller than that of the strain gages SG2, SG3, and SG6. Gage SG12 was on the FRP shell on the bottom face of the HCB bridge (**Fig. 3.11**). The strains in the FRP shell during impact were generally in tension. The maximum recorded compressive stress in the concrete arch was 20.1 MPa (2.92 ksi), which is 48.6% of the compressive strength of the SCC. The FRP shell is in tension in general during the impact and the maximum recorded tensile stress is 297 MPa (43.1 ksi). The FRP away from the impact zone exhibits linear elastic behavior and no tensile rupture or break of FRP fabric was observed on the beam after testing.

3.2.6.5 Bending Energy

For the dynamic problems, the reaction force is typically not equal to the applied force. Some portion of the applied external force is used to overcome the structure's inertia, or to accelerate the structure. The initial velocity, v_o , of the impact cart was determined to be 24 km/h (15 mph) from high speed video analysis of a marked wood frame that was placed along the direction of impact near the impact cart. The mass of the impact cart, m , was measured as 4080 kg (9000 lb.). The initial impact energy $E_i = mv_o^2/2$ is equal to 96 kJ (71 kip-ft). **Fig. 3.23** shows the load displacement curve of the HCB bridge during impact.

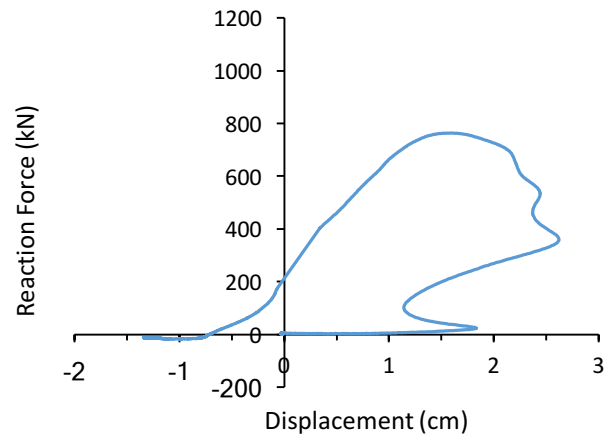


Figure 3.23 Load Displacement Curve; 1 kN =0.225 kip, 1 cm =0.39 inch

The bending energy E_b is the area under this curve. $E_b=12.5$ kJ (9.23 kip-ft), which indicates that about one eighth of the total energy was used to make the bridge bending. The rest of the energy was used to overcome the inertia of the HCB bridge, absorbed through strain energy of the tension reinforcement and the low-density foam, dissipated by local failure of the FRP shell, and balanced by kinetic energy of the impact cart rebounding.

3.2.6.6 Local deformation

High speed video with a speed of 240 frame per second was taken during the HCB bridge impact test. The video was analyzed by a software Traker in order to obtain the local deformation of the HCB bridge. At the beginning of the analysis, a calibration stick and a coordinate system were defined. The stick distance shown in **Fig. 3.24** is 152 cm (60 in.) as a reference. The position of this calibration stick should be close to the impact cart to get an accurate result. During the analysis, the movement of one back corner of the

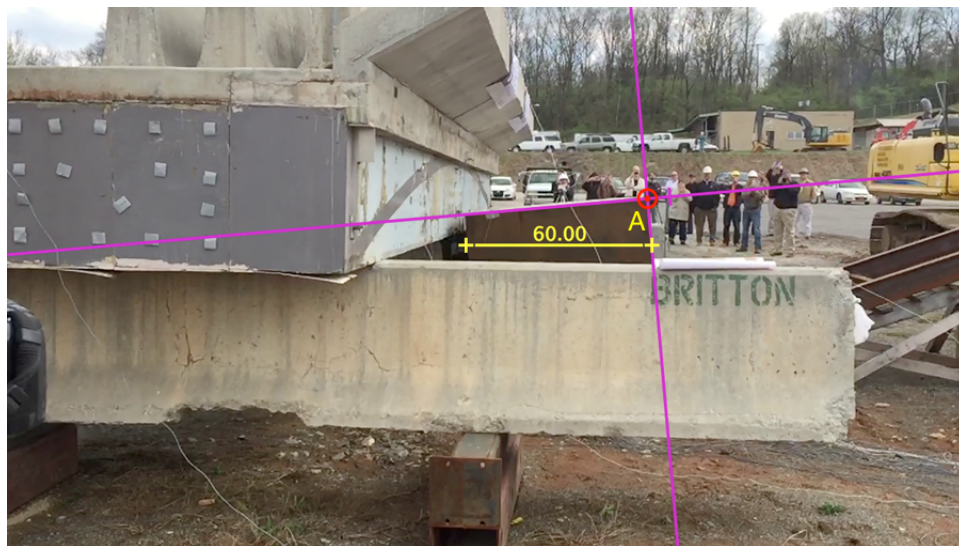


Figure 3.24 Calibration Stick and Coordinate System

impact cart (point A) can be obtained. The origin of the coordinate system was set at point A before the contact between the impact cart and the HCB bridge (**Fig. 3.24**). The red points in **Fig. 3.25** are the footprints of the point A from $t=0$ s to $t=0.0625$ s, at which the impact cart reaches the maximum displacement.

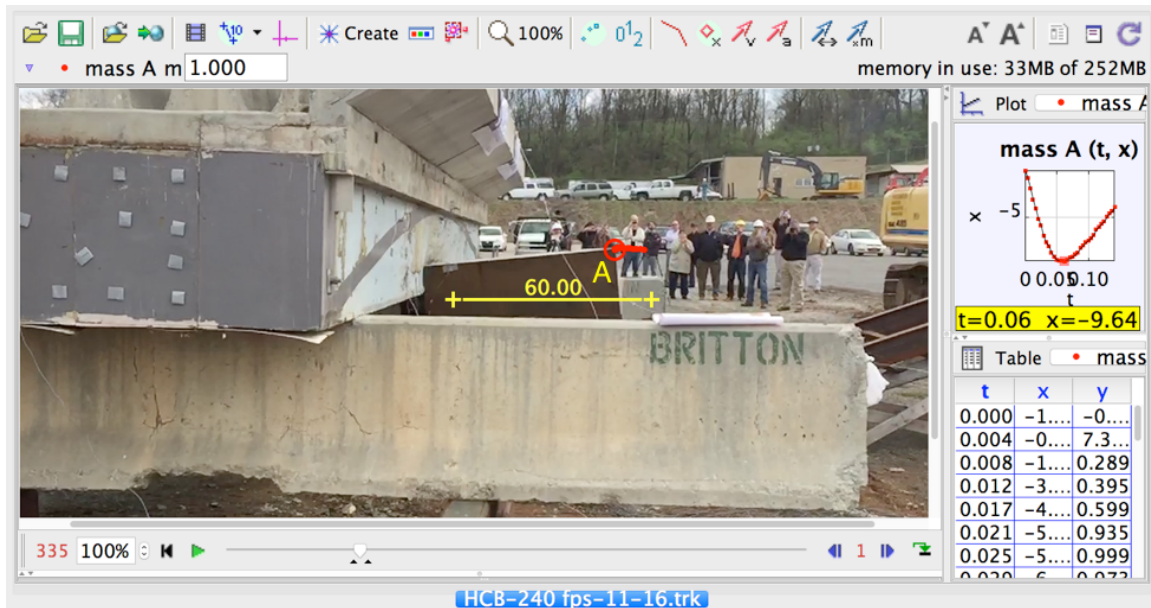


Figure 3.25 Footprints of Point A

Fig. 3.26 illustrates the displacement of point A. The data was manipulated so that the moment of the zero displacement in the curve represents the contact between the impact and the HCB bridge. The maximum displacement of the impact cart during impact was determined as 9.60 cm (3.78 in.). **Fig. 3.27** shows the displacement of the impact cart from video analysis and the horizontal displacement of the HCB bridge midspan from testing data. The local deformation of the HCB bridge is equal to the difference between these

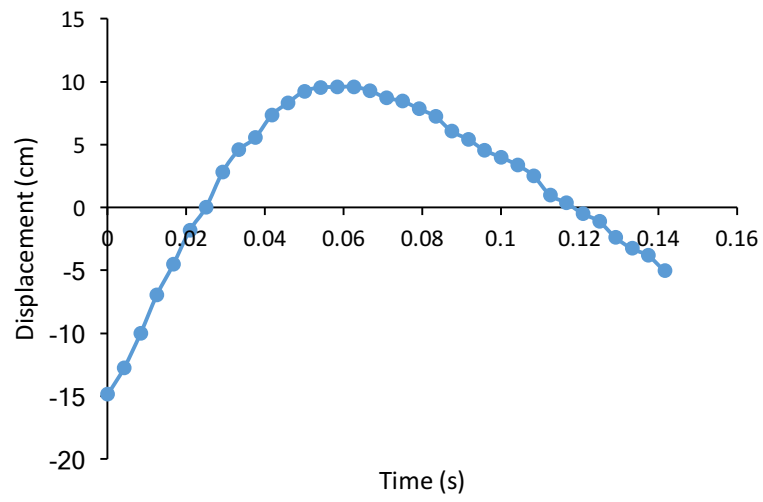


Figure 3.26 Displacement of Point A; 1 cm=0.39 inch

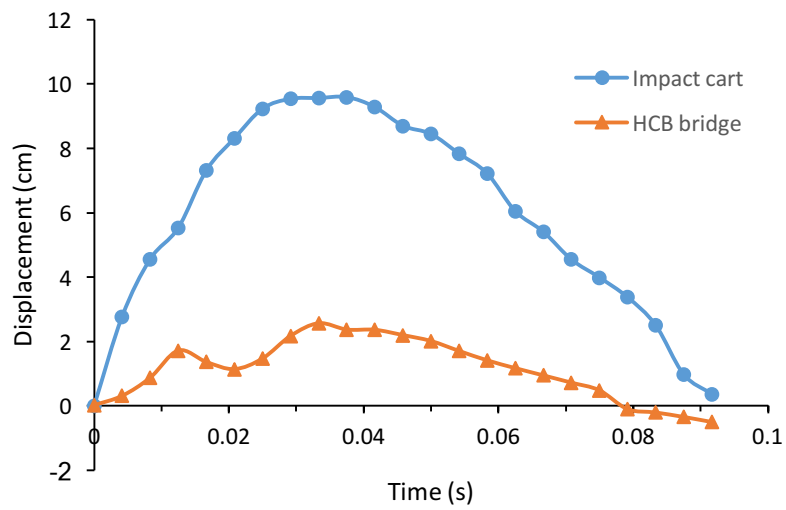


Figure 3.27 Displacements of the Impact Cart and the HCB Bridge; 1 cm=0.39 inch

two displacements and the indentation curve is presented in **Fig. 3.28**. The maximum local deformation of the HCB bridge during impact is 7.76 cm (3.05 in.). This large local deformation reserved a lot of elastic strain energy during impact and acted as a buffer zone under impact loading.

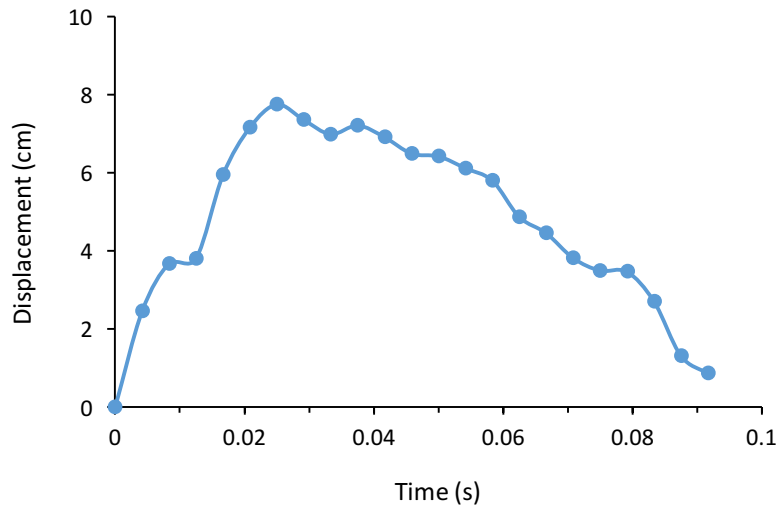


Figure 3.28 Indentation of the HCB Bridge during Impact; 1 cm=0.39 inch

HCB consists of four different materials, and the configuration of each material makes the cross sections along the beam span differ from each other. Under lateral impact, the main part of the HCB bridge to resist the impact force is the Hardwire tension reinforcement. The thickness of the tension reinforcement is 1.14 cm (0.45 in.). The stiffness of the tension reinforcement under lateral load can be calculated from the following equation when it is simply supported:

$$k = 48EI/L^3 \quad (3.1)$$

Where E is the Modulus of Elasticity of the Hardwire, $E=82254 \text{ MPa}$ (11930 ksi);

I is the Moment of Inertia around vertical axis, $I=799164 \text{ cm}^4$ (19200 in.⁴);

L is the span length between two lateral supports, $L=9.1 \text{ m}$ (360 in.).

The stiffness of the tension reinforcement under lateral load was determined as 41330 kN/m (236 kip/in.). The maximum midspan horizontal displacement of the HCB bridge is 2.62 cm (1.03 in.). Therefore, the strain energy due to the tension reinforcement bending can be calculated as $E=k\delta^2/2=14.1 \text{ kJ}$ (10.4 kip-ft). The bending energy from the reaction force-displacement in section 3.2.6.5 equals to 12.5 kJ (9.23 kip-ft). The difference between them is 10.7%, which is acceptable. Therefore, the HCB can be simplified as the tension reinforcement to analyze the beam's flexural behavior when subjected to lateral impact loading.

3.3 Comparison Between PC Girder and HCB Bridge under Lateral Impact Loading

Although the weight of the HCB bridge is about twice of the PC girder, the behavior of the two specimens are compared qualitatively here. Widely different durations and peak impact forces can be generated under the same impact energy (Skov and Olesen, 1975). High stiffness results in a short duration with a high peak value. The impact cart is a concrete block wrapped by steel plates, and the concrete compressive strength of PC girder was 97 MPa (14100 psi), which led to the high stiffness in the impact zone.

Therefore, the failure of the girder was induced by punching shear force and crushing of concrete could be obviously observed. For the HCB bridge, in the impact zone, the thickness of the tension reinforcement is 1.14 cm (0.45 in.), and the majority of the material is low-density foam under the FRP shell, as shown in **Fig. 3.29**. A lot of energy was absorbed by the local deformation of the HCB bridge. Therefore, the resilient nature of the materials around impact zone makes HCB an effective structure to resist lateral impact loading.

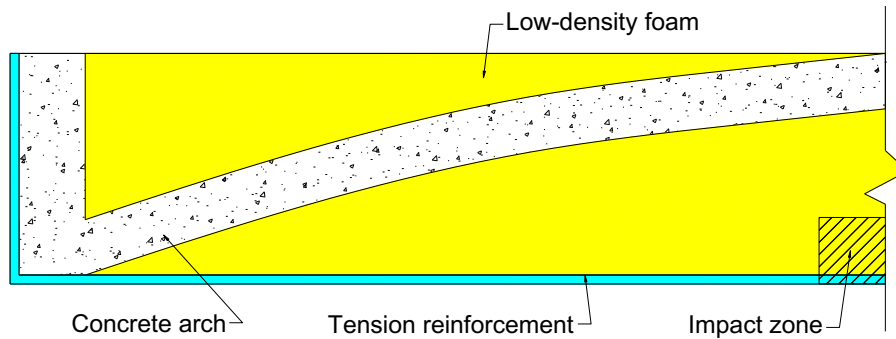


Figure 3.29 Internal Configuration of HCB

4. FINITE ELEMENT SIMULATION OF PC GIRDER UNDER IMPACT LOADING

4.1 FE Simulation of PC Girder Impact Process

4.1.1 Concrete Damaged Plasticity

Concrete damaged plasticity (CDP) model provided by Abaqus was used to model the PC girder in this study. This model has the capability to analyze concrete structures under dynamic loading and is suitable to simulate the brittle behavior of concrete when under low confining pressure. CDP model combines isotropic elasticity with isotropic tensile and compressive plasticity to represent the inelastic behavior of concrete, which allows for the concrete crushing under compression and cracking under tension. The elastic-plastic behavior for concrete of CDP model is summarized as follows:

$$\bar{\sigma} = D_0^{el} : (\varepsilon - \varepsilon^{pl}) \quad (4.1)$$

$$\dot{\bar{\varepsilon}}^{pl} = h(\bar{\sigma}, \bar{\varepsilon}^{pl}) \cdot \dot{\varepsilon}^{pl} \quad (4.2)$$

$$\dot{\varepsilon}^{pl} = \dot{\lambda} \frac{\partial G(\bar{\sigma})}{\partial \bar{\sigma}} \quad (4.3)$$

$$\sigma = (1 - d) \bar{\sigma} \quad (4.4)$$

Eq. (4.1) defines the relationship between the effective stress and the elastic strain. The evolution of hardening variables and plastic flow are described by Eqs. (4.2) and (4.3). The Cauchy stress is calculated in terms of the effective stress multiplied by the stiffness degradation variable in Eq. (4.4).

Yield surface of the CDP model is governed by the following equations:

$$F = \frac{1}{1-\alpha} (\bar{q} - 3\alpha\bar{p} + \beta(\bar{\varepsilon}^{pl})\langle\hat{\sigma}_{max}\rangle - \gamma\langle-\hat{\sigma}_{max}\rangle) - \bar{\sigma}_c(\bar{\varepsilon}_c^{pl}) = 0 \quad (4.5)$$

$$\alpha = \frac{\left(\frac{\sigma_{b0}}{\sigma_{c0}}\right)^{-1}}{2\left(\frac{\sigma_{b0}}{\sigma_{c0}}\right)^{-1}-1}; \quad 0 \leq \alpha \leq 0.5 \quad (4.6)$$

$$\beta = \frac{\bar{\sigma}_c(\bar{\varepsilon}_c^{pl})}{\bar{\sigma}_t(\bar{\varepsilon}_t^{pl})} (1 - \alpha) - (1 + \alpha) \quad (4.7)$$

$$\gamma = \frac{3(1-K_c)}{2K_c-1} \quad (4.8)$$

Where $\bar{p}$ is the hydrostatic pressure stress;

$\bar{q}$ is the Mises equivalent effective stress;

$\hat{\sigma}_{max}$ is the maximum principal effective stress;

σ_{b0}/σ_{c0} is the ratio of initial equibiaxial to uniaxial compressive yield stress;

K_c is the ratio of the second stress invariant on the tensile meridian to that on the compressive meridian

Fig. 4.1 shows the yield surfaces in the deviatoric plane.

The nonassociated potential plastic flow is assumed for the CDP model, and the flow potential G is from Drucker-Prager hyperbolic function:

$$G = \sqrt{(\varepsilon\sigma_{t0}\tan\psi)^2 + \bar{q}^2} - \bar{p}\tan\psi \quad (4.9)$$

When defining concrete plasticity, the parameters include dilation angle ψ , flow potential

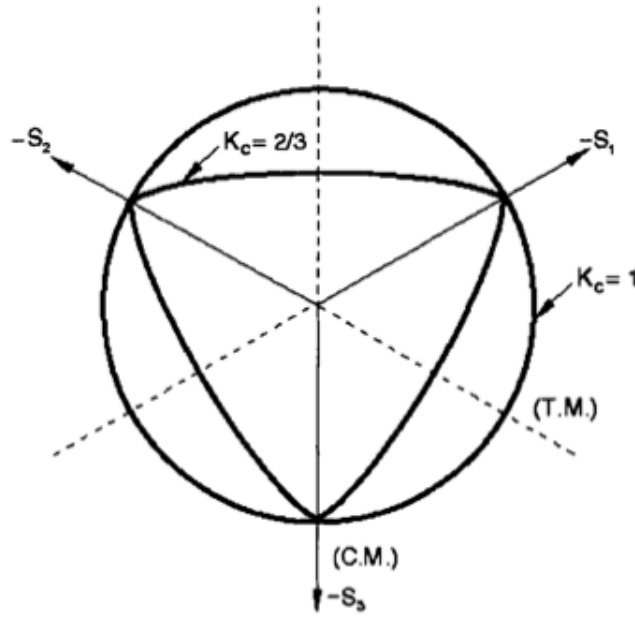


Figure 4.1 Typical Yield Surfaces on the Deviatoric Plane (Abaqus, 6.14)

eccentricity ϵ , the ratio of initial equibiaxial to uniaxial compressive yield stress σ_{b0}/σ_{c0} , the ratio of the second stress invariant on the tensile meridian to compressive meridian K_c , and viscosity parameter. The values of them were selected as 36° , 0.1, 1.16, 0.67, and 0.0001, respectively.

The mechanical behavior of concrete under uniaxial compressive and tensile loading used in the CDP model are presented in **Fig. 4.2** and **Fig. 4.3**, respectively. As shown in the **Figs. 4.2** and **4.3**, when the concrete is unloaded from any point on the strain softening part of the stress-strain curve, the elastic stiffness is degraded.

Two damage variables d_c and d_t are used to characterize the stiffness degradation. The

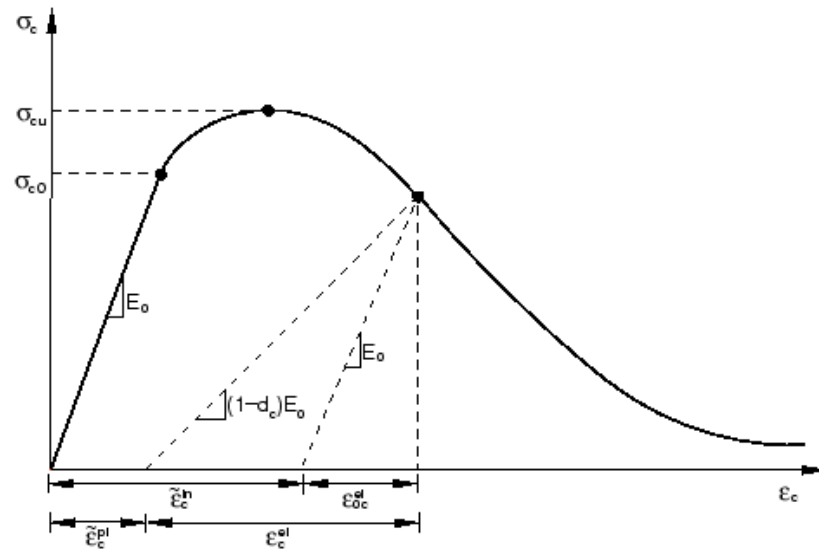


Figure 4.2 Concrete Uniaxial Response in Compression (Abaqus, 6.14)

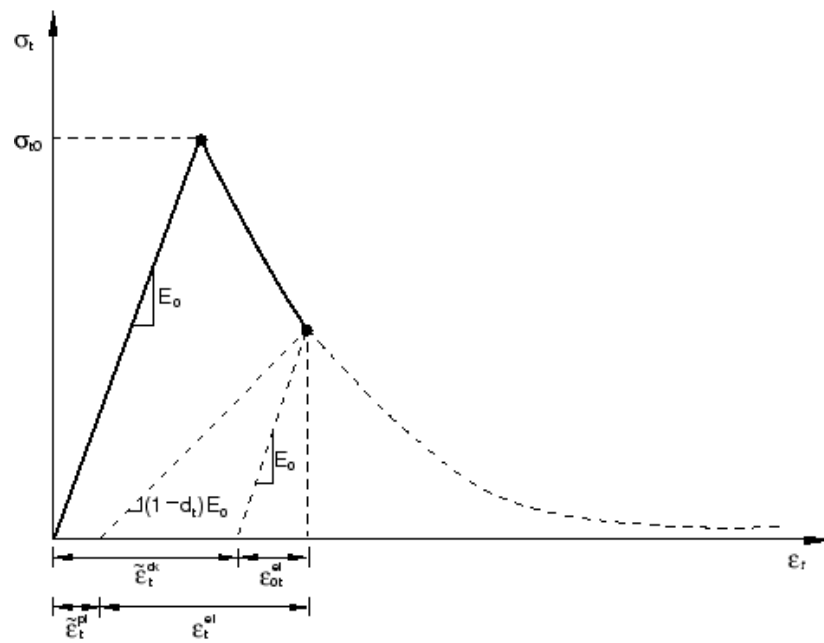


Figure 4.3 Concrete Uniaxial Response in Tension (Abaqus, 6.14)

damage variable changes from 0 to 1. The value zero indicates undamaged material and one represents total loss of strength. E_0 is the undamaged elastic stiffness, $\tilde{\varepsilon}_c^{pl}$ and $\tilde{\varepsilon}_t^{pl}$ are the equivalent plastic strains. The stresses can be expressed in Eq. (4.10) and Eq. (4.11), respectively.

$$\sigma_c = (1 - d_c)E_0(\varepsilon_c - \tilde{\varepsilon}_c^{pl}) \quad (4.10)$$

$$\sigma_t = (1 - d_t)E_0(\varepsilon_t - \tilde{\varepsilon}_t^{pl}) \quad (4.11)$$

In Abaqus, the stresses of concrete in compression and tension after undamaged elastic range are given as a function of inelastic strain and cracking strain, respectively. As shown in **Figs. 4.2** and **4.3**, $\tilde{\varepsilon}_c^{in}$ is the inelastic strain and $\tilde{\varepsilon}_t^{ck}$ is the cracking strain, which can be calculated from Eq. (4.12) and Eq. (4.13).

$$\tilde{\varepsilon}_c^{in} = \varepsilon_c - \varepsilon_{0c}^{el} \quad (4.12)$$

$$\tilde{\varepsilon}_t^{ck} = \varepsilon_t - \varepsilon_{0t}^{el} \quad (4.13)$$

where $\varepsilon_{0c}^{el} = \sigma_c/E_0$, $\varepsilon_{0t}^{el} = \sigma_t/E_0$

Abaqus automatically converts the inelastic strain and cracking strain values to plastic strains by using:

$$\tilde{\varepsilon}_c^{pl} = \tilde{\varepsilon}_c^{in} - \frac{d_c}{(1-d_c)} \frac{\sigma_c}{E_0} \quad (4.14)$$

$$\tilde{\varepsilon}_t^{pl} = \tilde{\varepsilon}_t^{ck} - \frac{d_t}{(1-d_t)} \frac{\sigma_t}{E_0} \quad (4.15)$$

If the compressive and tensile damage variables are not specified, $\tilde{\varepsilon}_c^{pl} = \tilde{\varepsilon}_c^{in}$, and $\tilde{\varepsilon}_t^{pl} = \tilde{\varepsilon}_t^{ck}$, and the model behaves as a plasticity model. In this study, the stiffness degradation due to tension d_t was assumed to be zero. The calculation of stiffness degradation damage variable d_c was determined as follows (Birtel and Mark, 2006):

$$d_c = 1 - \frac{\sigma_c E_c^{-1}}{\varepsilon_c^{pl}(1/b_c - 1) + \sigma_c E_c^{-1}} \quad (4.16)$$

$$\varepsilon_c^{pl} = b_c(\varepsilon_c - \sigma_c E_c^{-1}) \quad (4.17)$$

Where b_c is a constant factor with the value of 0.7.

4.1.2 Material Constitutive Relation for Concrete and Prestressing Strand

The concrete compressive strength of PC girder was 97 MPa (14100 psi). In this study, the stress strain relationship for high strength concrete in compression was obtained from equations proposed by Wee et al. (1996). The ascending branch of the stress-strain curve was determined by the following equations from Carreira and Chu (1985) with the SI unit system:

$$f_c = f'_c \left[\frac{\beta \left(\frac{\varepsilon}{\varepsilon_o} \right)}{\beta - 1 + \left(\frac{\varepsilon}{\varepsilon_o} \right)^\beta} \right] \quad (4.18)$$

$$\beta = \frac{1}{1 - (f'_c / \varepsilon_o E_{it})} \quad (4.19)$$

$$\varepsilon_o = 0.000078(f'_c)^{(1/4)} \quad (4.20)$$

$$E_{it} = 10,200(f'_c)^{(1/3)} \quad (4.21)$$

The descending part was calculated by using Eqs. (4.22) -(4.24) with SI unit proposed by

Wee et al. (1996):

$$f_c = f'_c \left[\frac{k_1 \beta \left(\frac{\varepsilon}{\varepsilon_0} \right)}{k_1 \beta - 1 + \left(\frac{\varepsilon}{\varepsilon_0} \right)^{k_2 \beta}} \right] \quad (4.22)$$

$$k_1 = \left(\frac{50}{f'_c} \right)^{3.0} \quad (4.23)$$

$$k_2 = \left(\frac{50}{f'_c} \right)^{1.3} \quad (4.24)$$

Where f_c = concrete stress;

ε = concrete strain;

β = a material parameter;

f'_c = compressive strength of concrete;

ε_0 = the strain at peak stress;

E_{it} = the initial tangent modulus;

k_1, k_2 = correction factors.

The tensile strength was estimated to be 10% of the ultimate compressive strength, and the cracking strain was calculated accordingly as of 207 $\mu\varepsilon$. After cracking, the stress-strain curve was assumed to decrease exponentially, and the equation used was from Jiang and Lu (2013) as follow:

$$\sigma_t = f_t e^{-\alpha(\varepsilon - \varepsilon_{ck})} \quad (4.25)$$

Where f_t is concrete tensile strength;

ε_{ck} is cracking strain;

α is a softening parameter.

Fig. 4.4 and **Fig. 4.5** show the stress-strain curves of the PC girder used in this study under compression and tension, respectively.

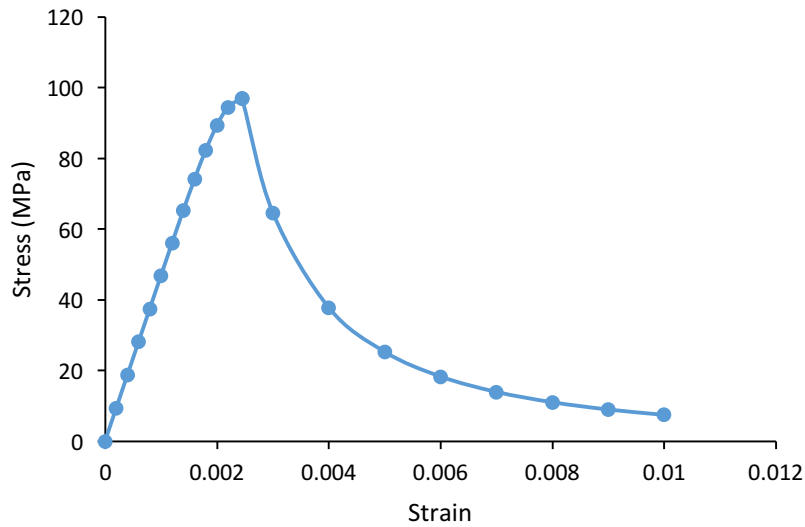


Figure 4.4 Stress-Strain Curve of High Strength Concrete in Compression; 1 MPa=145 psi

Table 4.1 shows the modulus of elasticity and Poisson’s ratio for different parts of the model. The effect of concrete plasticity was not considered in the model of the concrete deck and the impact cart. **Fig. 4.6** illustrates the constitutive relation of prestressing strand (Cabage, 2014).

4.1.3 Strain Rate Effect

Concrete is a rate sensitive material. The mechanical behavior of concrete under dynamic loading is different from its static behavior. The strain rate effect in this study was considered based on the suggestions of CEB-FIP MC 90 (CEB-FIP 1990). The strain rate was

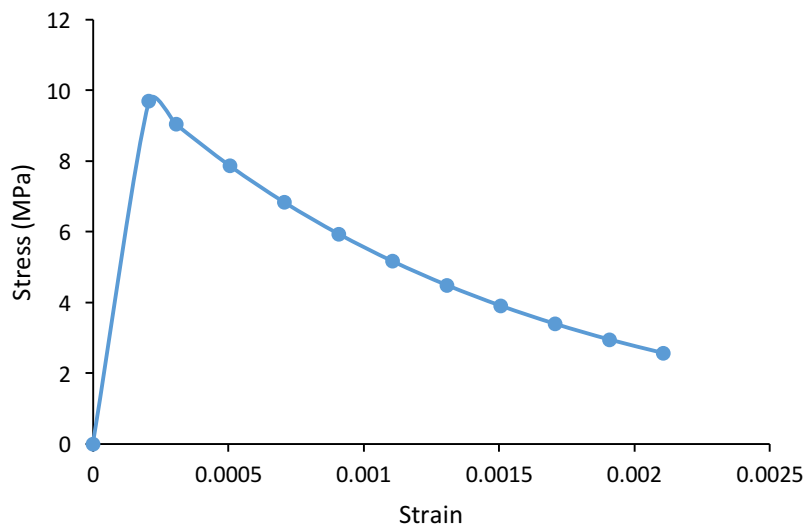


Figure 4.5 Stress-Strain Curve of High Strength Concrete in Tension; 1 MPa=145 psi

Table 4.1 Elastic Material Properties

	Modulus of Elasticity (MPa)	Poisson's Ratio
Concrete Girder	46864	0.2
Concrete Deck	40520	0.2
Prestressing Strand	199948	0.3
Impact Cart Concrete	27793	0.2
Impact Cart Steel	199948	0.3

Note: 1 MPa=145 psi.

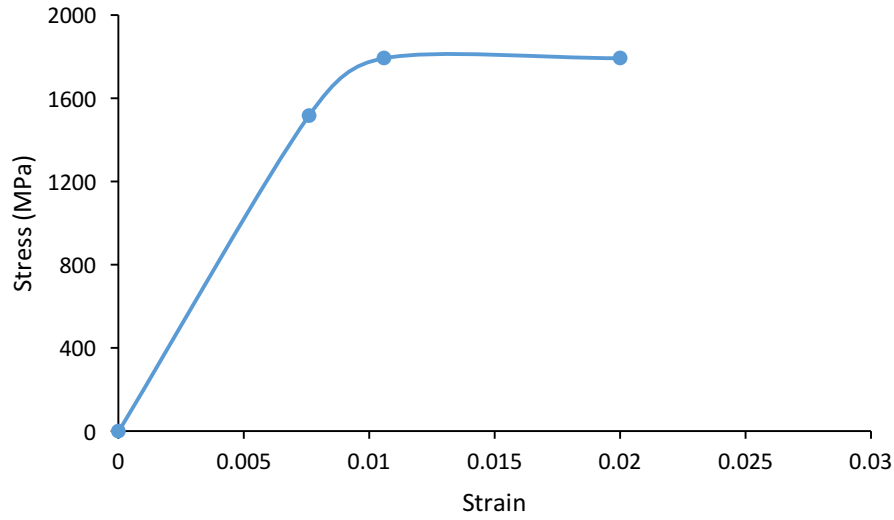


Figure 4.6 Stress-Strain Curve for Prestressing Strand; 1 MPa=145 psi

obtained according to the strain data from strain gage SG1 (**Fig. 3.3**) located above the impact zone, and the strain increased to 3000 $\mu\epsilon$ during 0.04 s upon impact. The strain rate was determined as 0.075 s^{-1} . The compressive strength of concrete under a given strain rate is calculated as follows:

$$f_{c,imp}/f_c = (\dot{\epsilon}_c/\dot{\epsilon}_{c0})^{1.026\alpha_s} \quad \dot{\epsilon}_c \leq 30 \text{ s}^{-1} \quad (4.26)$$

$$f_{c,imp}/f_c = \gamma_s (\dot{\epsilon}_c/\dot{\epsilon}_{c0})^{1/3} \quad \dot{\epsilon}_c > 30 \text{ s}^{-1} \quad (4.27)$$

$$\log \gamma_s = 6.156\alpha_s - 2 \quad (4.28)$$

$$\alpha_s = \frac{1}{5 + 9f_c/f_{c0}} \quad (4.29)$$

Where $\dot{\epsilon}_c$ is the strain rate;

$$\dot{\epsilon}_{c0} = 30 \times 10^{-6} \text{ s}^{-1};$$

$$f_{c0} = 10 \text{ MPa.}$$

The tensile strength of the concrete under dynamic loading is estimated from:

$$f_{ct,imp}/f_{ct} = (\dot{\epsilon}_{ct}/\dot{\epsilon}_{ct0})^{1.016\delta_s} \quad \dot{\epsilon}_{ct} \leq 30s^{-1} \quad (4.30)$$

$$f_{ct,imp}/f_{ct} = \beta_s(\dot{\epsilon}_{ct}/\dot{\epsilon}_{ct0})^{1/3} \quad \dot{\epsilon}_{ct} > 30s^{-1} \quad (4.31)$$

$$\log \beta_s = 7.112\delta_s - 2.33 \quad (4.32)$$

$$\delta_s = \frac{1}{10+6f_c/f_{c0}} \quad (4.33)$$

Where $\dot{\epsilon}_{ct}$ is the strain rate;

$$\dot{\epsilon}_{ct0} = 3 \times 10^{-6} s^{-1}.$$

The effect of strain rate on modulus of elasticity is determined from:

$$E_{c,imp}/E_{ci} = (\dot{\epsilon}_c/\dot{\epsilon}_{c0})^{0.026} \quad (4.34)$$

The effect of strain rate on the strains at maximum stresses under compression and tension can be estimated from:

$$\epsilon_{cl,imp}/\epsilon_{cl} = (\dot{\epsilon}_c/\dot{\epsilon}_{c0})^{0.02} \quad (4.35)$$

By using Eqs. (4.26) to (4.35), the dynamic factors when considering strain rate for concrete compressive strength, tensile strength, modulus of elasticity, strain at the maximum compression stress and strain at the maximum tension stress are 1.09, 1.16, 1.23, 1.17, and 1.22, respectively. After multiplied by these dynamic factors, the concrete material properties were substituted into Eqs. (4.18) to (4.25) to obtain the new stress-

strain curves, and then the new constitutive relation of concrete was applied to the whole PC girder model in this study.

4.1.4 FE Simulation of AASHTO Type-I PC Girder Impact Process

The number of elements of the whole model is 113154, and the total number of nodes is 146834. Linear (first-order), reduced-integration solid elements (C3D8R) were used to model concrete material. First-order elements are suitable for simulations of impact loading, and linear reduced integration elements are very tolerant of distortion (Abaqus 6.11, 2011).

Prestressing strands were modeled using truss elements (T3D2). The truss elements were coupled with solid elements with “EMBEDDED ELEMENTS” function of ABAQUS. With this function, the translational degrees of freedom of the embedded elements (truss) will be constrained by the host elements (solid), which means the slip between strands and concrete is neglected in this analysis. Prestressing force was introduced by applying initial tensile stresses to the strands.

An element size of 5.08 cm (2-in.) was used for the girder, and a refined mesh with 2.54 cm (1-in.) element size was used in the area near the impact zone to avoid heavy distortion of the elements and at the same time to meet the requirement of stability limit of the

analysis. Since the beam was tested statically in the lab before impact testing (Cabage, 2014), the portion of the damaged deck was measured and the real size of the deck was modeled. **Fig. 4.7** shows the FE model of the PC girder. The element size of 5.08 cm (2-in.) was used for impact cart and a refined mesh with 2.54 cm (1-in.) was used for the impactor, as shown in **Fig. 4.8**. The collision process was simulated by applying an initial velocity of 24 km/h (15 mph) to the impact cart. The two contact areas of 20.3 cm (8 in.) x 12.7 cm (5 in.) between girder and steel tubes were partitioned and restrained translationally in the direction of impact during the simulation duration of 0.1 s. General contact interaction was applied between the impact cart and the PC girder. Tangential friction properties with a friction penalty of 0.45, based on the friction between concrete and steel, were selected.

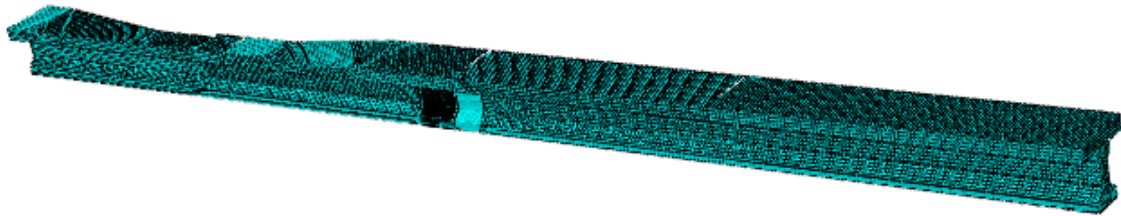


Figure 4.7 Finite Element Model of Prestressed Concrete Girder

4.1.5 FE Results and Data Calibration

The damaged concrete deck due to shear tests was cut off before the impact test. The actual dimension of the remaining concrete deck has been included in the FE model. After the impact test, it was observed that cracks due to impact are not an extension of prior existing minor flexural-shear cracks near the supports of the new impact setup. Due to

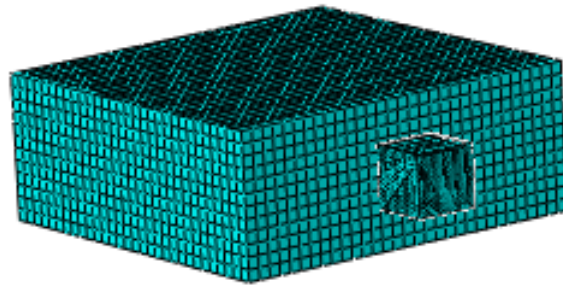


Figure 4.8 Finite Element Model of Impact Cart

the disastrous failure of the girder, most of the sensors failed to collect useful data during the impact testing. The comparisons in the following are based on the data from the string potentiometer SP1 (**Fig. 3.3**) and the pictures from video analysis.

Fig. 4.9 and Fig. 4.10 show the horizontal displacement time history curve at midspan of the girder and the horizontal displacement contour of the enlarged local zone at the girder midspan obtained from FE analysis, respectively. The horizontal displacement at $t=0.1$ s is 16.2 cm (6.38 in.). It can be observed that this displacement is a localized displacement instead of the flexural displacement of the girder (**Fig. 4.10**).

Fig. 4.11 illustrates the comparison of vertical displacement at midspan of the girder (SP1) between experimental results and FE results. The FE results match well with the testing results before $t=0.04$ s. After that, the vertical displacement from FE analysis continues to increase. However, the data from impact testing starts to decrease. During the impact

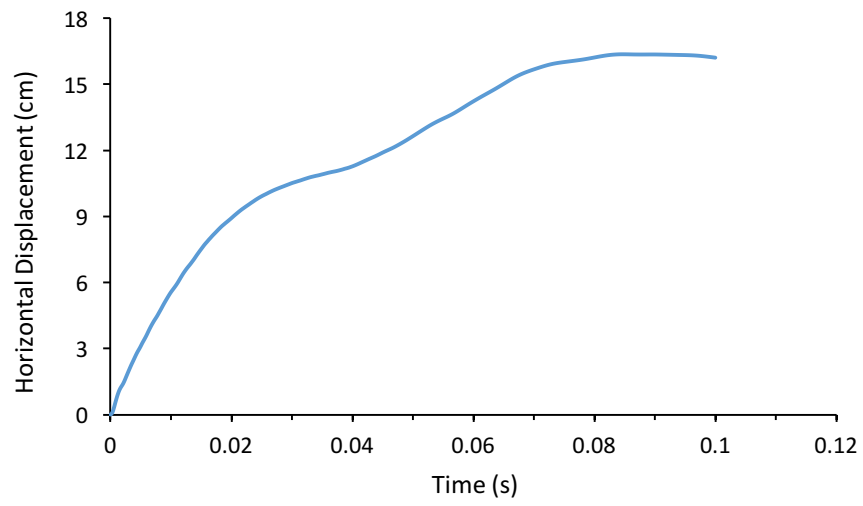


Figure 4.9 Horizontal Displacement at Midspan from FE Analysis; 1 cm=0.39 inch

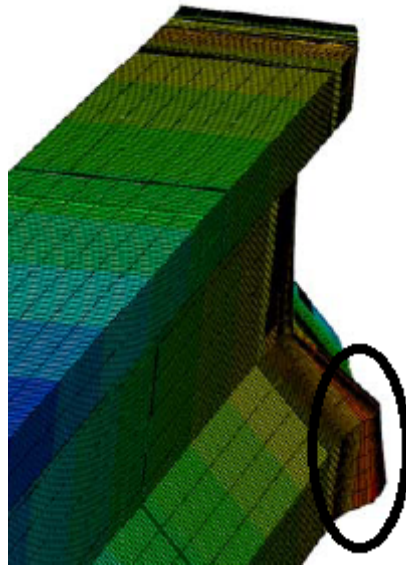


Figure 4.10 Horizontal Displacement Contour at Midspan

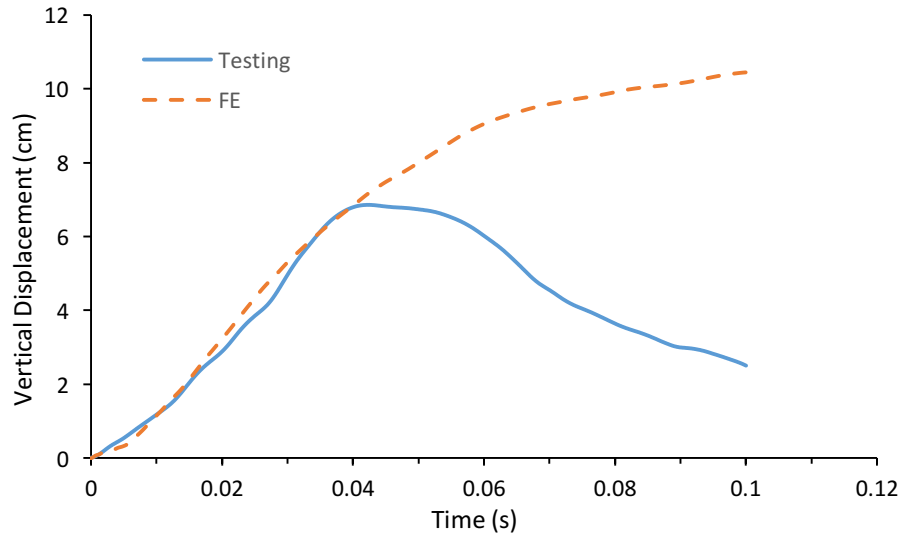


Figure 4.11 Vertical Displacement from SP1 vs. FE Results; 1 cm=0.39 inch

testing, the PC girder was observed moving up with time based on the video. The reason for the decrease of the testing data after $t=0.04$ s is may be due to small concrete blocks fell off during the impact because of concrete crushing and the string potentiometer was connected to one of the blocks.

During the impact test, a video from side view was taken with a speed of 30 frame per second. The movement of the impact cart upon impact till $t=0.033$ s could be obtained from video analysis. The horizontal displacement of the impact cart at $t=0.033$ s from video analysis was determined as 9.7 cm (3.80 in.). The displacement of the impact cart obtained from FE analysis is 9.8 cm (3.84 in.), which is essentially the same.

Fig. 4.12 is a damage contour of the PC girder at $t=0.1$ s. The ultimate strain $\epsilon_u=0.01$ was

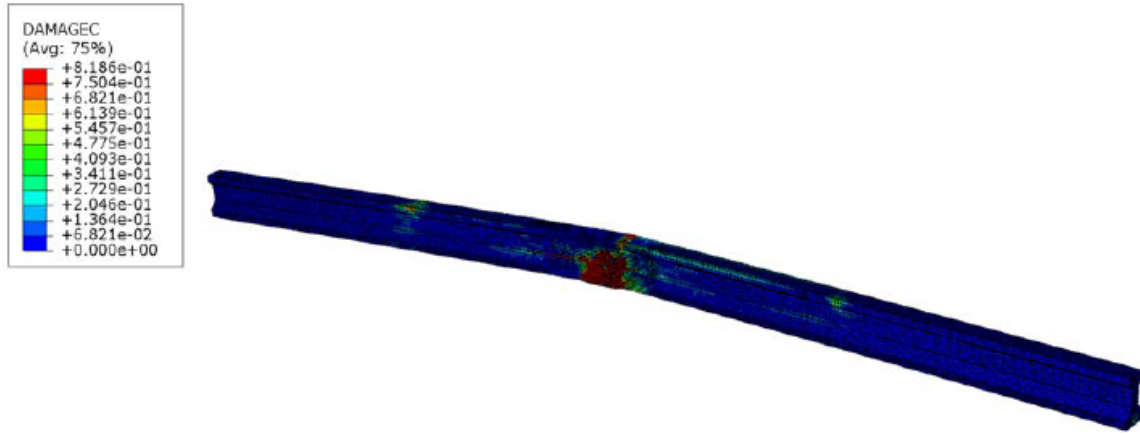


Figure 4.12 Damage Contour of the PC Girder at $t=0.1$ s

defined for the compressive strain-stress curve in this study. Based on Eqs. (4.16) and (4.17), the stiffness degradation damage variable d_c corresponding to $\varepsilon_u=0.01$ was determined as 0.82. As shown in **Fig. 4.12**, at $t=0.1$ s, the concrete around the impact zone crushed and reached the ultimate strain. As a comparison, the side view of the PC girder at $t=0.1$ s from impact test video is presented in **Fig. 4.13**. **Fig. 4.14** is a damage contour of the PC girder bottom face, which indicates that the failure surface of the PC girder is formed beginning from wedge-shaped diagonal cracks under impact zone.

4.1.6 Parametric Study

4.1.6.1 Velocity

During our impact test, the initial velocity of the impact cart was determined as 24 km/h (15 mph). Velocities of 16 km/h (10 mph), 13 km/h (8 mph), and 8 km/h (5 mph) were studied and the results were compared. **Fig. 4.15** shows the horizontal displacements of



Figure 4.13 PC Girder at $t=0.1$ s from Side View

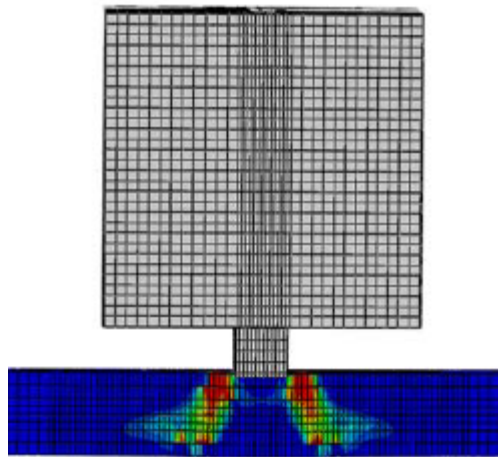


Figure 4.14 Damage Contour at the Bottom Face

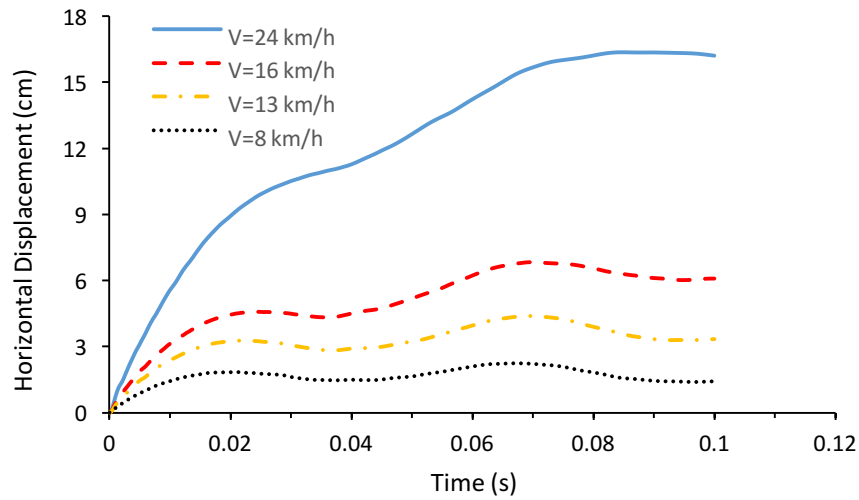


Figure 4.15 Horizontal Displacement under Different Velocities; 1 cm=0.39 inch

the girder midspan in four situations. The maximum values are 16.4 cm (6.5 in.) for $V=24$ km/h (15 mph), 6.8 cm (2.7 in.) for $V=16$ km/h (10 mph), 4.4 cm (1.7 in.) for 13 km/h (8 mph), and 2.2 cm (0.9 in.) for $V=8$ km/h (5 mph), respectively. The impact force time histories under four different initial velocities are illustrated in **Fig. 4.16**, and with the peak values as 3894 kN (875 kip) for $V=24$ km/h (15 mph), 2985 kN (671 kip) for $V=16$ km/h (10 mph), 2510 kN (564 kip) for 13 km/h (8 mph), and 1658 kN (373 kip) for $V=8$ km/h (5 mph). The impact force drops dramatically after reaching the maximum values. The reason for this is because the speed of the specimen moving forward is larger than the speed of the impact cart and the girder is trying to separate from the impactor. The change of the stiffness during the impact process resulted from the local deformation of the PC girder under impact and the variation of the contact area between the impact cart and the girder. **Fig. 4.17** present the damage contours of the PC girder at $t=0.1$ s under different velocities.

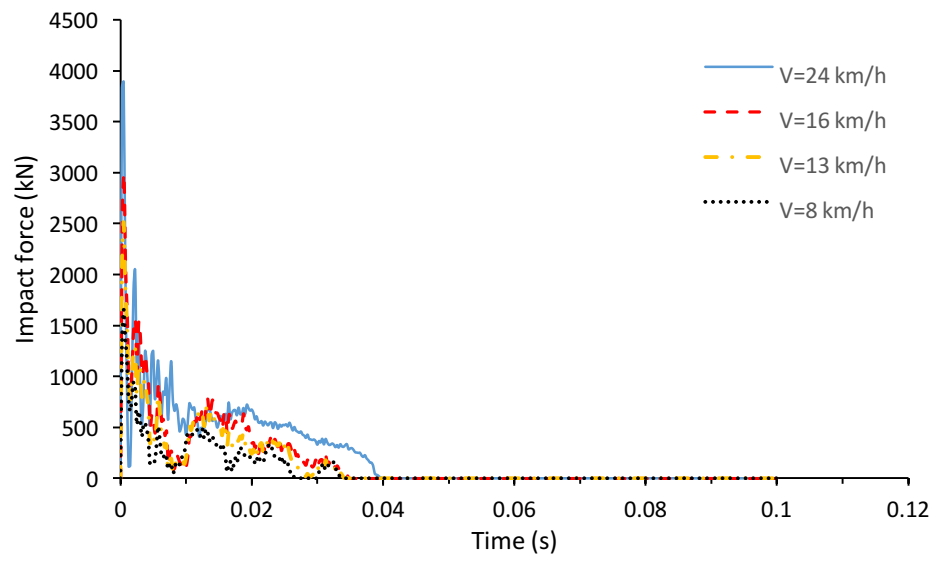
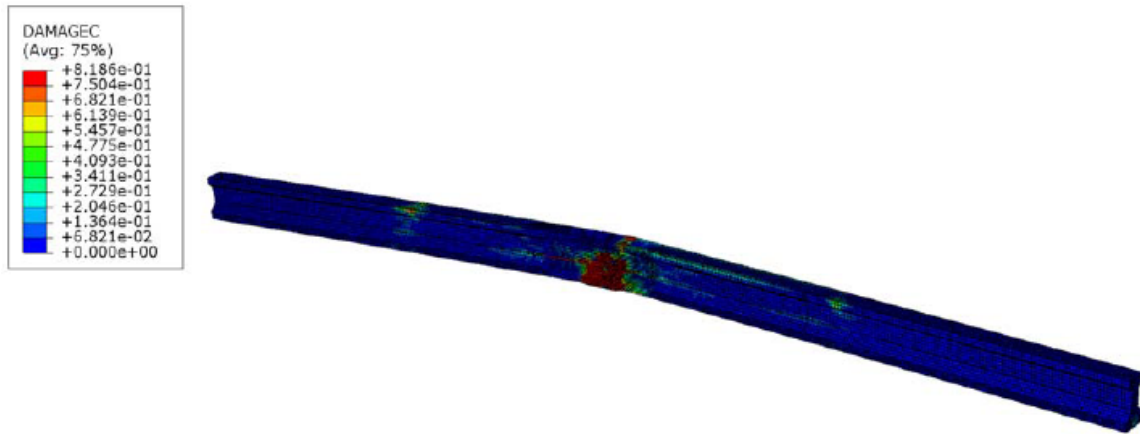
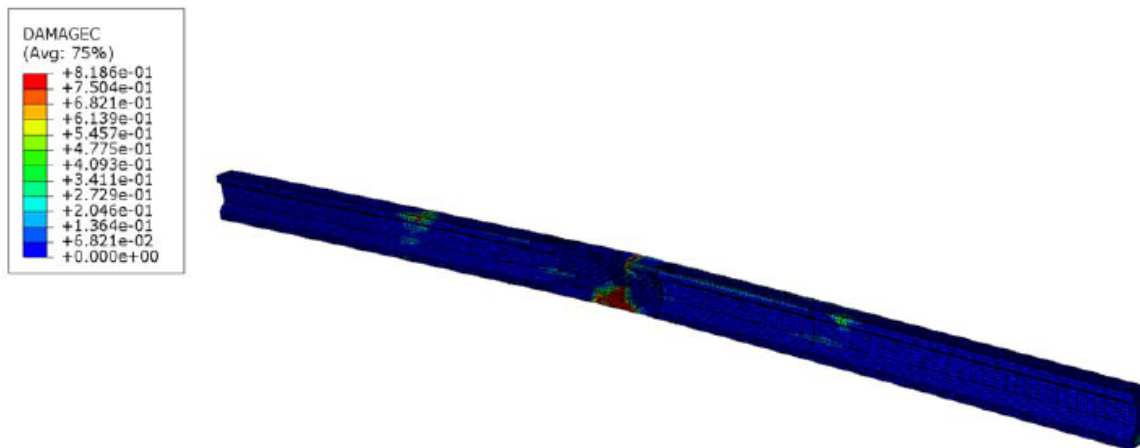


Figure 4.16 Impact Force under Different Velocities; 1 kN=0.225 kip

Figure 4.17 Damage Contours under Different Velocities at $t=0.1$ s

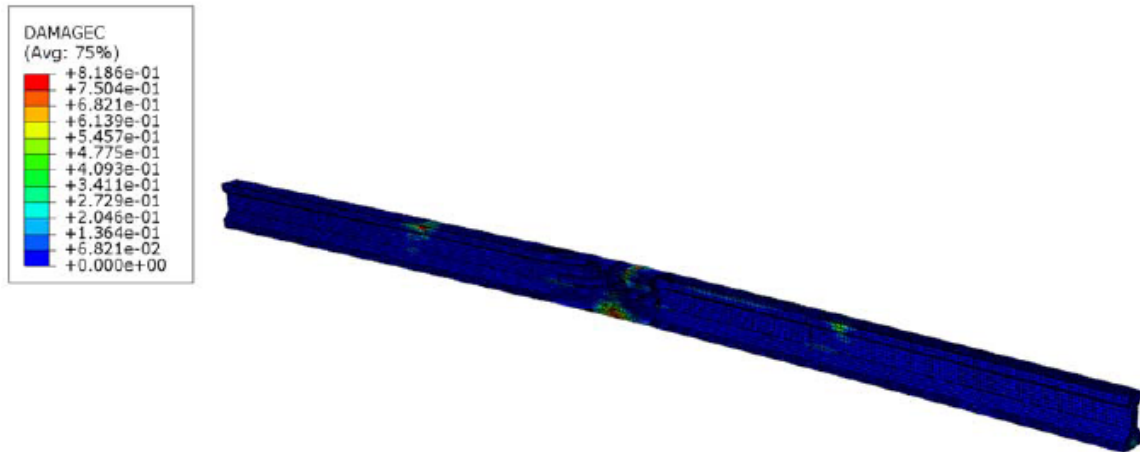


(a) Damage contour at $t=0.1$ s with $V=24$ km/h (15 mph)

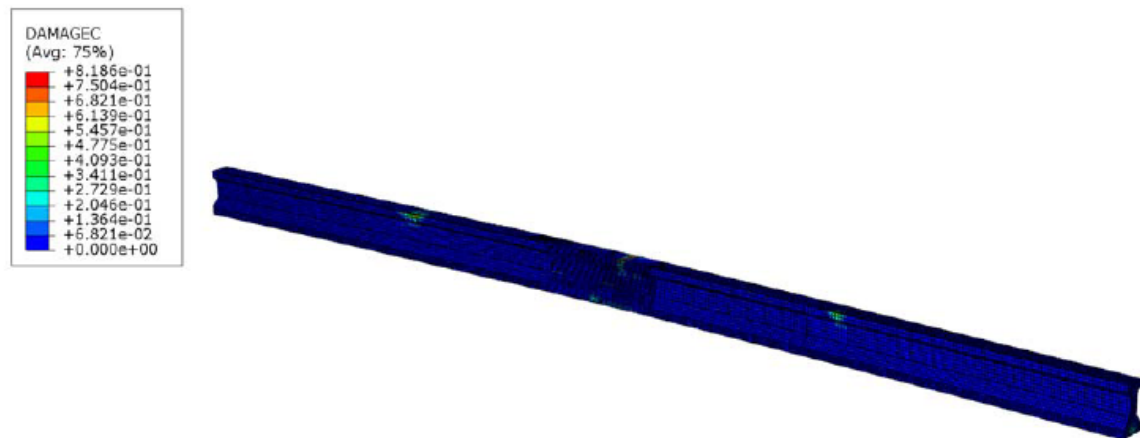


(b) Damage contour at $t=0.1$ s with $V=16$ km/h (10 mph)

Figure 4.17 Continued



(c) Damage contour at $t=0.1$ s with $V=13$ km/h (8 mph)



(d) Damage contour at $t=0.1$ s with $V=8$ km/h (5 mph)

Figure 4.17 Continued

4.1.6.2 Compressive Strength

The concrete compressive strength of the PC girder was 97 MPa (14100 psi). The behavior of the PC girder under different concrete strengths were also studied. **Fig. 4.18** and **Fig. 4.19** present the horizontal displacements and impact forces, respectively. For the concrete with compressive strengths of 97 MPa (14100 psi), 69 MPa (10000 psi), and 41 MPa (6000 psi), the corresponding maximum horizontal displacements are 16.4 cm (6.5 in.), 21.2 cm (8.3 in.), and 28 cm (11 in.). The peak impact forces are 3894 kN (875 kip), 3656 kN (822 kip), and 3403 kN (765 kip), respectively. In **Fig. 4.19**, it can be noticed that with the increasing of the compressive strength, the impact force goes up and the impact duration becomes smaller. Under the same initial impact energy, the PC girder with lower compressive strength has larger horizontal displacement and experiences severer damage. **Fig. 4.20** shows the damage contours of the PC girder at $t=0.1$ s under different concrete compressive strengths.

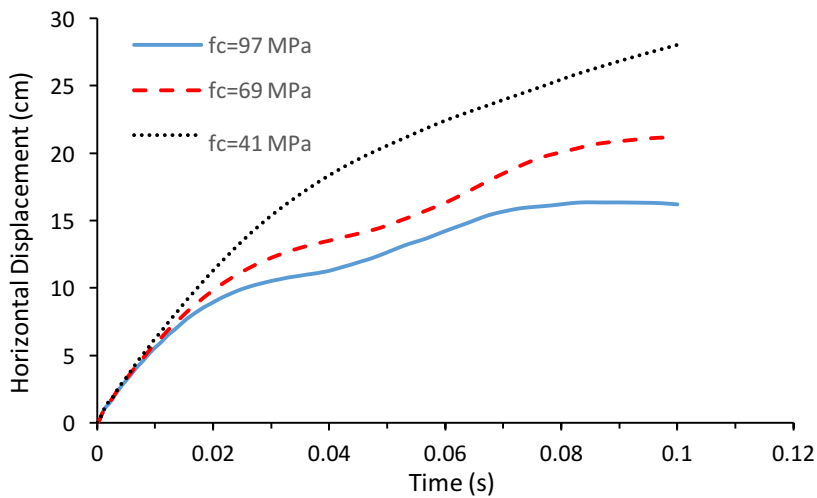


Figure 4.18 Horizontal Displacement under Different Compressive Strengths; 1 cm=0.39 inch

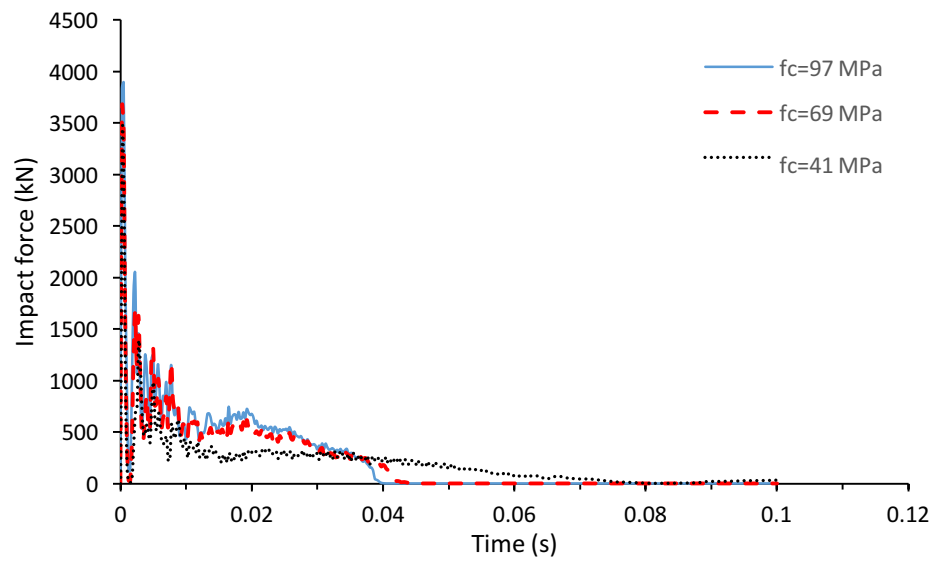
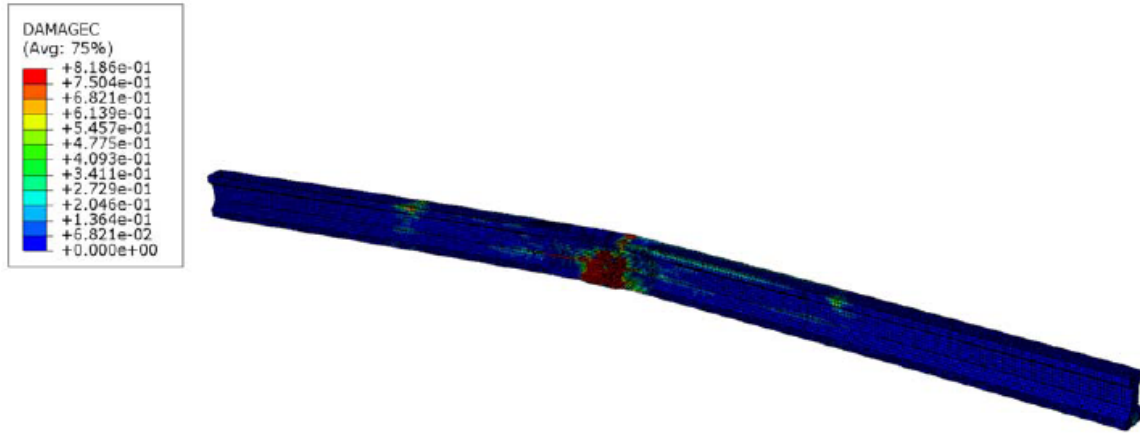
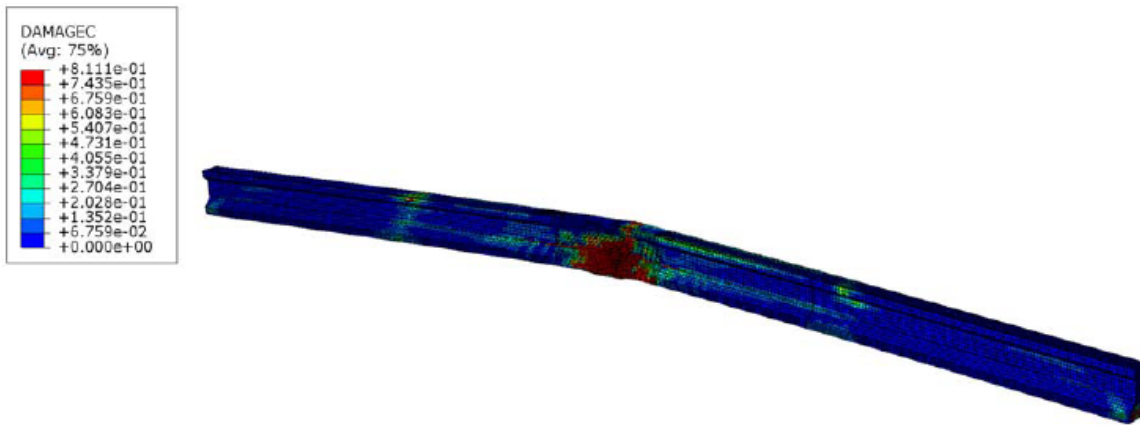


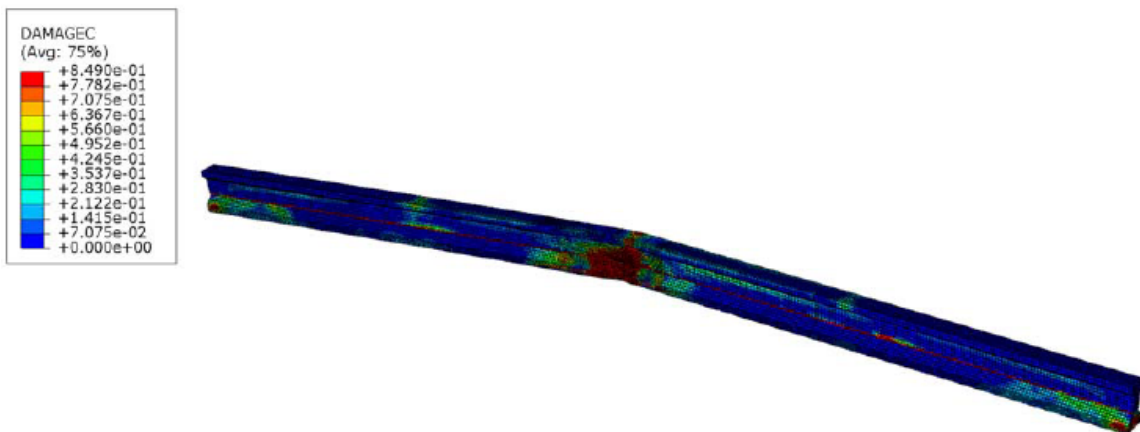
Figure 4.19 Impact Force under Different Compressive Strengths; 1 kN=0.225 kip



(a) Damage contour at $t=0.1$ s with $f_c=97$ MPa (14100 psi)



(b) Damage contour at $t=0.1$ s with $f_c=69$ MPa (10000 psi)



(c) Damage contour at $t=0.1$ s with $f_c=41$ MPa (6000 psi)

Figure 4.20 Damage Contours under Different Compressive Strengths at $t=0.1$ s

4.1.6.3 Impact Area

The behavior of the PC girder under larger impact area ($A=1935 \text{ cm}^2/300 \text{ in.}^2$) was also simulated, the maximum midspan horizontal displacement is 11.5 cm (4.5 in.), which is 4.9 cm (1.9 in.) smaller than that with the impact area of 323 cm^2 (50 in.^2), as shown in **Fig. 4.21**. **Fig. 4.22** shows the damage contours of the PC girder at $t=0.1 \text{ s}$ under different impact areas.

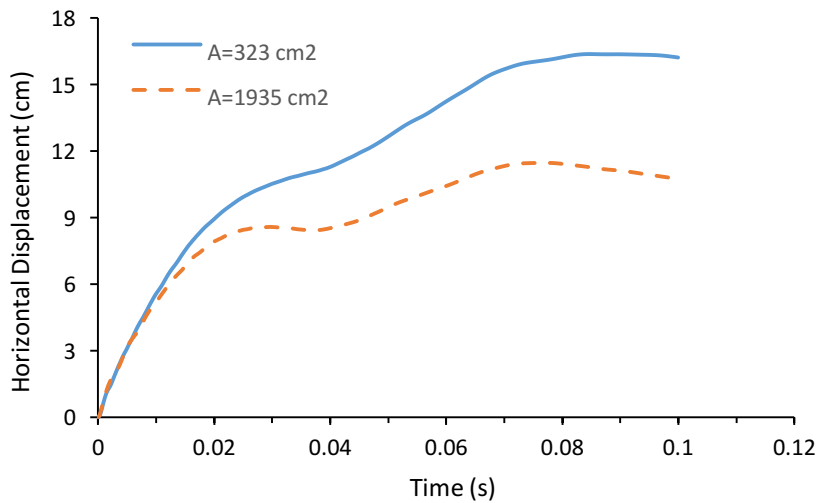
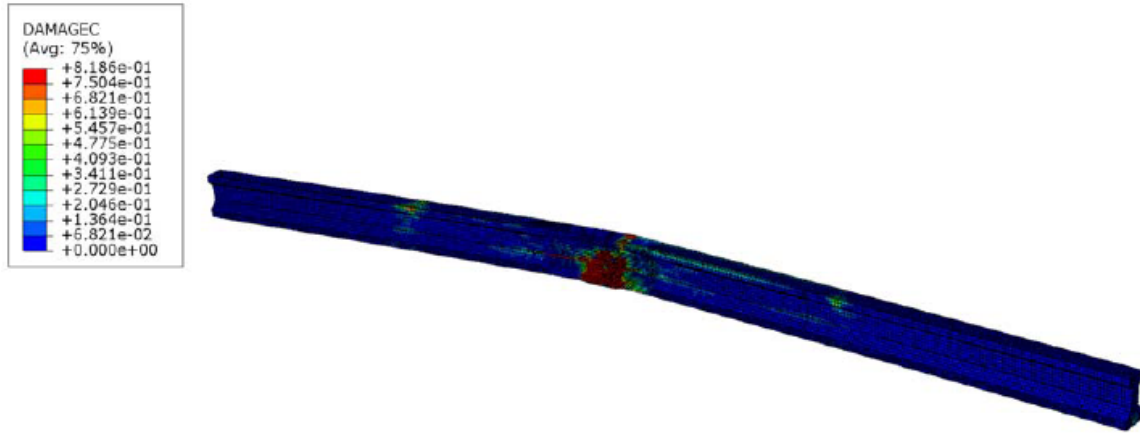


Figure 4.21 Horizontal Displacement under Different Impact Areas; 1 cm=0.39 inch

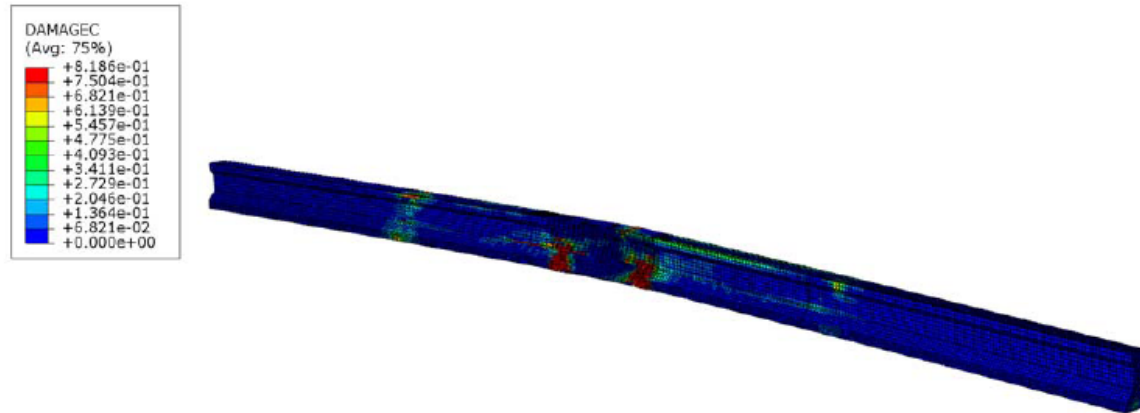
4.1.7 Results Discussion

In this study, based on both testing results and FE results, the PC girder had a shear failure mode when subjected to lateral impact loading. With an initial impact energy of 96 kJ (71 kip-ft), the PC girder was destroyed.

The parametric studies were performed on the PC girder FE model. Under the same initial



(a) Damage contour at $t=0.1$ s with $A=323 \text{ cm}^2$ (50 in.^2)



(b) Damage contour at $t=0.1$ s with $A=1935 \text{ cm}^2$ (300 in.^2)

Figure 4.22 Damage Contours under Different Impact Areas at $t=0.1$ s

impact energy, with the decrease of the concrete compressive strength, the maximum impact force reduces due to the lower stiffness of the impact zone. However, the midspan horizontal displacement increases and the PC girder experiences severer damage. Under the same impact energy, with the increase of the impact area, the maximum midspan horizontal displacement reduces. The concrete didn't crush under the impact zone and the stiffness degradation started from the edge of the impactor when the impact area equals to 1935 cm^2 (300 in.^2).

When the impact velocity decreases, the maximum midspan horizontal displacement, the peak impact force, and the damage zone of the PC girder become smaller. With $V=13 \text{ km/h}$ (8 mph), the damaged area at $t=0.1 \text{ s}$ at the front surface is within the bottom flange, and no wedge-shaped diagonal stiffness degradation is observed from the bottom face. At the same time, the horizontal displacement at midspan is not a localized deformation and the maximum value is 4.4 cm (1.7 in.), and there is no obvious vertical movement observed and the maximum vertical displacement is 2.8 cm (1.1 in.). Therefore, under this circumstance with the initial impact energy of 26 kJ (19 kip-ft) that almost all goes into the structure, the PC girder would not collapse and the girder is treated as safety under the vertical traffic loading but the repair is needed.

5. CONCLUSIONS

5.1 Conclusions

In this research, experimental and analytical studies were performed to evaluate the dynamic behavior of bridge superstructures under lateral impact from over-height vehicles. A full-scale lateral impact testing facility was designed and built on a construction site in Knoxville, Tennessee. An impact cart with an elevated track was selected to perform the impact testing after evaluating the cost and safety of various methods. The setup system consists of an impact cart, a track system, a backstop system, and vertical supports. In a test, the impact cart rolls down the track, and the center of the impactor impacts the bottom of the specimen. The backstop includes steel tubes and wide-flange steel beams in order to prevent the specimen from sliding horizontally during the collision, and the steel tubes are removable so that multiple specimens with different widths can be tested using this facility. In this dissertation, two full-scale lateral impact tests were conducted, with the specimens of a prestressed concrete girder and an HCB bridge. FE simulation of the collision process of the PC girder was performed by using commercial software Abaqus/Explicit, and the FE results were compared with the experimental data. Parametric study was conducted on the PC girder. The conclusions drawn from the tests and analysis are obtained as follows:

1. In the impact testing setup condition, the failure of the prestressed concrete girder

began with punching shear around the impact zone due to the relatively high rigidity of the impactor, as well as the high compressive strength of the PC girder. With the penetration of the impactor, the damaged impact zone behaved as a “hinge” which moved upward due to the heavy weight of both overhangs.

2. HCB bridge experienced no global failure during lateral impact test but only local damage of the FRP shell around the impact zone. Large cracks, debonding of FRP shell from the interior low-density foam, tearing of FRP shell, and discoloration were all observed after testing. FRP has brittle failure modes and impact can introduce damage to FRP laminate in the form of matrix cracking, fiber breakage and delamination between lamina layers.
3. Compared with PC girder, for HCB bridge, at the impact zone, the thickness of the tension reinforcement is 0.45 in., and the majority of the material is low-density foam under the FRP shell. A lot of energy was absorbed through local strain energy of the tension reinforcement and the low-density foam. Therefore, the resilient nature of the materials around impact zone makes HCB bridge an effective structure to resist lateral impact loading.
4. FE results from PC girder impact simulation was calibrated by available testing data.

Concrete Damaged Plasticity (CDP) constitutive model in ABAQUS/Explicit was adopted in this study. The CDP model combines isotropic elasticity with isotropic tensile and compressive plasticity to represent the inelastic behavior of concrete, and captures the stiffness degradation of the concrete.

5. Parametric study was performed on the PC girder by FE analysis. Damage contour indicated that the failure surface of the PC girder was formed beginning from wedge-shaped diagonal cracks under impact zone. With the initial impact energy of 26 kJ (19 kip-ft) that almost all goes into the structure, the PC girder would not collapse and the girder is treated as safety under the vertical traffic loading but the repair is needed.

5.2 Future Work

The FE simulation of the HCB bridge under lateral impact will be continuously performed as the future work.

REFERENCES

Abaqus version 6.11. (2011). Dassault Systems Simulia Corp, Providence, RI.

Abaqus version 6.14. (2014). Dassault Systems Simulia Corp, Providence, RI.

Abendroth, R.E., Fanous, F., and Andrawes, B.O. (2004). "Steel Diaphragms in Prestressed Concrete Girder Bridges", *Final Report to Iowa DOT, Project TR-424*, Center for Transportation Research and Education, Iowa State Univ., Ames, IA.

Aboelseoud, M., Earley, R., and Myers, J.J. (2014). "In-Situ Load Testing Results of Hybrid Composite Beam Bridges in Missouri, USA", the 7th International Conference on FRP Composites in Civil Engineering, Vancouver, Canada.

Aboelseoud, M.A. and Myers, J.J. (2015). "Finite-Element Modeling of Hybrid Composite Beam Bridges in Missouri", *Journal of Bridge Engineering*, V. 20, No. 1.

Agrawal, A.K., Xu, X.C., and Chen, Z. (2011). "Bridge Vehicle Impact Assessment", Project #C-07-10, Final Report to the New York State Department of Transportation.

Ahsan, S. (2012). "Evaluation of Hybrid-Composite Beam for Use in Tide Mill Bridge", Master Thesis, Virginia Polytechnic Institute and State University, September.

American Association of State Highway and Transportation Officials. (2016). "AASHTO LRFD Bridge Design Specification", 2016 Interim Revisions, Washington D.C.

Ando, T., Kishi, N., Mikami, H., and Matsuoka, K.G. (2000). "Weight Falling Impact Tests on Shear-Failure Type RC Beams without Stirrups", *Structures and Materials, Structures under Shock and Impact VI*, V. 8, pp. 579-587.

Banthia, N.P., Mindess, S., and Bentur, A. (1987). "Impact Behavior of Concrete Beams", *Materials and Structures*, V. 20, pp. 293-302.

Bate, S.C.C. (1961). "The Effect of Impact Loading on Prestressed and Ordinary Reinforced Concrete Beams", Her Majesty's Stationery Office, research paper 35, London.

Birtel, V. and Mark, P. (2006). "Parameterised Finite Element Modeling of RC Beam Shear Failure", ABAQUS Users' Conference.

Buth, C.E., Brackin, M.S., Williams, W.F., and Fry, G.T. (2010). "Collision Loads on Bridge Piers: Phase 2. Report of Guidelines for Designing Bridge Piers and Abutments for Vehicle Collisions", Research/Test Report, Texas Transportation Institute.

Cabage, J.V. (2014). "Behavior of Larger-Diameter Strands in the Disturbed Region of Prestressed Concrete Girders", Dissertation, University of Tennessee Knoxville, December.

Carreira, D.J. and Chu, K.H. (1985). "Stress-Strain Relationship for Plain Concrete in Compression", *ACI Journal*, V. 82, No. 6, pp. 797-804.

Comité Euro-International du Béton—Fédération Internationale de la Pré-contrainte (CEB-FIP). (1990). "Model Code 1990", Thomas Telford, Lausanne, Switzerland.

Deng, Y.H., and Tuan, C.Y. (2013). "Design of Concrete-Filled Circular Steel Tubes under Lateral Impact", *ACI Structural Journal*, V. 110, No. 4, pp. 691-701.

EN 1991-1-7 (1996). *Eurocode 1: Actions on Structures- Part 1-7: General Actions- Accidental Actions*.

Erki, M.A., and Meier, U. (1999). "Impact Loading of Concrete Beams Externally Strengthened with CFRP Laminates", *Journal of Composites for Construction*, V. 3, pp. 117-124.

Feldman, L.R., Jirsa, J.O., and Kowal, E.S. (1998). "Repair of Bridge Impact Damage",

Concrete International 20, No. 2, pp: 61-66.

Fu, C.C., Burhouse, J.R., and Chang, G.L. (2004). "Overheight Vehicle Collisions with Highway Bridges", *Transportation Research Record*, No. 1865, pp. 80-88.

Fujikake, K., and Li, B., and Soeun, S. (2009). "Impact Response of Reinforced Concrete Beam and Its Analytical Evaluation", *Journal of Structural Engineering*, V. 135, No. 8, pp. 938-950.

Hillman, J. R. (2005). "Production Application of a Hybrid-Composite Beam System", *IDEA Program Stage 1 Report*, unpublished appendix to IDEA Program Final Report, Contract No. HSR-43, IDEA Program, TRB, National Research Council.

Hillman, J.R. (2008). "Product Application of a Hybrid-Composite Beam System", IDEA Program Final Report for HSR-43, Transportation Research Board National Research Council.

Hillman, J. R. (2012). "Hybrid-Composite Beam (HCB) Design and Maintenance Manual", Final Draft to the Missouri Department of Transportation, Bridge No. B0439-MO 76 over Beaver Creek, Jackson Mill, MO.

Ishikawa, N., Katsuki, S., and Takemoto, K. (2000). "Dynamic Analysis of Prestressed Concrete Beams under Impact and High Speed Loadings", *Structures and Materials, Structures under Shock and Impact VI*, V. 8, pp. 247-256.

Jiang, J.J., and Lu, X.Z. (2013). "Finite Element Analysis of Concrete Structures", *Tsinghua University Press*, Beijing, China.

Jing, Y., Ma, Z.J., and Clarke, D.B. (2016). "Full-Scale Lateral Impact Testing of Prestressed Concrete Girder", *Structural Concrete*, doi: 10.1002/suco.201500224.

Jing, Y., Ma, Z.J., Bennett, R.M., and Clarke, D.B. (2016). "Behavior of Hybrid Composite Beam Bridge under Full-Scale Lateral Impact", *Journal of Bridge Engineering*, under review.

Mindess, S., Banthia, N.P., Ritter, A., and Skalny, J.P. (1986). "Crack Development in Cementitious Materials under Impact Loading", *Materials Research Society Symposia Proceedings*, V. 64, pp. 217-223.

Mitchell, C.A. (2014). "Design and Construction of a Full-Scale Lateral Impact Testing Facility", Master Thesis, University of Tennessee Knoxville, December.

Rokugo, K., Kunieda, M., and Kamada, T. (2001). "Evaluation and Improvement of Performance of Concrete Beams against Impact Loading", *Fracture Mechanics of Concrete Structures*, pp. 1065-1070.

Shi, J., Bao, L., and Kemmochi, K. (2014). "Low-velocity Impact Response and Compression after Impact Assessment of Recycled Carbon Fiber-Reinforced Polymer Composites for Future Applications", *Polymer Composites*, V. 35, No. 8, pp. 1494-1506.

Simms, L.G. (1945). "Actual and Estimated Impact Resistance of Some Reinforced Concrete Units Failing in Bending", *Journal of the Institution of Civil Engineers*, No. 4.

Skov, K and Olesen S.O. (1975). "Part 5: Impact Resistance of Reinforced and Prestressed Concrete Members", *Material and Structures*, V. 8, No. 2, pp. 116-125.

Snape, T. and Lindyberg, R. (2009). "Test Results: HC Beam for the Knickerbocker Bridge", AEWB Report 10-15, Orono: Advanced Structures & Composites Center.

Tang, T.P., and Saadatmanesh, H. (2005). "Analytical and Experimental Studies of Fiber-Reinforced Polymer-Strengthened Concrete Beams under Impact Loading", *ACI Structural Journal*, technical paper, V. 102, No. 1.

Telang, N.M., Dumlao, C., Mehrabi, A.B., Ciolko, A.T., and Gutierrez, J. (2006). "Field Inspection of In-Service FRP Bridge Decks", *NCHRP Report 564*, Washington, DC.

Wee, T.H., Chin, M.S., and Mansur, M.A. (1996). "Stress-strain relationship of high-strength concrete in compression", *Journal of Materials in Civil Engineering*, V. 8, No. 2, pp. 70-76.

Xu, L.J., Lu, X.Z., Guan, H., and Zhang, Y.S. (2013). "Finite-Element and Simplified Models for Collision Simulation between Overheight Trucks and Bridge Superstructures", *Journal of Bridge Engineering*, V. 18, No. 11, pp. 1140-1151.

Xu, L.J., Lu, X.Z., Smith, S.T., and He, S.T. (2012). "Scaled Model Test for Collision between Over-Height Truck and Bridge Superstructure", *International Journal of Impact Engineering*, V. 49, pp. 31-42.

Yang, M.J., Qiao, P.Z., McLean, D.I., and Khaleghi, B. (2010). "Effects of Overheight Truck Impacts on Intermediate Diaphragms in Prestressed Concrete Bridge Girders", *PCI Journal*, V. 55, No. 1, pp. 58-78.

Zaouk, A.K., Bedewi, N.E., Kan, C.D., and Marzougui, D. (1996). "Validation of A Non-linear

Finite Element Vehicle Model Using Multiple Impact Data”, the George Washington University, FHWA/NHTSA National Crash Analysis Center.

VITA

Yuan Jing was born in Xianyang, Shaanxi, China on November 9, 1989. She got her BS Degree in Highway Engineering from Chang'an University, China in July, 2012.

Yuan Jing is currently pursuing a Doctorate Degree in Civil and Environmental Engineering Department from the University of Tennessee, Knoxville. She has accepted a position as an Assistant Professor in Bridge Engineering at Chang'an University, China.