

Performance of sub-standard reinforced concrete barriers in protecting bridge piers against
vehicular collision force

by

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B.Sc., An-Najah National University, 2012

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AN ABSTRACT OF A DISSERTATION

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Department of Civil Engineering
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Abstract

An important purpose of using Reinforced Concrete (RC) barriers in the highway inventory is to act as railing that protects bridge piers against vehicular collision force (VCF). These barriers are designed to absorb the collision energy and/or redirect the vehicle away from the parts being protected. According to the bridge design specifications of the American Association of State Highway Transportation Officials (AASHTO), barriers used to protect bridge piers should have a minimum height of 1067 mm and survive MASH Test Level 5 (TL-5). Many existing barriers that are currently used in roadways do not meet this requirement, hereby, they are referred to as "sub-standard" barriers. While these barriers are found to be inadequate to solely protect bridge piers against VCF, their presence to intervene vehicle-pier collisions will contribute to reducing the severity of the crash effects. Consequently, they reduce the VCF demand that AASHTO specifies for piers to resist when upgrading existing bridges. This research aims at assessing the performance of sub-standard RC barriers as protective structures to bridge piers against VCF and quantifying their contribution towards reducing the Equivalent Static Force (ESF) that piers required to resist as per the modern AASHTO specifications. The research will present existing procedures in determining the transverse static capacity of RC barriers and propose alternative and accurate methodologies to predict this capacity. Also, a matrix of crash scenarios is simulated in dynamic explicit analysis through the multi-purpose finite element software LS-DYNA to obtain a comprehensive idea about the behavior of sub-standard RC barriers. The investigated parameters include energy dissipation, velocity reduction, contact force absorption and lateral displacement. Finally, a bridge pier that was found to be deficient to address the VCF under the required AASHTO's ESF is used in a simulation matrix behind the sub-standard barrier. A series of Dynamic Impact Force (DIF) time histories are obtained and used to extract the Equivalent Static

Force (ESF). Comparing the resulting ESF with the force required by AASHTO, a reduction in the ESF due to the presence of sub-standard RC barriers is proposed. The main findings of this research revealed that the existing procedure used by AASHTO to determine the transverse static capacity of RC barriers might, in some cases, result in about 50% capacity underestimation. Also, the inadequacy of sub-standard barriers in absorbing and/or redirecting the impacting vehicle refers to geometrical deficiency (insufficient height). In other words, the sub-standard barrier can resist high impact load demands and protect the piers behind if sufficient height, such as 1070 mm for TL-4, exists to prevent contact between the impacting vehicle and the piers. Finally, the presence of sub-standard barriers might cause a reduction in the ESF that AASHTO requires bridge piers to resist by at least 25%.

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Abbreviations

FHWA: Federal Highway Administration

AASHTO: American Association of State Highway Transportation Officials

NCHRP: National Cooperative Highway Research Program

MASH: Manual for Assessing Safety Hardware

KDOT: Kansas Department Of Transportation

TL: Test Level

RC: Reinforced Concrete

LRFD: Load and Resistance Factor Design

VCF: Vehicular Collision Force

ESF: Equivalent Static Force

YLA: Yield Line Analysis

FEA: Finite Element Analysis

DIF: Dynamic Impact Force

PIF: Peak Impact Force

TTI: Texas Transportation Institute

QoI: Quantities of Interest

SUT: Single Unit Truck

TST: Tractor Semi Trailer

LESF: Local Equivalent Static Force

GESF: Global Equivalent Static Force

PFMSA: Peak of Fifty Milli-Second Average

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Dedication

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To my brothers, my sisters, and my friends.

I dedicate this dissertation to you. Your support, guidance, and love have been invaluable to me.

Thank you for being my rock and for helping throughout this achievement.

With all my love and gratitude,

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Chapter 1 - Introduction

1.1 Background

Critical infrastructures such as bridges should be resilient to extreme loading events. Among the potential hazardous events, vehicle collisions are becoming increasingly common. Safety of bridge piers during vehicular Collision events is critical to maintain the bridge operational. Many vehicle-pier collision events have been reported with catastrophic consequences, Figure 1-1. Between 1951-2000, out of 617 reported bridge collapse events, 31 were caused by vehicle impacts with bridge piers, representing 5% of the collapse events. [1,2].



Figure 1-1. Failure of bridge structures due to vehicle collision events (N. Nemec, [3])

Vehicular Collision Force (VCF) was first addressed by the American Association for State Highway Transportation Officials (AASHTO) as a design limit state for bridge piers in the 2nd edition of the bridge design specification. This edition was published in 1998 [4] and required bridge piers to be designed to structurally resist an Equivalent Static Force (ESF) of 1,780 kN (400 kips). After studying a collection of crash data, analyzing bridge pier collision scenarios through Finite Element Analysis (FEA) and collecting empirical data of the forces experienced during full-scale collision tests, the research team at the Texas Transportation Institute (TTI) recommended

increasing the ESF to 2,670kN (600 kips), [5,6]. This value is reflected in the latest AASHTO bridge design specification (Section 3.6.5) [7].

The alternative solution suggested by the AASHTO provisions (Section 3.6.5.1) to address the VCF is redirecting the impacting vehicle or absorbing the VCF through providing protection to bridge piers. One of the methods stated by AASHTO to protect bridge piers is the use of a 1,070 mm (42 in)-high intervening structure that has its traffic face located more than 1 m (3.25 ft) from the component being protected. Also, the used structure should be able to survive Test Level-5 (TL-5) crash event as specified in the guidelines of the AASHTO's Manual for Assessing Safety Hardware (MASH), [8].

Reinforced Concrete (RC) barriers are commonly used as intervening structures to protect bridge piers against VCF. Many existing RC barriers that are currently used in old bridge infrastructures do not meet the acceptance criteria for TL-5. Hereby, they are referred to as "sub-standard" barriers. While these barriers are found to be inadequate to solely protect bridge piers against VCF, their presence to intervene vehicle-pier collisions will contribute to reducing the severity of the collision effects. Consequently, they reduce the ESF that AASHTO specifies for piers to account for VCF.

The current AASHTO specifications neglect the presence of sub-standard barriers when addressing the VCF on bridge piers. This refers to the absence of research studies that accurately estimate their capacity, assess their performance during crash events, and quantify their contribution in reducing the ESF required to be resisted by piers. Therefore, having accurate analysis methodologies to obtain the structural capacity of sub-standard RC barriers, and assess their efficiency during crash events, is crucial to determine their contribution in absorbing the VCF and reducing the ESF that piers are required to resist. For this purpose, this research will present

analytical methodologies to accurately estimate the lateral static capacity of RC barriers. Also, numerical simulations will be conducted to investigate the behavior of a sub-standard RC barrier during vehicular collision events and evaluate its response under dynamic impact load effects. Finally, the numerical simulations will be extended to obtain an estimate of the VCF on bridge piers in the presence of a sub-standard RC barrier and the reduction in the ESF that piers are required to resist as per the modern AASHTO specifications.

1.2 Aim and objectives

The aim of this research is to assess the performance of sub-standard RC barriers as protective structures to bridge piers against VCF, and to quantify their contribution towards reducing the ESF that piers required to resist as per the modern AASHTO specifications. In order to achieve this aim, several objectives have been set as follows:

1. Develop rigorous analysis and innovative methodology to evaluate the transverse structural capacity of RC barriers.
2. Assess the performance of sub-standard RC barriers as protective structures to bridge piers against vehicular collision force
3. Estimate the VCF on bridge piers in the presence of sub-standard intervening RC barriers

1.3 Scope

This dissertation is composed of five chapters. Chapter one introduces the problem, defines the aim and objectives of this research, and explains the scope of the work that will be presented in this dissertation. Chapter two presents the development of a detailed analysis methodology to obtain the transverse load capacity of RC barriers using theories and mechanics of reinforced concrete. Also, this chapter introduces an alternative innovative truss analogy method to simulate RC barriers and obtain their transverse capacity.

Chapter three and four include explicit dynamic simulations of a matrix of crash events using the general-purpose FEA program LS-DYNA. In chapter three, the simulation matrix of the crash environment consists of only the barrier and the crashing vehicle. A comprehensive behavior analysis on the performance of sub-standard RC barriers during heavy vehicles crash events is obtained in this chapter. Several Quantities of Interest (QoI) will be presented and discussed. In chapter four, the simulation matrix of the crash environment will include a representative bridge pier placed behind a sub-standard RC barrier. The time histories of the dynamic impact forces on the pier including the Peak Impact Force (PIF) are obtained through the analysis of the simulation matrix. The ESF on the pier in the presence of the sub-standard RC barrier is then derived from the time histories. The research conclusions and recommendations will be presented in chapter five.

Chapter 2 - Rigorous Analysis and Innovative Methodology to Evaluate the Capacity of RC Barriers Subjected to Vehicular Collision Force

Fahed H. Salahat¹, Hayder A. Rasheed², Christopher A. Jones³, Isaac Klugh⁴

2.1 Abstract

Reinforced Concrete (RC) barriers are used for different purposes in the highway inventory. An important purpose is the use of concrete barriers to act as railing that protects bridge piers against vehicular collision force (VCF). Therefore, these barriers are designed to absorb the collision energy and/or redirect the vehicle away from the parts being protected. Accurate estimation of the capacity of RC barriers during crash event is an important consideration in their design and placement. The American Association of State Highway Transportation Officials (AASHTO) considers yield line analysis (YLA) to predict the barrier's capacity. The analysis method proposed in AASHTO involves some assumptions that are intended to simplify the analysis process. These assumptions and simplifications have been shown to significantly underestimate the barrier's actual capacity and might disqualify many existing RC barriers to act as intervening structures due to structural inadequacy. Many researchers have proposed alternative failure patterns and methodologies in an attempt to better predict the capacity of RC barriers. This research shows that AASHTO's YLA, with the current failure mechanism, can still predict the barrier's capacity when some of the simplifying assumptions are eliminated. Also, the research presents an alternative

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innovative truss analogy model to accurately predict the capacity of RC barriers. The results of the rigorous YLA and the proposed truss model are validated by finite element analysis (FEA) using Abaqus. The results of this research will help structural engineers in the highway industry to find detailed analysis guidance on how to accurately estimate the capacity of RC barriers using YLA for design and/or assessment purposes.

Keywords: RC barriers, Yield line analysis, Truss analogy approach, Finite element analysis.

2.2 Introduction

Reinforced concrete (RC) barriers are widely used in transportation infrastructures for different purposes. For instance, median barriers are used to separate opposing lanes of traffic, and railing barriers are used to prevent vehicles access roadway shoulders and protect sign structures and pedestrians. In addition, it is very common to find RC barriers used as intervening structures protecting bridge piers against vehicular collision force (VCF). Therefore, estimating the capacity of these barriers when subjected to VCF is an important consideration in their design and placement.

The framework that leads to a successful placement of these barriers includes three main factors: knowing the required performance level, accurately analyzing and designing the barrier to meet the demands of the required performance level and testing the barrier to ensure satisfying the acceptance criteria. Among the previous three factors, the second one seems to be the most challenging. The reason is simply because one can set objectives (performance levels), and measures to decide whether the objectives are met (acceptance criteria), yet the methodology leads to achieving the objectives.

The first document that considered specifying the performance levels and the acceptance criteria was the NCHRP-Report 350 published in 1993 [9]. The performance levels, referred to as “Test

Levels (TLs)”, in the NCHRP-Report 350, differ according to the impact conditions. A specific impact condition is a function of the vehicle designation (passenger car, single unit truck, tractor-semitrailer, etc.) the impact speed, and the impact angle. Higher TLs involve more severe impact conditions such as heavy vehicles travelling at high velocity. The same document included the acceptance criteria which were in three categories: structural adequacy, occupant risk, and post-impact vehicular trajectory. The criteria imply that, for a test article (barrier) to be acceptable for a certain TL, it has to be structurally adequate to dissipate the energy of the impacting vehicle specified for that TL without excessive damage to the barrier or penetration. Also, minimizing the risk of occupant injury and any potential subsequent multivehicle accident (bringing the impacting vehicle to a controlled stop).

The NCHRP-Report 350 recommended procedures that were implemented by the Federal Highway Administration (FHWA) in 1998. About a decade later, the American Association for State Highway Transportation Officials (AASHTO) published a Manual for Assessing Safety Hardware (MASH) in 2009. This manual upgraded the impact conditions presented in NCHRP-Report 350 for some TLs to reflect more damaging crash events and produce more robust barriers for certain applications. The latest MASH was published in 2016 [8], and is currently adopted by AASHTO’s LRFD Bridge Design Specifications (Section 13) to define the TLs that will decide the design forces for traffic railings [7]. The different TLs as per MASH are shown **Table 2-1**.

After knowing the performance level and the acceptance criteria, the challenging task becomes designing the barrier to meet the performance level and pass the acceptance criteria. This is typically achieved through designing barriers to resist certain level of loading and satisfy geometry proportioning. For example, Table A13.2.1 in AASHTO LRFD Bridge Design Specifications [7] shows the design forces and designation for traffic railing required for different TLs.

Table 2-1: MASH test levels

Vehicle Type	Small automobiles	Pickup truck	Single-unit truck	Tractor-trailer	Tractor-tanker-trailer
Mass (10^3 kg)	1.1 – 1.5	2.27	10	36	36
Angle of attack	25°	25°	15°	15°	15°
Speed (km/h)					
TL-1	50	50	na	na	na
TL-2	50	70	na	na	na
TL-3	100	100	na	na	na
TL-4	100	100	90	na	na
TL-5	100	100	na	80	na
TL-6	100	100	na	na	80

na: not applicable

The most common analysis method used to verify whether or not a proposed barrier design meets the requirements of a performance level is the Yield Line Analysis (YLA). It was first introduced by Hirsch in 1978 [10], and is currently adopted by AASHTO's LRFD Bridge Design Specifications (Section 13). This method is based on equating the work done by the external applied forces and the internal energy developed through the formation of yield lines due to triangular (V-shape) failure pattern. However, the current AASHTO's procedure of YLA include some assumptions that are intended to simplify the analysis process.

Many researchers criticized the simplified AASHTO's procedure of YLA in terms of the capacity estimation and the failure pattern. Jeon et al. [11] conducted a full-scale test varying the loading patterns to simulate vehicle crash. The results revealed a lower load than that predicted by AASHTO's equations and a different failure pattern that looks more as W-shape. This failure pattern is agreed on by Cao et al. [12] through dynamic explicit numerical simulations as well as quasi-static pushover analysis. Loken et al. [13] referred to the suggested W-shape as the modified YLA and made a comparison between the traditional V-shape and the modified W-shape failure patterns in terms of punching shear capacities. The study suggested that significant increase of the punching shear capacity exists in the modified YLA. While punching shear is a possible failure

mechanism that was reported in many studies [14–16], it is less likely for this failure mode to control when enough transverse reinforcement is available.

The existing literature seems to agree that the current AASHTO's procedure of YLA needs to be subject to further research. In this context, a distinction needs to be made between using AASHTO's YLA as an initial qualifier for proposed barrier designs and judging the capacity of barriers for decisions related to their structural adequacy and crashworthiness. The former is a conservative analysis that will produce barriers designed with higher capacity than needed. On the other hand, the latter is a very conservative analysis that will significantly underestimate the actual capacity of RC barriers. While both cases seem to have negative economic effects, underestimating the capacity of existing RC barriers has more significance. This is due to the load demands for some TLs that have been upgraded, which disqualified many of these barriers from being structurally adequate for their intended purpose.

An example of the first case is the proposed design and development of a TL-5 RC barrier by Rosenbaugh et al. [17]. AASHTO's procedure of YLA was used to verify the initial loading capacity and geometric requirements as per Table A13.2.1 [7]. Then the barrier underwent cycles of design cost optimization before it was subjected to an actual TL-5 crash event to pass the acceptance criteria.

On the other hand, Bullard et al. [18] reported an experimental study of different crash tests according to the upgraded MASH requirements to evaluate the performance of commonly existing barriers. It was reported that 45% of the tested barriers failed to meet the acceptance criteria due to vehicle rolling over. No significant damage such as major structural cracks or penetration of the barrier was observed. Although this reason is sufficient to disqualify the barrier, this failure is related to geometrical deficiency (insufficient height). To further support this argument, a

numerical dynamic analysis study, conducted by Salahat et al. [19], estimated the VCF transferred to bridge piers in the presence of a disqualified RC Jersey barrier as a protective structure to bridge piers. The results showed that the disqualified barrier was able to reduce the equivalent static load demand by at least 25%. Interestingly, it was reported that the remaining load transferred to the pier was due to the contact between the cargo area and the pier; no structural damage or penetration of the barrier was observed.

Therefore, it is crucial to have a guidance on detailed analysis methods and procedures that can accurately estimate the structural capacity of RC barriers. This will help distinguishing the geometrical deficiencies from the structural inadequacy as it relates to the acceptance criteria. For this purpose, this research presents a rigorous YLA according to the current and common V-shape failure pattern. Many of the simplifying assumptions will be eliminated and a clear analytical solution will be presented. Also, an original and innovative alternative method is presented to predict the lateral capacity of RC barriers using a truss analogy model. The developed model provides guidelines on how to simulate the RC barrier as an equivalent space truss. This approach tracks the formation of the V-shaped yield lines through incremental analysis. Finally, a case study of an RC barrier will be analyzed using three different techniques: the rigorous YLA, the equivalent truss method, and the numerical finite element analysis (FEA) using Abaqus. The findings confirmed the accuracy of the proposed solutions. The scope of this work includes the flexural capacity of barriers since shear failures are assumed to be avoided through sufficient distribution of transverse reinforcement.

2.3 AASHTO's Procedure of YLA

The simplified YLA procedure by AASHTO implements several assumptions, some of which are reasonable to assume but others may yield significant underestimation of the RC barrier capacity.

These assumptions are summarized as:

1. The deck has sufficient resistance to the applied transverse forces thus the yield line failure pattern will remain within the parapet.
2. The presence of sufficient longitudinal length of the parapet to produce the assumed V-shape yield line failure pattern.
3. The flexural capacity of the RC barrier is only from the concrete contribution; the contribution of the stirrups and/or ties is to prevent shear and diagonal tension.
4. The wall resistance as the average of its value along the height when the width of the barrier varies along the height.
5. The negative and positive wall resisting moments are equal

In the first and second assumptions, the deck slab is usually heavily reinforced to provide sufficient anchorage to the parapet dowels; and barriers usually run for distances long enough to ensure the development of the yield lines according to the suggested V-shape failure pattern. However, the last three assumptions seem to be causing underestimation. It is obvious in the third assumption that neglecting the longitudinal and transverse reinforcement will result in a lower flexural capacity. In the fourth assumption, the cantilever action produces a moment around an axis parallel to the longitudinal axis of the bridge. This moment demand has a linear function that is minimum at the top of the barrier and peaks at the support. Therefore, averaging the sectional capacity along the height reduces the flexural capacity at locations of high demand and increases the capacity where it is not needed as shown in **Figure 2-1**. The three zones in **Figure 2-1** indicate that the

flexural capacity function has geometrical (width) discontinuity along the height of the barrier. The same applies when calculating the capacity function around an axis vertical to the longitudinal axis of the barrier.

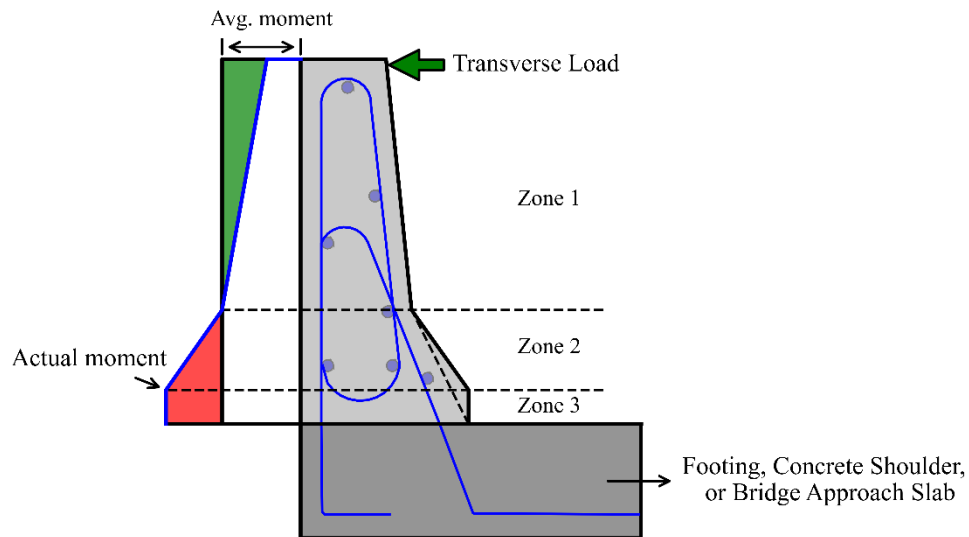


Figure 2-1. Variation of the actual vs. the assumed moment capacity

For the final assumption, equating the positive and negative resisting moment will not make a big difference because of assumptions three and four. However, if the barrier's reinforcement is considered in calculating the flexural capacity, minimum variation between the positive and negative sectional capacities happens around the longitudinal axis (x-axis in **Figure 2-2**). This is because each side will have same number of stirrups legs. On the other hand, the sectional capacity around the vertical axis (z-axis in **Figure 2-2**) is lower at the back side of the barrier (less longitudinal rebar). This lower capacity will conservatively be used for both positive and negative moments as per assumption five above. According to the proposed V-shape failure pattern, this lower resistance coming from the positive moment will represent the two front yield lines in terms of the resisting components around the vertical axis, M_z in **Figure 2-2**. Therefore, one can see how the error imposed by these assumptions accumulates to result in a significant underestimation of the barrier's transverse capacity. Based on the previous assumptions, the current equations as per AASHTO LRFD Bridge Design Specifications (Section 13) [7] are given in **Eq. 1** and **Eq. 2**.

$$R_w = \left(\frac{2}{2L_c - L_t} \right) \times \left(8M_b + 8M_z + \frac{M_x L_c^2}{H} \right) \quad \text{Eq. 1}$$

$$L_c = \frac{L_t}{2} + \sqrt{\left(\frac{L_t}{2} \right)^2 + \frac{8H(M_b + M_z)}{M_x}} \quad \text{Eq. 2}$$

Where:

R_w = total transverse resistance of the railing (kips)

L_c = critical length of yield line failure pattern (ft)

L_t = longitudinal length of distribution of impact force F_t (ft), specified in (Table A13.2-1) [7]

M_b = additional flexural resistance of beam in addition to M_z , if any, at top of wall (kip-ft)

M_x = flexural resistance of cantilevered walls about an axis parallel to the longitudinal axis of the bridge (kip-ft/ft), (referred to as M_c in AASHTO's specifications).

M_z = flexural resistance of the wall about its vertical axis (kip-ft), (referred to as M_w in AASHTO's specifications).

H = height of wall (ft)

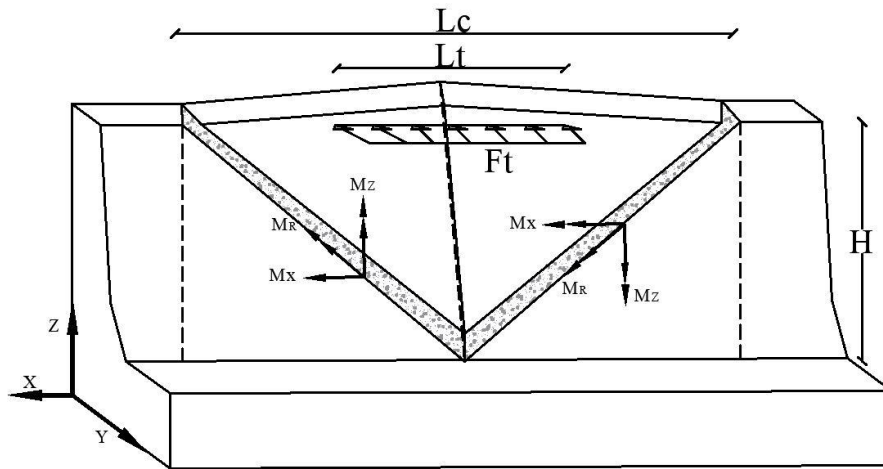


Figure 2-2. Difference in negative and positive moments capacities (a) face yield lines (b) back yield line
The total transverse resistance R_w should be equal or greater than F_t , the resultant transverse force specified in (Table A13.2-1) [7] and assumed to be acting at top of a concrete wall.

2.4 Rigorous YLA of RC Barriers

The procedure explained in this section is targeted to cover the generalized case of RC barriers where the barrier width is not uniform along its height and the positive and negative flexural capacities are not equal due to differences in reinforcement distribution at the back and front face of the barrier. This procedure covers other simpler barriers configurations such as when the barrier has uniform width along its height, or the reinforcement is similar at both sides.

2.4.1 Sectional capacity

The flexural capacity around certain axis is found after dividing the barrier into different sections at the locations of geometrical discontinuities. For simplicity, the moment function of zone 2 is merged with that of zone 3 by following sloped dotted line in **Figure 2-1**. The procedure for obtaining the sectional flexural capacity around the vertical and longitudinal axes is shown in **Figure 2-3 (a)** and **Figure 2-3 (b)** respectively. The procedures shown **Figure 2-3** will be applied in detail in the case study section.

2.4.2 Formation of yield lines

The following are the assumptions that were made to develop the yield line analysis.

1. Concrete is inextensible through the thickness.
2. For barriers that have sloped sides, the value of the deformation angle measured with respect to an assumed vertical plane and the deformation angle of the actual sloped side is almost the same. Therefore, the angle used in the derivations are referenced with respect to the vertical plane.

To prove the second assumption, consider a unity deformation at the top of a barrier with two slopes as shown in **Figure 2-4**. The rotation of the vertical plane is given in **Eq. 3**.

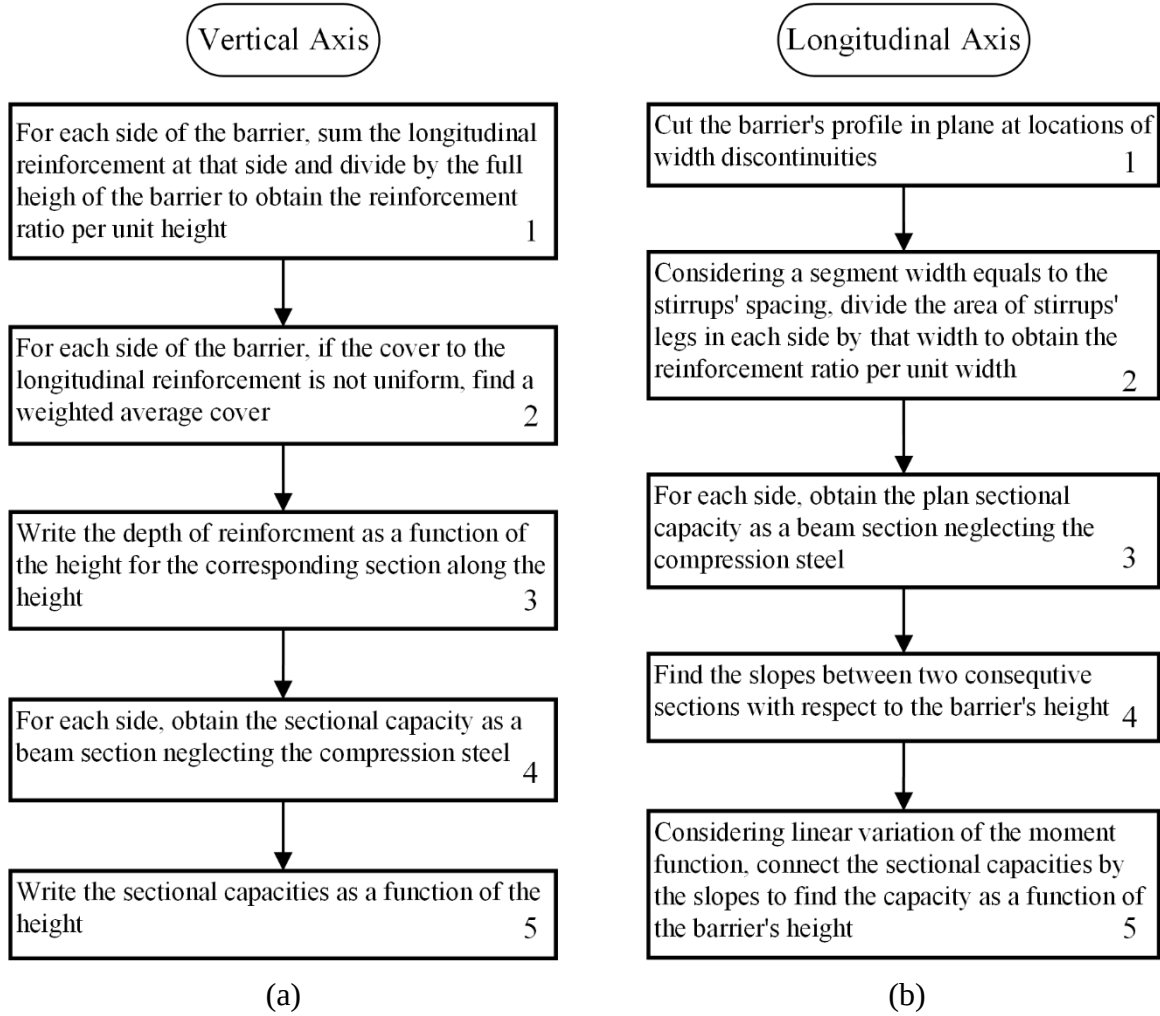


Figure 2-3. Procedure to obtain sectional capacity around (a) vertical axis (b) longitudinal axis

$$\theta_1 = \tan^{-1} \frac{1}{h_1} \quad \text{Eq. 3}$$

The distance d_1 is obtained as in **Eq. 4**. Using the first assumption, the inextensibility of concrete, the distances d_1 , d_2 , and d_3 should all be equal:

$$d_1 = d_2 = d_3 = h_2 \tan \theta_1 = \frac{h_2}{h_1} \quad \text{Eq. 4}$$

The distances a_2 and a_3 are given in **Eq. 5** and **Eq. 6**.

$$a_2 = h_2 \tan \beta_2 \quad \text{Eq. 5}$$

$$a_3 = h_3 \tan \beta_3 \quad \text{Eq. 6}$$

The tangent of total inclination at the lower part of the barrier is given in **Eq. 7** as:

$$\tan (\beta_2 + \theta_2) = \frac{a_2 + d_2}{h_2} \quad \text{Eq. 7}$$

Substituting the values of d_2 and a_2 from **Eq. 4** and **Eq. 5** yields:

$$\tan (\beta_2 + \theta_2) = \frac{1 + h_1 \tan \beta_2}{h_1} \quad \text{Eq. 8}$$

Expanding the tangent of two summed angles using trigonometric identities, and simplifying the expression gives the rotation of the lower part as in **Eq. 9**.

$$\theta_2 = \tan^{-1} \frac{1}{h_1 + h_1 \tan^2 \beta_2 + \tan \beta_2} \quad \text{Eq. 9}$$

Similarly, the tangent of total inclination at the upper part of the barrier is given in **Eq. 10** as:

$$\tan (\beta_3 + \theta_3) = \frac{1 + a_3 - d_3}{h_3} \quad \text{Eq. 10}$$

Substituting the values of d_3 and a_3 from **Eq. 4** and **Eq. 6** yields:

$$\tan (\beta_3 + \theta_3) = \frac{1 + h_1 \tan \beta_3}{h_1} \quad \text{Eq. 11}$$

Expanding the tangent of two summed angles using trigonometric identities, and simplifying the expression gives the rotation of the upper part as in **Eq. 12**.

$$\theta_3 = \tan^{-1} \frac{1}{h_1 + h_1 \tan^2 \beta_3 + \tan \beta_3} \quad \text{Eq. 12}$$

It is obvious from **Eq. 9** and **Eq. 12** that the rotation of the upper part is similar to rotation of the lower part in that they both depend on the initial inclination of the corresponding surfaces (β_2 and β_3). The more important observation is that when comparing **Eq. 3**, **Eq. 9**, and **Eq. 12** it will be obvious that the difference is due to the extra terms in the denominator. These extra terms are small compared to h_1 . Also, since all the calculations are based on a unit deformation at the tip, which is exaggerated, the actual angle difference is even smaller in the context of small deformation theory.

Therefore, the detailed YLA equations that will be derived in the coming section are based on rotations with respect to the vertical plane (θ_I).

The internal work along the yield lines is the sum of the products of the yield moments and the rotations through which they act integrated along the barriers height (z-axis). The rotations correspond to $M_z(z)$ and $M_x(z)$ are $\theta_z(z)$ and $\theta_x(z)$ respectively as shown in **Figure 2-5**, and given by **Eq. 13-Eq. 15**.

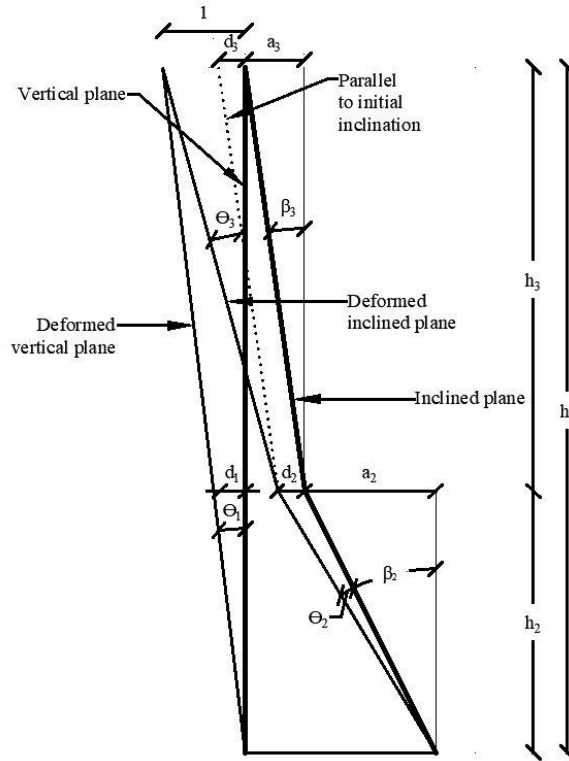


Figure 2-4. Rotations of inclined vs. vertical planes

$$\theta_z(z) = \frac{\delta_z}{x} \quad \text{Eq. 13}$$

$$\delta_z = \frac{H-z}{H} \quad \text{Eq. 14}$$

$$x = \frac{(H-z)}{\tan \alpha} \quad \text{Eq. 15}$$

Substituting **Eq. 14** and **Eq. 15** in **Eq. 13** gives:

$$\theta_z(z) = \frac{\delta_z}{x} = \frac{(H-z) \tan \alpha}{H(H-z)} = \frac{\tan \alpha}{H} \quad \text{Eq. 16}$$

The rotation about the horizontal axis is given as in **Eq. 17-Eq. 20**

$$\theta x(z) = \frac{\delta_x}{z} \quad \text{Eq. 17}$$

$$\delta_x = 1 - \frac{2x}{L_c} \quad \text{Eq. 18}$$

$$x = \frac{(H-z)}{\tan \alpha} \quad \text{Eq. 19}$$

$$\mathbf{1} = \frac{2H}{l_c \tan \alpha} \quad \text{Eq. 20}$$

Substituting **Eq. 18-Eq. 20** in **Eq. 17** gives:

$$\theta x(z) = \frac{1}{H} \quad \text{Eq. 21}$$

Finally, the differential distance along an inclined yield line d_s is given in terms of d_z as in **Eq. 22**.

$$d_s = \sqrt{dx^2 + dz^2} = dz \sqrt{\left(\frac{dx}{dz}\right)^2 + 1} = dz \sqrt{1 + \cot^2 \alpha} = dz \cdot \csc \alpha \quad \text{Eq. 22}$$

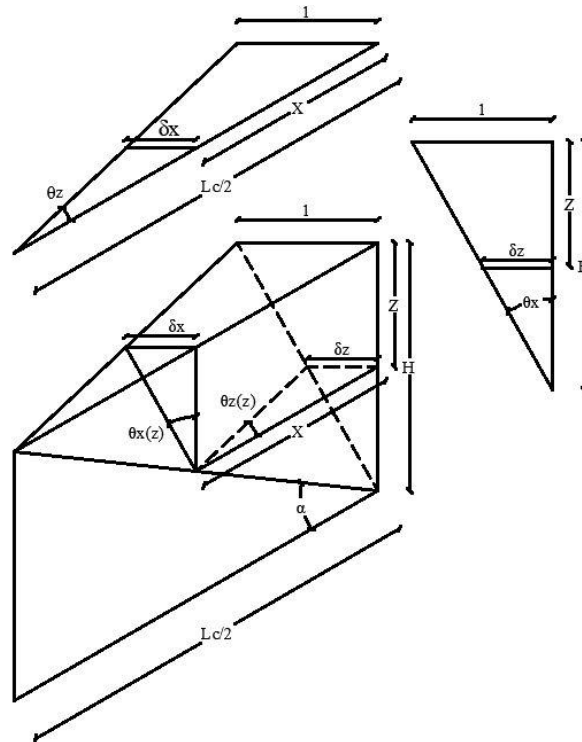


Figure 2-5. Formation of yield lines

2.4.3 Internal work along the yield lines

After obtaining the geometrical relationships of the moments and the rotations along the formed yield lines, the integrals of the internal work along the yield line at the back that is associated with

two rotations around the z-axis (one rotation from each triangular piece) and the two face yield lines are given as in **Eq. 23**.

$$U_i = \int M_{z \text{ back}}(z) \times 2\theta_z \times dz + 2 \int M_{s \text{ front}}(z) \times \theta_s \times ds \quad \text{Eq. 23}$$

Expanding the components of the front face yield lines gives results in **Eq. 24**.

$$U_i = 2 \int M_{z \text{ back}}(z) \times \theta_z \times dz + 2 \int M_{x \text{ front}}(z) \times \theta_x \times ds + 2 \int M_{z \text{ front}}(z) \times \theta_z \times ds \quad \text{Eq. 24}$$

Substituting **Eq. 16**, **Eq. 21**, and **Eq. 22** gives:

$$U_i = 2 \int \frac{M_{z \text{ back}}(z) \times \tan \alpha \times dz}{H} + 2 \int \frac{M_{x \text{ front}}(z) \times \csc \alpha \times dz}{H} + 2 \int \frac{M_{z \text{ front}}(z) \times \tan \alpha \times \csc \alpha \times dz}{H} \quad \text{Eq. 25}$$

2.4.4 External work by applied loads

The length of contact between the vehicle and the concrete barrier is L_t as shown in **Figure 2-6**. Also, the force that is applied by the vehicle is equal to F_t . The value of L_t is specified in (Table A13.2-1) [7].

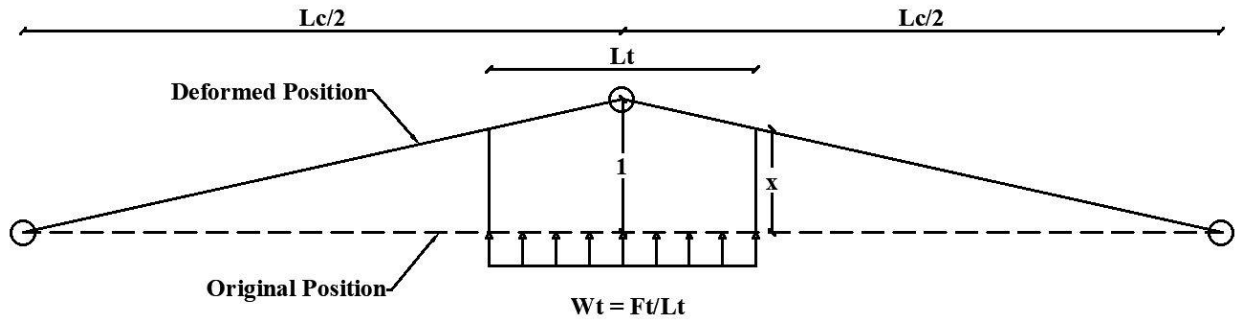


Figure 2-6. Top view for vehicle-barrier contact

The external work is given as in **Eq. 26**.

$$U_e = \frac{1}{2} \times (1 + x) \times \frac{L_t}{2} \times W_t \times 2 = \frac{L_t W_t (1 + x)}{2} \quad \text{Eq. 26}$$

By similar triangles the value of x can be found as:

$$x = 1 - \frac{L_t}{L_c} \quad \text{Eq. 27}$$

Substituting the value of L_t from **Eq. 20** into **Eq. 27**, then back into **Eq. 26** gives the external work as in **Eq. 28**.

$$U_e = L_t W_t \left(1 - \frac{L_t \tan \alpha}{4H} \right) \quad \text{Eq. 28}$$

When the load acting on the barrier is concentrated at one point (the tip of the barrier), **Eq. 28** will become as in **Eq. 37**.

$$U_e = F_t \times 1 = F_t \quad \text{Eq. 29}$$

Where the load F_t is the applied concentrated load.

Equating the internal work from **Eq. 25** with the external work from **Eq. 28** or **Eq. 29** yields the solution for W_t and F_t as in **Eq. 30** and **Eq. 31** respectively. A closed form solution for **Eq. 30** and **Eq. 31** can be obtained by solving $\frac{dW_t}{d\alpha} = 0$, $\frac{dF_t}{d\alpha} = 0$ respectively. Another approach is to plot their functions against a range of angles for α and selecting the minimum value of W_t in **Eq. 30** and F_t in **Eq. 31**. This approach will be used in the case study section of this research.

$$W_t = \frac{2 \left(\int M_{z \text{ back}}(z) \times \tan \alpha \times dz + \int M_{x \text{ front}}(z) \times \csc \alpha \, dz + \int M_{z \text{ front}}(z) \times \tan \alpha \csc \alpha \times dz \right)}{H \times L_t \left(1 - \frac{L_t \tan \alpha}{4H} \right)} \quad \text{Eq. 30}$$

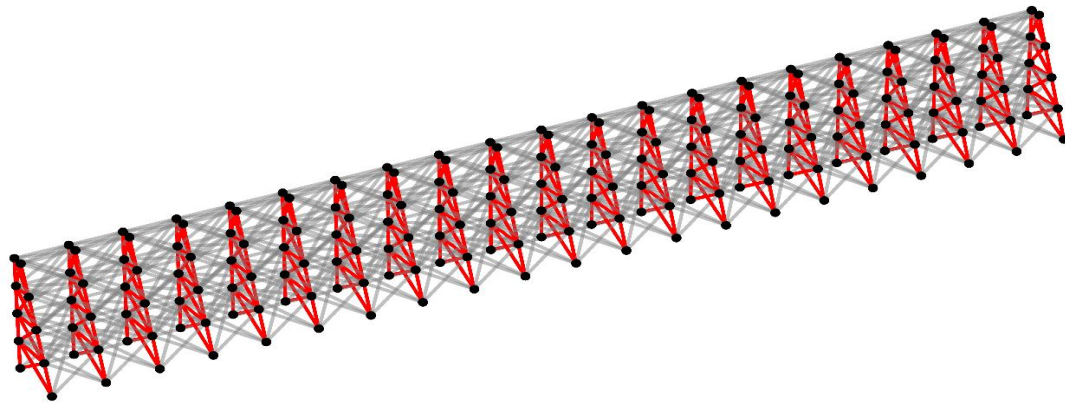
$$F_t = \frac{2 \left(\int M_{z \text{ back}}(z) \times \tan \alpha \times dz + \int M_{x \text{ front}}(z) \times \csc \alpha \, dz + \int M_{z \text{ front}}(z) \times \tan \alpha \csc \alpha \times dz \right)}{H} \quad \text{Eq. 31}$$

2.5 Truss Analogy

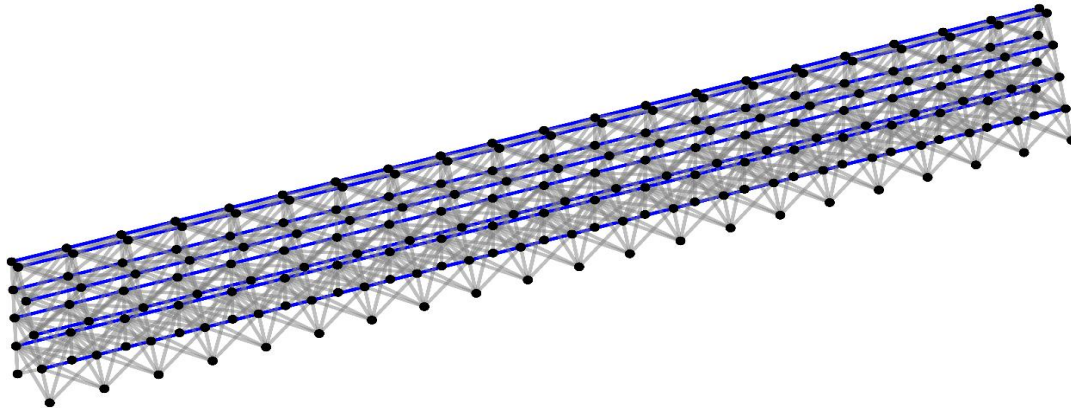
One of the main contributions of this research is to propose an innovative approach for obtaining the ultimate transverse capacity of RC barriers. The approach is based on an equivalent space truss model that has properties defined to mimic the reinforcement of an actual RC barrier. This analogy is different from the AASHTO's YLA presented in section 2.3 in terms of relating the barrier's capacity to the steel reinforcement only and neglecting the contribution of the concrete.

2.5.1 Modeling concept

The model is based on representing the flexural capacity of the barrier about the longitudinal and vertical axes by truss elements. The profile of the truss will have the same shape of the barrier's stirrups. The vertical framing elements of the truss will have an equal area and spacing to the existing stirrups in the actual barrier as shown in **Figure 2-7 (a)**. Longitudinal reinforcement of the barrier is distributed on the truss elements that run longitudinally as in **Figure 2-7 (b)**. Diagonal members were provided to satisfy the barrier's ability to transfer shear forces and diagonal tension.



(a)



(b)

Figure 2-7. Representation of the truss analogy (a) vertical frames for stirrups (b) longitudinal elements for longitudinal reinforcement

2.5.2 Equivalent failure mechanism

The proposed failure mechanism to simulate the failure of the barrier is by tracking the yielding of a vertical or longitudinal bar at a junction node along the first yield line as shown in **Figure 2-8**.

The tracked members are the ones on the front face of the barrier and represent the two yield lines running diagonally. It is worth mentioning that by the time those two yield lines complete formation, the back yield line will be formed. Therefore, the ultimate capacity of the barrier is attached to the completion of those two yield lines. Three important aspects have been observed necessary to get accurate results of the barrier's ultimate capacity by the proposed truss analogy as in the coming paragraphs.

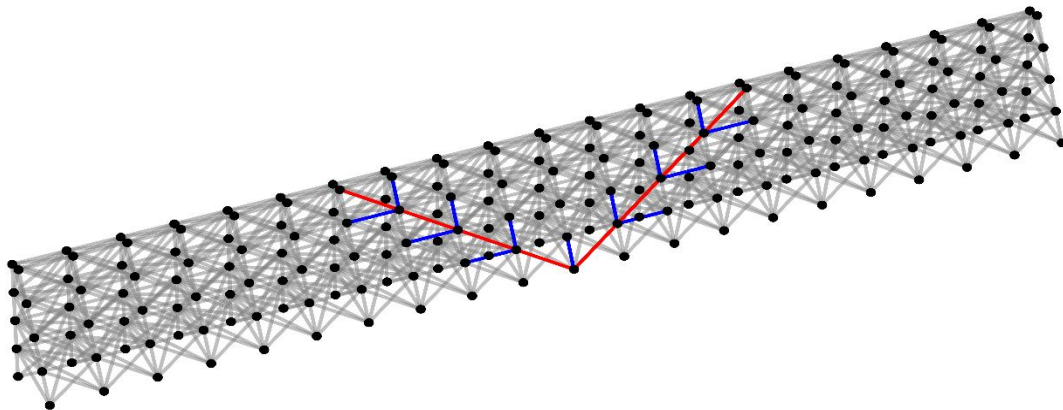


Figure 2-8. Representation of the truss analogy in terms of formation of the first yield line

First, the analysis type needs to be nonlinear to consider the material nonlinearity of the truss elements and capture the progressive failure due to the formation of the yield lines. For this purpose, the authors developed a Python script to read the input files that contain nodes, members, boundary conditions, and loading point. The script then solves the truss using the stiffness method. The load was applied incrementally using small steps. At each load increment, all the members were checked for yielding. The yielded members will have an updated stiffness that will be used when the truss global stiffness matrix is assembled for the next loading step.

Second, the angle of the first yield line, the red line in **Figure 2-8**, is varied to pick the critical angle of inclination that gives the least ultimate load. This variation is obtained through changing the distance between the vertical frames of the truss. However, varying this distance will change the configuration of the actual barrier, therefore, the stiffness of the vertical members is multiplied

by a factor equals to the new spacing divided by the original spacing to maintain the same stiffness distribution of the original barrier model. This factor is sufficient to produce equivalent truss since the members' length and orientation are not affected by the angle variation.

Finally, sizing the diagonal members is a crucial step that affects the results of ultimate capacity. The recommended approach to size the diagonal members is by starting with an area equal to that of the vertical or longitudinal members, then analyzing one loading step while the structure is still all elastic. The purpose of this step is to proportion the magnitude of the load in a diagonal member with respect to the longitudinal and the vertical members that form a joint along the yield line. After that, the stresses are calculated in the vertical and longitudinal members and scaled up to the yielding stress level. Applying Mohr's circle for the two normal stresses and rotating the angle to the desired diagonal orientation, the stress at the diagonal orientation can be found as in **Eq. 32-Eq. 34**. The obtained diagonal stress should be scaled up to the yielding stress by reducing the initial area of the diagonal. For more accuracy, one can repeat the elastic analysis with the updated reduced diagonal area, repeat the stress calculations, and obtain the diagonal area reduction ratio until no change in this ratio is observed.

$$R = \frac{\sigma_{vertical} - \sigma_{longitudinal}}{2} \quad \text{Eq. 32}$$

$$C = \frac{\sigma_{vertical} + \sigma_{longitudinal}}{2} \quad \text{Eq. 33}$$

$$\sigma_{diagonal} = R + C \times \cos 2\alpha \quad \text{Eq. 34}$$

In **Eq. 32-Eq. 34**, α is the angle of the desired orientation from the longitudinal direction.

In the coming section, a case study will be presented and solved by the three previously mentioned procedures. Namely, the current AASHTO's procedure of YLA, the detailed YLA discussed in section 2.4, and the truss analogy model presented in section 2.5. The solution will be verified against numerical analysis of the barrier using Abaqus.

2.6 Case Study

This section will consider the Jersey barrier shown in **Figure 2-1**. The geometry was simplified by merging zone 2 and 3. The design of the barrier was provided by Kansas Department of Transportation (KDOT). The dimensions and reinforcement details are shown in **Figure 2-9**. The concrete compressive strength (f'_c) is 27.6 MPa and the steel yield stress (f_y) is 413 MPa. **Table 2-2** shows the concrete cover and rebar area information to be used in the capacity calculations.

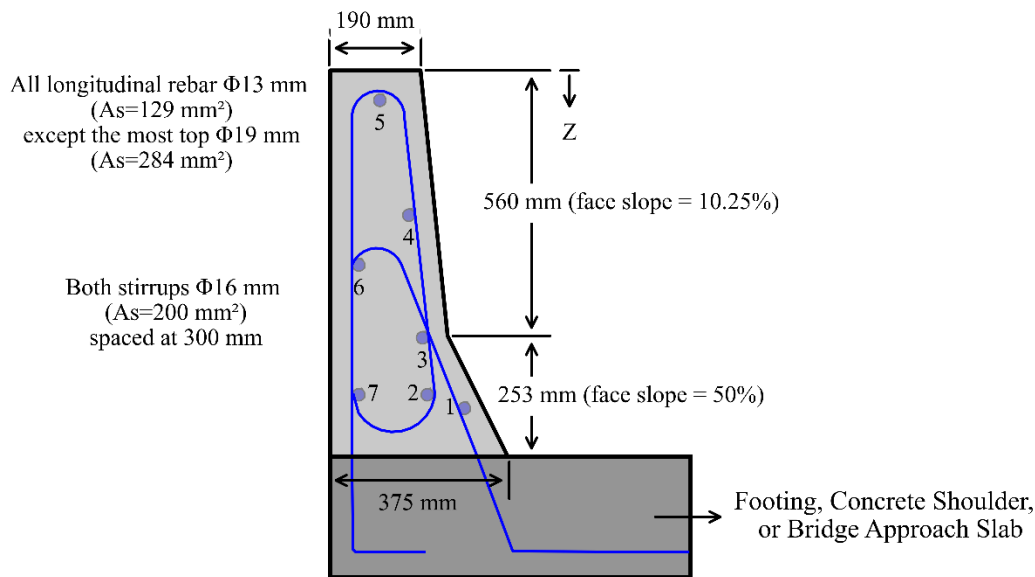


Figure 2-9. Case study barrier (Jersey type)

Table 2-2: Rebar layout in the barrier

	Front side					Back side		
Bar Id (as in Figure 2-9)	1	2	3	4	5	5	6	7
Area (mm ²)	129	129	129	129	284	284	129	129
Cover (mm)	45	122	76	76	107	92	60	60

2.6.1 AASHTO'S procedure of YLA

The equations provided by AASHTO Section 13 are in U.S. customary units. Therefore, the calculations presented in this section will be in the same units. Neglecting the section reinforcement and considering the ultimate moment capacity as the capacity corresponding to the concrete rupture given in **Eq. 35**.

$$f_r = \frac{M_r \times c}{I_g} \quad \text{Eq. 35}$$

Where M_r is the rupture moment, c is the centroid of the section, and I_g is the gross moment of inertia. According to the ACI 318 [20], the rupture modulus is given in **Eq. 36** as:

$$f_r = 7.5 \lambda \sqrt{f'c} \quad \text{Eq. 36}$$

Where λ is 1 for normal weight concrete and $f'c$ is the concrete compressive strength in psi.

Substituting the value of $f'c$ as 4000 psi gives a value of f_r equals to 0.474 ksi. The calculation of the sectional capacity is given in **Table 2-3**.

Table 2-3: Sectional capacity by AASHTO's procedure of YLA

Property	Mz		Mx (for 1ft segment width)		
	Section 1 (Z ₁ = 0-22 in)	Section 2 (Z ₁ = 22-32 in)	Section 1 (Z ₁ = 0 in)	Section 2 (Z ₂ = 22 in)	Section 3 (Z ₃ = 32 in)
b (in)	22	10	12	12	12
h (in)	(7.5+9.75)/2=8.625	(14.75+9.75)/2=12.25	7.5	9.75	14.75
c (in)	4.3125	6.125	3.75	4.875	7.375
Ig (in ⁴)	1176.3	1531.8	421.875	926.86	3209
	118.64 kip.in	129.38 kip.in	53.4 kip.in	90.2 kip.in	206.4 kip.in
Moment	Sum = 248 kip.in = 20.67 kip.ft		Weighted avg. = (0.5(53.4+90.2)22+0.5(90.2+206.4)10)/32 = 95.7 kip.in = 8 kip.ft		

According to Table A13.2-1 in Section 13 of AASHTO [7], the height of this barrier classifies as TL-4 barrier. Thus, the loading will be applied at a distributed distance of $L_t = 3.5$ ft. Substituting the values from **Table 2-3** in **Eq. 1** and **Eq. 2** gives $L_c = 9.39$ ft and $R_w = 56.2$ (250 kN). It is worth mentioning that, according to Table A13.2-1, TL-4 barriers are required to resist a transverse load F_t of 54 kips (240 kN), which this barrier satisfies. However, the coming sections will show that this is not the actual capacity of this barrier.

2.6.2 Rigorous YLA

In this section, the capacity of this barrier will be obtained through the detailed process described in section 2.4. The procedure indicated in **Figure 2-3** will be followed in detail to find the sectional

flexural capacity. According to **Figure 2-9**, there are two regions with different linear variation in terms of the depth (z). Therefore, two sections are defined to find the flexural capacity around the vertical axis (z -axis) at each side (back and front sides). Also, there are three different planes that have distinct geometry and reinforcement, the top of the barrier, the point of slope discontinuity, and the base of the barrier. Therefore, the flexural capacity around the longitudinal axis (x -axis) is represented by the capacity of these planes and will be assumed to vary linearly in between these planes. The detailed calculations of these capacities are shown in the appendix in section 2.8 and summarized in **Eq. 37-Eq. 40**.

$$M_{z_back}(z) = \begin{cases} 29.7 + 0.0284 z \text{ kN} \cdot \frac{\text{mm}}{\text{mm}}, & 0 \leq z \leq 560 \\ 80.6 - 0.1378 (813 - z) \text{ kN} \cdot \frac{\text{mm}}{\text{mm}}, & 560 \leq z \leq 813 \end{cases} \quad \text{Eq. 37}$$

$$M_{z_front}(z) = \begin{cases} 37.35 + 0.0418 z \text{ kN} \cdot \frac{\text{mm}}{\text{mm}}, & 0 \leq z \leq 560 \\ 112.26 - 0.203 (813 - z) \text{ kN} \cdot \frac{\text{mm}}{\text{mm}}, & 560 \leq z \leq 813 \end{cases} \quad \text{Eq. 38}$$

$$M_{x_back}(z) = \begin{cases} 19.6 + 0.1514 z \text{ kN} \cdot \frac{\text{mm}}{\text{mm}}, & 0 \leq z \leq 560 \\ 88.89 + 0.06(813 - z) \text{ kN} \cdot \frac{\text{mm}}{\text{mm}}, & 560 \leq z \leq 813 \end{cases} \quad \text{Eq. 39}$$

$$M_{x_front}(z) = \begin{cases} 19.6 + 0.137 z \text{ kN} \cdot \frac{\text{mm}}{\text{mm}}, & 0 \leq z \leq 560 \\ 78.82 + 0.0684 (813 - z) \text{ kN} \cdot \frac{\text{mm}}{\text{mm}}, & 560 \leq z \leq 813 \end{cases} \quad \text{Eq. 40}$$

Substituting the moment functions from **Eq. 37-Eq. 40** in **Eq. 30** for the distributed load along $L_t = 1067 \text{ mm}$ (3.5 ft) and in **Eq. 31** for a concentrated load at the middle of the barrier gives the integrations as in **Eq. 41** and **Eq. 42**.

$$W_t = \frac{91.19 \tan \alpha + 134.3 \csc \alpha + 121.47 \sec \alpha}{1067 \left(1 - \frac{1067 \tan \alpha}{4 \times 813}\right)} \quad \text{Eq. 41}$$

$$F_t = 91.19 \tan \alpha + 134.3 \csc \alpha + 121.47 \sec \alpha \quad \text{Eq. 42}$$

Solving **Eq. 41** and **Eq. 42** for a range of angles and extracting the minimum values as in **Figure 2-10**, **Eq. 41** gives $W_t = 0.533 \text{ N/mm}$, which when multiplied by $L_t = 1067 \text{ mm}$ results in a capacity

F_t of 569 kN. The capacity F_t under the concentrated load, **Eq. 42**, is 443 kN. While the concentrated load does not represent the vehicular impact force on RC barriers, it is intended to present the solution of this loading case to provide means of comparison when solving the case study using the truss analogy where the loading is concentrated at the center-upper joint.

It can be seen from **Figure 2-10** that the ultimate capacity of the barrier when subjected to a distributed load pattern is more than the double of the capacity obtained using AASHTO's YLA. Also, for the analytical solution, the distributed loading pattern gave about 30% higher than that of the concentrated load. This might be attributed to the smaller region of the barrier engaged in resisting the load in the case of the concentrated load which causes a localized failure. Another observation is that the range of angles providing almost the same barrier capacity was around 20°-30° when the load pattern is distributed, and around 30°-50° under the concentrated load effect, **Figure 2-10**. This indicates that the variation of the yield lines' orientations within these ranges does not have significant influence on the barrier's capacity, even though it alters the range of L_c .

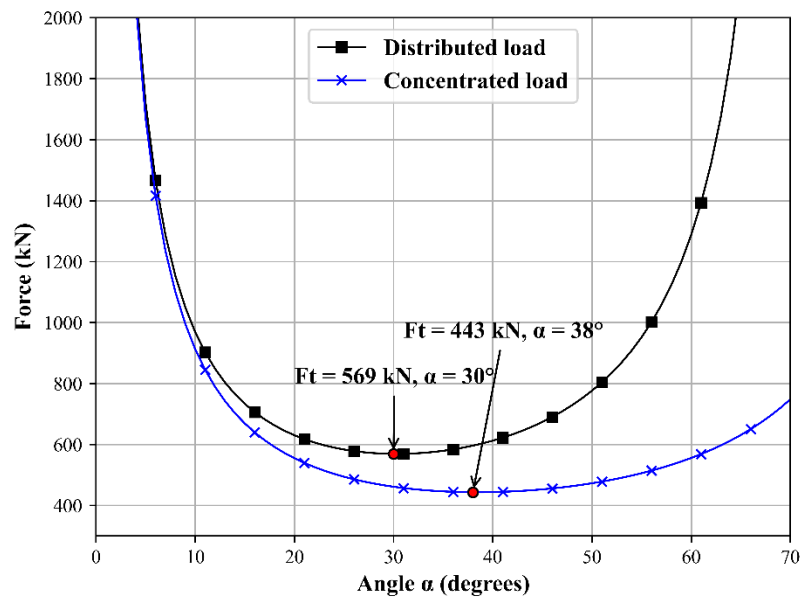


Figure 2-10. Finding the critical capacity using the analytical model

2.6.3 Truss analogy

In this section, the barrier's reinforcement is modeled as an equivalent truss structure that has similar profile shape to the stirrups' cage shown in **Figure 2-11**. The vertical elements have an area equal to the stirrups area (200 mm²) and spaced at 300 mm as shown in **Figure 2-9**. The longitudinal elements have an area equal to the sum of all the front side bars divided by four, the vertical panels, (800/4 = 200 mm²). The truss can be divided into a higher number of vertical panels along its height and yield more accurate results, however, as the number of elements increases so will the complexity of the analysis. An elastic analysis under 1 kN load step with an initial diagonal size of 200 mm² gave vertical and longitudinal stresses of 1.865 and -0.075 MPa respectively. Scaling the higher value to the yielding stress of 413 MPa gives a scale factor of about 221 (413/1.865 = 221.4). That is, when the vertical member reaches the yielding stress of 413 MPa, the longitudinal member will have 221 times the stress of -0.075, which is -16.6 MPa. The negative value indicates that the member is under compression. The angle α is found as in **Eq. 43**.

$$\alpha = \tan^{-1} \frac{180}{300} \approx 31^\circ \quad \text{Eq. 43}$$

Where the 180 mm is the height of the first vertical panel, **Figure 2-11**, and the 300 mm is the spacing of the stirrups.

Applying **Eq. 32-Eq. 34** gives a diagonal stress of 300 MPa as shown in **Figure 2-12**. Therefore, to cause yielding in the diagonal member when the vertical member starts yielding, the area of the diagonal member should be scaled down by a factor of (300/413 = 0.72), which gives an adjusted diagonal area of (0.72x200 = 144 mm²).

Solving the truss using incremental loading of 1 kN step for a range of α angles gives the critical load Ft , **Figure 2-13**, that causes the failure of the truss according to the mentioned criteria in section 2.5.2.

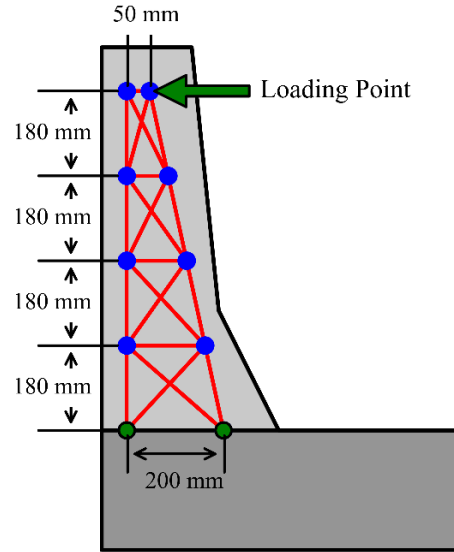


Figure 2-11. Modeling barrier's reinforcement as truss

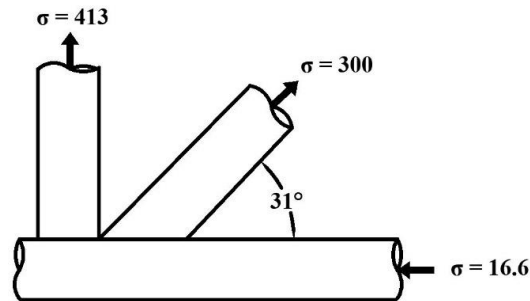


Figure 2-12. Mohr's circle stress calculation for sizing the diagonal elements

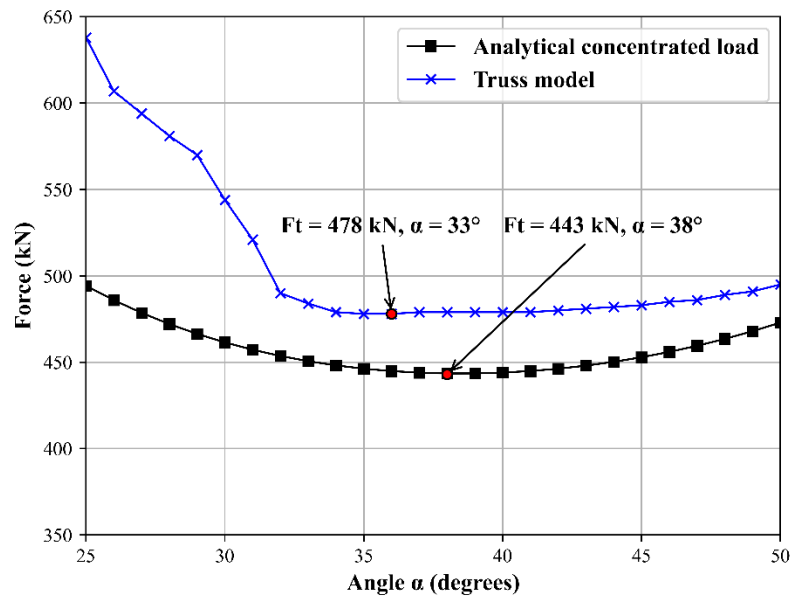


Figure 2-13. Finding the critical capacity using the truss analogy model

The results presented in **Figure 2-13** show the efficiency of the proposed truss analogy model in predicting the ultimate capacity and capturing the behavior of the barrier. The comparison here is made against the analytical model of the concentrated load which is the similar loading pattern to the truss model. The differences between the two models at angles below 32° are attributed to the discrete nature of the truss. The truss model predicted the capacity of the barrier as 478 kN as compared to 443 kN by the analytical model. Although the truss model is less conservative by about 8% of capacity increment, the concentrated loading pattern does not represent the actual vehicular loading, that is distributed, and the predicted value is still below the limit obtained by the analytical model of the distributed load (569 kN). Also, the increment in the capacity of the truss model might be attributed to the presence of the diagonal members in the truss. While these members are important to stabilize the truss and avoid unstable failure mechanisms, they provide means of load transfer beyond the concrete resistance. Nevertheless, the results are promising for further refinement of the currently proposed model and procedure.

2.6.4 Finite element analysis by Abaqus

The final analysis method is the numerical finite element analysis (FEA) using the implicit solver of Abaqus. The model assembly consisted of the barrier (concrete) part and the reinforcement (steel) parts as shown in **Figure 2-14**.

The concrete material was modeled using the concrete damage plasticity model [21] as shown in **Figure 2-15** (a). The steel material was modeled bilinearly with a post yielding modulus of 5% of the initial modulus as shown in **Figure 2-15** (b). The concrete elements had solid sections with an 8-node linear brick element type (C3D8R). To describe the model mesh, the largest concrete element had dimensions in mm as 60x60x40 amounting to a total of 5044 solid elements. The steel elements were modeled as beam elements with profiles correspond to the values in **Table 2-2**. The

barrier's boundary condition was fixed by restraining the translational degrees of freedom of the concrete elements at the base.

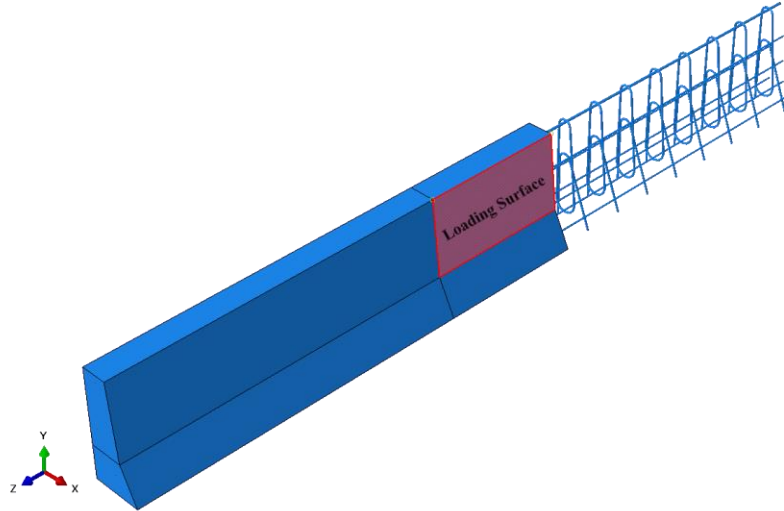
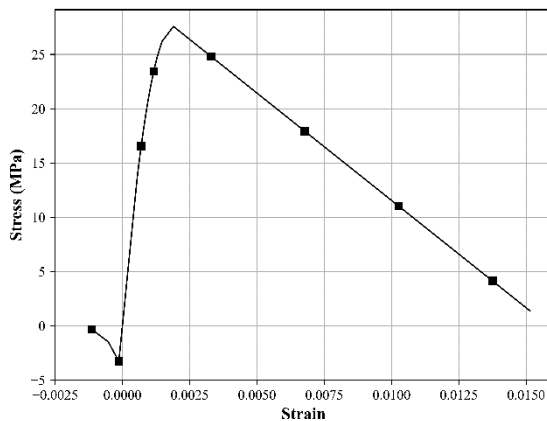
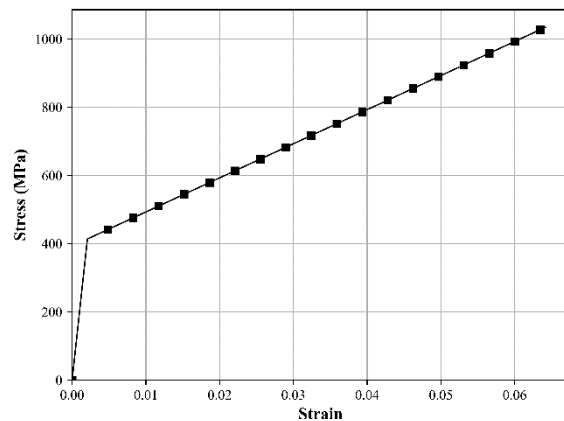


Figure 2-14. Abaqus FEA model of the case study

The analysis type was nonlinear static Riks with the loading defined as a pressure applied at the loading surface shown in **Figure 2-14**. This surface has a length equals to 1067 mm (3.5 ft) and its width extends down until the discontinuity in the barrier's height occurs at 560 mm. The loading surface represents the area that will come in contact with an impacting vehicle in a real crash scenario. The target pressure was set to 1.72 MPa (250 psi); considering the area of pressure application, this is equivalent to a target load of 1027 kN.



(a)



(b)

Figure 2-15. Material models (a) concrete (b) steel

The analysis predicts a proportionality factor of 0.55 of the target load (1027 kN). This results in a peak capacity of $0.55 \times 1027 = 554$ kN as shown in **Figure 2-16**. This result is in perfect agreement with the analytical solution for the distributed load shown in **Figure 2-10**, (569 kN). The displacement in **Figure 2-16** is measured at a node located at the middle-upper edge of the loading surface, (the middle of the entire barrier).

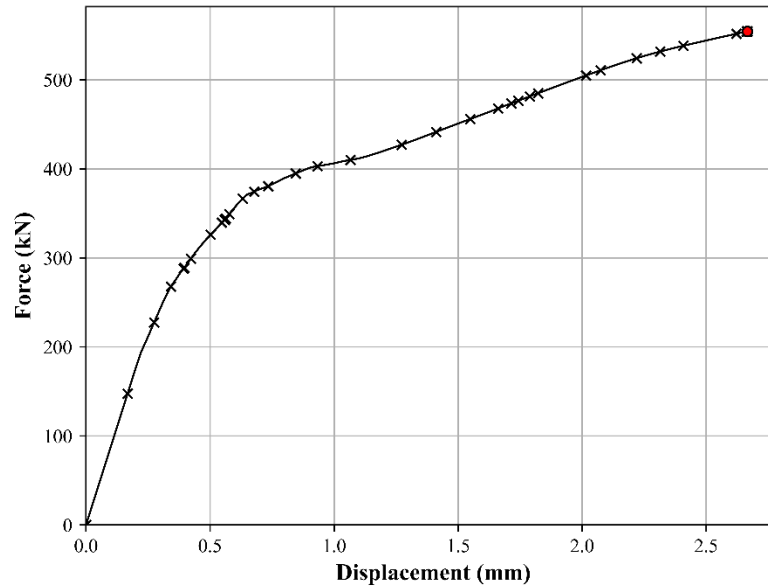


Figure 2-16. FEA load-displacement response of the barrier

Figure 2-17 (a) shows the stress contour map of the barrier's reinforcement at the peak capacity. By connecting the points of the highest stress value in each stirrup, the formation of the yield lines becomes obvious as in **Figure 2-17 (a)**. According to the shown measurements, the angle α is calculated as 31° . The legend of the stress contour shown in **Figure 2-17 (a)** is in psi units, the yield stress of 61,610 psi is equivalent to the 425 MPa. The model showed that yielding happened mainly in the vertical stirrups, which supports the truss analogy model. However, at the ultimate capacity of the barrier, it was observed that stirrups at higher levels did not reach complete yielding within the FEA model. This might be attributed to the complete failure of the concrete material that caused no further load resistance as shown in **Figure 2-17 (b)**. Also, this justifies the lower

capacity when compared to the analytical solution of the distributed load, (554 vs. 569 kN), since the analytical solution considers yielding of the entire line. The deformations shown in **Figure 2-17** are magnified by 50 times.

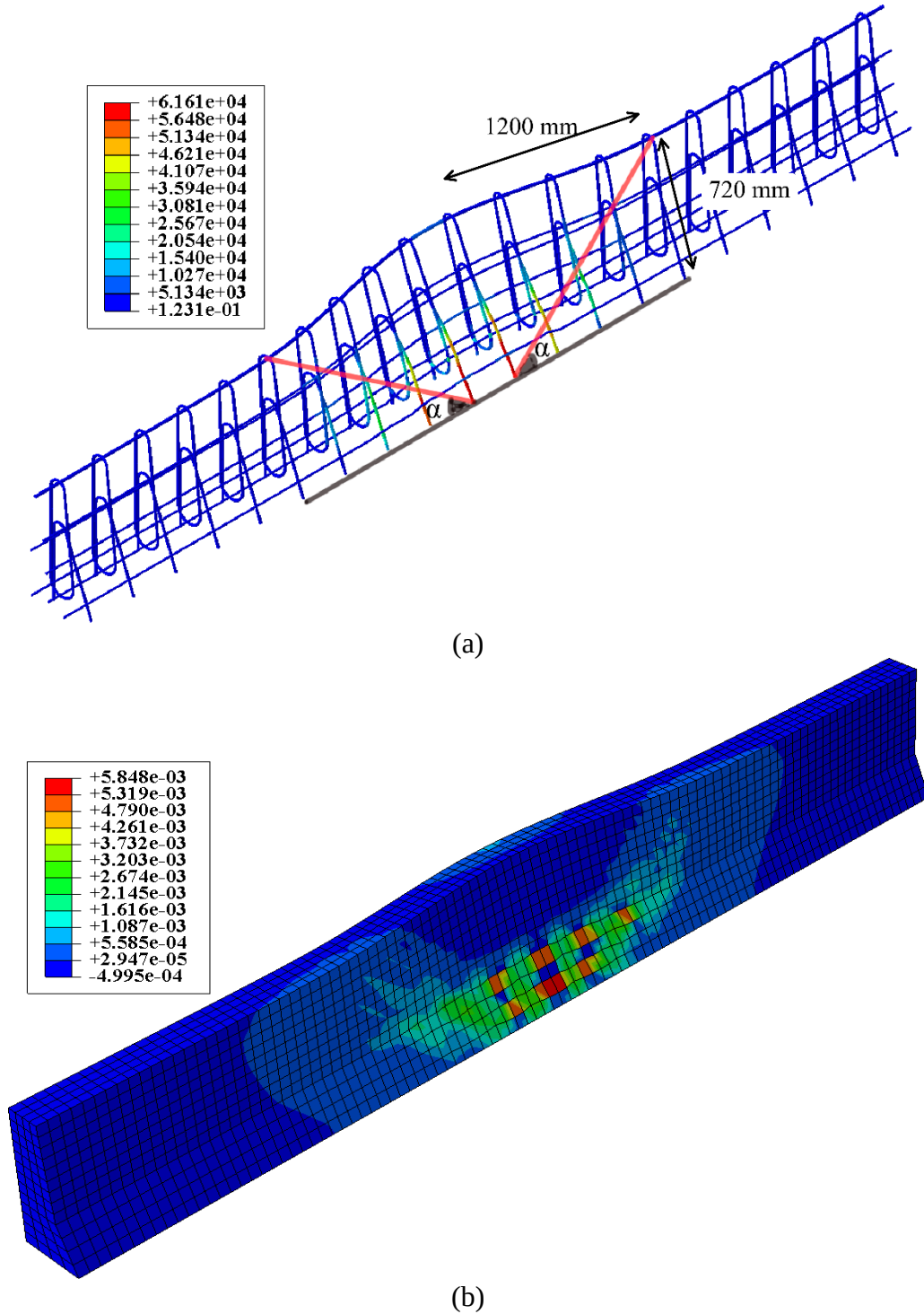


Figure 2-17. (a) Stress profile in the barrier's reinforcement (b) max. absolute principal strain in concrete

2.7 Conclusion and Recommendations

This paper presents several methodologies for obtaining an accurate transverse capacity of RC barriers used as protective structures against VCF. The currently used AASHTO's procedure of YLA was first discussed and the disadvantages of this procedure in term of the capacity underestimation were presented and explained. An alternative rigorous YLA methodology based on the same V-shape failure pattern was derived, and the procedure for implementing this methodology was explained. Furthermore, an innovative truss analogy model was proposed as a new technique for obtaining the transverse capacity of RC barriers by tracking the yielding of certain members along the anticipated yield lines. To verify the presented methodologies, a case study of an RC barrier of Jersey type was analyzed for the ultimate transverse capacity and a comparison was made. The conclusions of this study can be summarized as following:

- 1 The current AASHTO's YLA underestimated the barrier's transverse capacity by more than 50% compared to the detailed YLA and the FEA.
- 2 The proposed detailed YLA is very powerful in obtaining the barrier's capacity under different transverse loading patterns. A perfect agreement between the capacity obtained by this method and the FEA is observed.
- 3 The new truss analogy proposed to obtain the barrier's transverse capacity is capable of capturing the ultimate resistance of the barrier when subjected to a transverse concentrated load. This model showed very good agreement compared to the analytical solution for the same loading pattern with a slight overestimation of the capacity. This model can be further refined to better represent the barrier and maybe obtain the capacity under different loading pattern than the concentrated load.

- 4 Based on the results shown in this study, the authors recommend making a distinction in the structural adequacy criterion between the barrier's load resistance and its geometrical adequacy of vehicle redirection for a specific TL.

2.8 Appendix

A. 1: Back side sectional capacity around the vertical axis

	Back side	
Step	Section 1 ($Z_1 = 0 - 560$)	Section 2 ($Z_2 = 560 - 813$)
1	$\frac{As}{h} = \frac{284 + 129 + 129}{813} = 0.67 \text{ mm}^2/\text{mm}$	
2	$Avg. Cover = \frac{92 \times 284 + 60 \times 129 + 60 \times 129}{542} = 76.7 \text{ mm}$	
3	$d_0 = 190 - 76.7 = 113.23 \text{ mm}$ $d_{z1} = 113.23 + Slope1 \times Z_1$ $= 113.23 + 0.1025 Z_1$	$d_0 = 190 + Slope1 \times Z_1 - 76.7$ $d_{z2} = 170.7 + Slope2 \times Z_2$ $= 170.7 + 0.5 Z_2$
4	$M_{z1} = A_s f_y \left(d - \frac{a}{2} \right)$ $a = \frac{A_s f_y}{0.85 f'c' b} = \frac{542 \times 413}{0.85 \times 27.6 \times 813} = 11.74 \text{ mm}$ $M_{z1} = 0.67 \times 413 \times \left(113.23 + 0.1025 Z_1 - \frac{11.74}{2} \right)$	$M_{z2} = A_s f_y \left(d - \frac{a}{2} \right)$ $a = \frac{A_s f_y}{0.85 f'c' b} = \frac{542 \times 413}{0.85 \times 27.6 \times 813} = 11.74 \text{ mm}$ $M_{z2} = 0.67 \times 413 \times \left(170.7 + 0.5 Z_2 - \frac{11.74}{2} \right)$
5	$M_{z1} = 29.7 + 0.0283 \times Z_1 \text{ kN.mm/mm}$	$M_{z2} = 45.6 + 0.138 \times Z_2 \text{ kN.mm/mm}$ $M_{z2} = 80.6 - 0.138 (813 - Z_2) \text{ kN.mm/mm}$

A. 2: Front side sectional capacity around the vertical axis

	Front side	
Step	Section 1 ($Z_1 = 0 - 560$)	Section 2 ($Z_2 = 560 - 813$)
1	$\frac{As}{h} = \frac{284 + 129 + 129 + 129 + 129}{813} = 0.984 \text{ mm}^2/\text{mm}$	
2	$Avg. Cover = \frac{107 \times 284 + 76 \times 129 + 76 \times 129 + 122 \times 129 + 45 \times 129}{800} = 89.4 \text{ mm}$	
3	$d_0 = 190 - 89.4 = 100.57 \text{ mm}$ $d_{z1} = 100.57 + Slope1 \times Z_1$ $= 100.57 + 0.1025 Z_1$	$d_0 = 190 + Slope1 \times Z_1 - 89.4$ $d_{z2} = 158 + Slope2 \times Z_2$ $= 158 + 0.5 Z_2$
4	$M_{z1} = A_s f_y \left(d - \frac{a}{2} \right)$ $a = \frac{A_s f_y}{0.85 f'c' b} = \frac{800 \times 413}{0.85 \times 27.6 \times 813} = 17.323 \text{ mm}$ $M_{z1} = 0.984 \times 413 \times \left(100.57 + 0.1025 Z_1 - \frac{17.323}{2} \right)$	$M_{z2} = A_s f_y \left(d - \frac{a}{2} \right)$ $a = \frac{A_s f_y}{0.85 f'c' b} = \frac{800 \times 413}{0.85 \times 27.6 \times 813} = 17.323 \text{ mm}$ $M_{z2} = 0.984 \times 413 \times \left(158 + 0.5 Z_2 - \frac{17.323}{2} \right)$
5	$M_{z1} = 37.35 + 0.0417 \times Z_1 \text{ kN.mm/mm}$	$M_{z2} = 60.69 + 0.203 \times Z_2 \text{ kN.mm/mm}$ $M_{z2} = 112.26 - 0.203 (813 - Z_2) \text{ kN.mm/mm}$

A. 3: Back side sectional capacity around the longitudinal axis

Step	Back side		
1	Section 1 (Z ₁ = 0 mm)	Section 2 (Z ₂ = 560 mm)	Section 3 (Z ₃ = 813 mm)
2	$\frac{A_s}{b} = \frac{0}{300} = 0 \text{ mm}^2/\text{mm}$	$\frac{A_s}{b} = \frac{400}{300} = 1.33 \text{ mm}^2/\text{mm}$	$\frac{A_s}{b} = \frac{200}{300} = 0.67 \text{ mm}^2/\text{mm}$
3	$M_{Z1} = \frac{f_r I}{h/2} \frac{1}{300}$ $fr = 0.62\sqrt{27.6} = 3.257 \text{ MPa}$ $I = \frac{bh^3}{12} = \frac{300 \times 190^3}{12} = 171.475 \times 10^6 \text{ mm}^4$ $M_{Z1} = \frac{3.257 \times 171.48 \times 10^6}{190 \times 300/2}$ $= 19.57 \text{ kN.mm/mm}$	$d = 190 + Slope1 \times Z_2 - cover - 0.5 \times \emptyset_{stirrup}$ $d = 190 + 0.1025 \times 560 - 38.3 - 8 = 201.1 \text{ mm}$ $M_{Z2} = A_s f_y \left(d - \frac{a}{2}\right)$ $a = \frac{A_s f_y}{0.85 f_c' b} = \frac{400 \times 413}{0.85 \times 27.6 \times 300} = 23.47 \text{ mm}$ $M_{Z2} = 1.33 \times 413 \times \left(201.1 - \frac{23.47}{2}\right)$ $= 104.27 \text{ kN.mm/mm}$	$d = 375 - cover - 0.5 \times \emptyset_{stirrup}$ $d = 375 - 38.3 - 8 = 328.7 \text{ mm}$ $M_{Z3} = A_s f_y \left(d - \frac{a}{2}\right)$ $a = \frac{A_s f_y}{0.85 f_c' b} = \frac{200 \times 413}{0.85 \times 27.6 \times 300} = 11.74 \text{ mm}$ $M_{Z3} = 0.67 \times 413 \times \left(328.7 - \frac{11.74}{2}\right)$ $= 88.89 \text{ kN.mm/mm}$
4	$slope_{12} = 0.1514$		
		$slope_{23} = -0.0605$	
5	$M_{x_back}(z) = \begin{cases} M_{Z1} + slope_{12} Z \text{ kN} \cdot \frac{\text{mm}}{\text{mm}}, & 0 \leq z \leq 560 \\ M_{Z3} - slope_{23}(\text{height} - Z) \text{ kN} \cdot \frac{\text{mm}}{\text{mm}}, & 560 \leq z \leq 813 \end{cases} \rightarrow M_{x_back}(z) = \begin{cases} 19.6 + 0.1514 Z \text{ kN} \cdot \frac{\text{mm}}{\text{mm}}, & 0 \leq z \leq 560 \\ 88.89 + 0.06(813 - Z) \text{ kN} \cdot \frac{\text{mm}}{\text{mm}}, & 560 \leq z \leq 813 \end{cases}$		

A. 4: Face side sectional capacity around the longitudinal axis

Step	Front side		
1	Section 1 ($Z_1 = 0$ mm)	Section 2 ($Z_2 = 560$ mm)	Section 3 ($Z_3 = 813$ mm)
2	$\frac{As}{b} = \frac{0}{300} = 0 \text{ mm}^2/\text{mm}$	$\frac{As}{b} = \frac{400}{300} = 1.33 \text{ mm}^2/\text{mm}$	$\frac{As}{b} = \frac{200}{300} = 0.67 \text{ mm}^2/\text{mm}$
3	$M_{Z1} = \frac{f_r I}{h/2} \frac{1}{300}$ $f_r = 0.62\sqrt{27.6} = 3.257 \text{ MPa}$ $I = \frac{bh^3}{12} = \frac{300 \times 190^3}{12} = 171.475 \times 10^6 \text{ mm}^4$ $M_{Z1} = \frac{3.257 \times 171.48 \times 10^6}{190 \times 300/2}$ $= 19.57 \text{ kN.mm/mm}$	$d = 190 + \text{Slope1} \times Z_2 - \text{cover} - 0.5 \times \phi_{stirrup}$ $d = 190 + 0.1025 \times 560 - 54 - 8 = 185.4 \text{ mm}$ $M_{Z2} = A_s f_y \left(d - \frac{a}{2}\right)$ $a = \frac{A_s f_y}{0.85 f_c' b} = \frac{400 \times 413}{0.85 \times 27.6 \times 300} = 23.47 \text{ mm}$ $M_{Z2} = 1.33 \times 413 \times \left(201.1 - \frac{23.47}{2}\right)$ $= 96.2 \text{ kN.mm/mm}$	$d = 375 - \text{cover} - 0.5 \times \phi_{stirrup}$ $d = 375 - 75 - 8 = 292 \text{ mm}$ $M_{Z3} = A_s f_y \left(d - \frac{a}{2}\right)$ $a = \frac{A_s f_y}{0.85 f_c' b} = \frac{200 \times 413}{0.85 \times 27.6 \times 300} = 11.74 \text{ mm}$ $M_{Z3} = 0.67 \times 413 \times \left(292 - \frac{11.74}{2}\right)$ $= 78.82 \text{ kN.mm/mm}$
4	$\text{slope}_{12} = 0.137$		$\text{slope}_{23} = -0.0684$
5	$M_{x_front}(z) = \begin{cases} M_{Z1} + \text{slope}_{12} Z \text{ kN} \cdot \frac{\text{mm}}{\text{mm}}, & 0 \leq z \leq 560 \\ M_{Z3} - \text{slope}_{23}(\text{height} - Z) \text{ kN} \cdot \frac{\text{mm}}{\text{mm}}, & 560 \leq z \leq 813 \end{cases} \rightarrow M_{x_front}(z) = \begin{cases} 19.6 + 0.137 Z \text{ kN} \cdot \frac{\text{mm}}{\text{mm}}, & 0 \leq z \leq 560 \\ 78.82 + 0.0684 (813 - Z) \text{ kN} \cdot \frac{\text{mm}}{\text{mm}}, & 560 \leq z \leq 813 \end{cases}$		

Chapter 3 - Assessment of sub-standard concrete barriers as protective structures to bridge piers against vehicular collision force

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3.1 Abstract

Concrete barriers are used for different purposes in the highway inventory. An important purpose is the use of concrete barriers to act as intervening structures protecting bridge piers from the colliding vehicles during crash events. Thus, they are subjected to vehicular collision force (VCF). Different test levels have been specified to help design these barriers to function as expected. According to the American Association of State Highway Transportation Officials (AASHTO) standards, barriers used to protect bridge piers should have a minimum height of 1067 mm and survive test level 5 (TL-5). Many existing barriers that are currently used in roadways do not meet this requirement, hereby, they are referred to as "sub-standard" barriers. While these barriers are found to be inadequate to solely protect bridge piers against VCF, their presence to intervene vehicle-pier collisions will contribute to reducing the severity of the crash effects. Consequently, they reduce the VCF demand that AASHTO specifies for piers to resist when upgrading existing bridges. Therefore, it is important to have a comprehensive understanding about the performance of sub-standard barriers during high level crash events. In this study, the behavior of a representative sub-standard barrier, subjected to TL-4 and TL-5 collision events, was investigated through a matrix of dynamic finite element simulations using the general-purpose FEA program

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LS-DYNA. The quantities of interest were the energy dissipation, velocity reduction, contact force absorption and lateral displacement. The results showed that sub-standard barriers were capable of reducing the values of the parameters studied in the range of 20-45%. Specific reduction values are addressed in detail in the paper.

Keywords: RC barriers, LS-DYNA, Vehicle collision, Dynamic finite element analysis.

3.2 Introduction

Concrete barriers are common and widely used in the roadways. Therefore, they are expected to be subjected to vehicular collision force (VCF). The performance of these barriers depends on several factors including the barrier profile (i.e., cross sectional shape and size), base boundary condition, (i.e., fixed, or free boundary condition as will be discussed in following sections) and the barrier segment length.

The National Cooperative Highway Research Program (NCHRP)-Report 350, [9], provided guidelines for evaluating the safety performance of roadside features, such as concrete barriers. These guidelines contain information about the different test levels that correspond to different applications and the performance acceptance criteria. These levels are functions of the impacting vehicle type and the impact conditions (angle of attack and initial velocity). The acceptance criteria include three factors: structural adequacy, occupant risk and post-impact vehicular response. The last two criteria are concerned with the safety of the vehicle occupants due to the impact and the likelihood of the vehicle resulting in a subsequent collision with another vehicle or fixed object, respectively. The structural adequacy criterion reflects the ability of the barrier to contain and redirect the impacting vehicle or absorb the VCL resulting from the crash event and bring the impacting vehicle to a controlled stop. This criterion is directly related to the performance of the barrier in protecting bridge piers.

These guidelines were implemented by the Federal Highway Administration (FHWA) in 1998. Therefore, the highway community started to follow these guidelines when designing barriers that can meet the performance criteria corresponding to different test levels. In 2009, the American Association for State Highway Transportation Officials (AASHTO) published a Manual for Assessing Safety Hardware (MASH) that contains improvements on the guidelines of the NCHRP-Report 350. In this improved MASH, the requirements for certain test levels were upgraded to reflect more damaging crash events as shown in **Table 2-1**. Currently, FHWA recommendation is to evaluate the protective barriers in accordance with MASH guidelines.

This upgrade in MASH requirements disqualified many existing concrete barriers from being sufficient for the corresponding test level. Lance Bullard et al. [18] reported an experimental study conducted by the NCHRP, the study contained 11 crash tests to evaluate the performance of nine different types of the commonly existing roadside safety hardware, including concrete barriers, under the upgraded MASH requirements. It was reported that five of the 11 crash tests failed to meet the evaluation criteria. Among the failing barriers is the New Jersey shape barrier tested under TL-4 crash event. This barrier is widely used in the existing highway systems due to the fact that most of the barriers were installed before recognizing the minimum height required to survive high test levels. Also, only about 10 percent of the current bridge infrastructure existing in the United States were built after 2009, the year when the first MASH was published, [22]. It is worth mentioning that the experimental program conducted by NCHRP, [18], for this barrier included only qualitative description about its structural adequacy. For instance, no information about the resulting impact force or lateral displacement of the barrier were presented. Therefore, it is important to extensively assess the performance of this barrier under the upgraded MASH requirements for different test levels.

While full-scale testing is neither easy to accomplish nor economically feasible for many agencies, numerical simulations can provide global sense of the crash behavior and yield accurate results for parameters being investigated given that validation against a suitable experimental reference has been accomplished. Therefore, many researchers studied the performance of concrete barriers through numerical simulations. For instance, Itoh et al. [23] simulated a crash event of 20,000 kg truck impacting a median F-Shaped concrete barrier with a clear height of 1100 mm plus 300 mm into the pavement. The barrier segment length, the velocity and the angle of attack were 50 m, 100 km/h and 17° respectively. The barrier successfully passed the structural adequacy criterion and redirected the truck with a maximum lateral displacement of the barrier of about 25 mm and a maximum impact force of about 1000 kN. Atahan [24] simulated a 30,000 kg truck impacting Jersey barriers with several heights (810, 950, 1000, 1050 mm) in order to examine the height effect on the vehicle stability after collision. The angle of attack and impact speed were 20° and 65 km/h respectively. The author concluded that the only height that was able to redirect the vehicle without rolling over the barrier was the 1050 mm. Cao et al. (2020) simulated two crash events for TL-4 and TL-5. The barrier under consideration for TL-4 was single sloped with a clear height of 900 mm, while for TL-5, it was jersey type- barrier with a clear height of 1070 mm. Both barriers were able to redirect the impacting vehicle with a lateral displacement and impact force during TL-4 event of about 13 mm and 550 kN respectively and about 12 mm and 1250 kN respectively during TL-5 event.

While the previous simulations are considered informative for the highway community, the simulated crash events, except for Cao et al. [12] TL-5 event, do not align with a specific test levels as per MASH as shown in **Table 2-1**. Also, none of the previous simulations considered evaluating the performance of the barriers to act as intervening structures protecting bridge piers against VCF.

Rather, the primary focus of most barrier impact studies has been the crashworthiness of a particular barrier design (e.g. the performance of the barrier in re-directing the impacting vehicle). It is important to mention that AASHTO's requirements for the standard barrier used for the purpose of bridge pier protection implies that the barrier has a minimum clear height of 1067 mm and can survive MASH TL-5 crash event. Since many bridges have been constructed prior to the upgraded MASH requirements, many sub-standard barriers, barriers that are less than 1067mm and incapable of surviving the MASH TL-5 crash event, are currently being used for bridge pier protection.

Therefore, a comprehensive study of the behavior of sub-standard barriers used as protective structures against VCF during MASH crash events is necessary. For this purpose, this study considered conducting a matrix of numerical simulations using LS-DYNA. The commonly existing New Jersey shape barrier that was designed prior to the upgraded MASH guidelines is subjected to the current MASH TL-4 crash event (10,000 kg single unit truck traveling at 90 km/h) and MASH TL-5 crash event (36,000 kg tractor-semitrailer traveling at 80 km/h). The quantities of interest (QOI) in this study are the impact force, energy dissipation, velocity reduction and lateral displacement. Based on the results, some discussion about the structural adequacy of the sub-standard barrier is presented. Finally, a case study reflecting the effectiveness of the sub-standard barrier in protecting bridge piers during TL-4 crash event is presented.

3.3 Finite Element Modeling

3.3.1 Single-Unit Truck (SUT)

The finite element model of the SUT vehicle used in this study is Ford F800 shown in **Figure 3-1** (a). It was developed by the FHWA at the National Crash Analysis Center (NCAC) and downloaded from the National Transportation Research Center Inc. (NTRCI) website [25]. The

model was modified by the authors to comply with the current MASH specifications for TL-4. The modifications include increasing the vehicle inertial mass from 8,000 to 10,000 kg and the impact velocity from 80 to 90 km/h. The added mass was through a ballast concealed within the cargo area located directly behind the truck cab as shown in **Figure 3-1 (b)**. The center of gravity (CG) of the ballast is at a height of 1700 mm. The modified model contained 154 parts, 35,000 deformable elements out of which 96% are shell elements, 2.5% solid elements and about 1.5% beam elements. An accelerometer was located at the CG coordinates to measure the kinetic properties of the vehicle during the crash event.

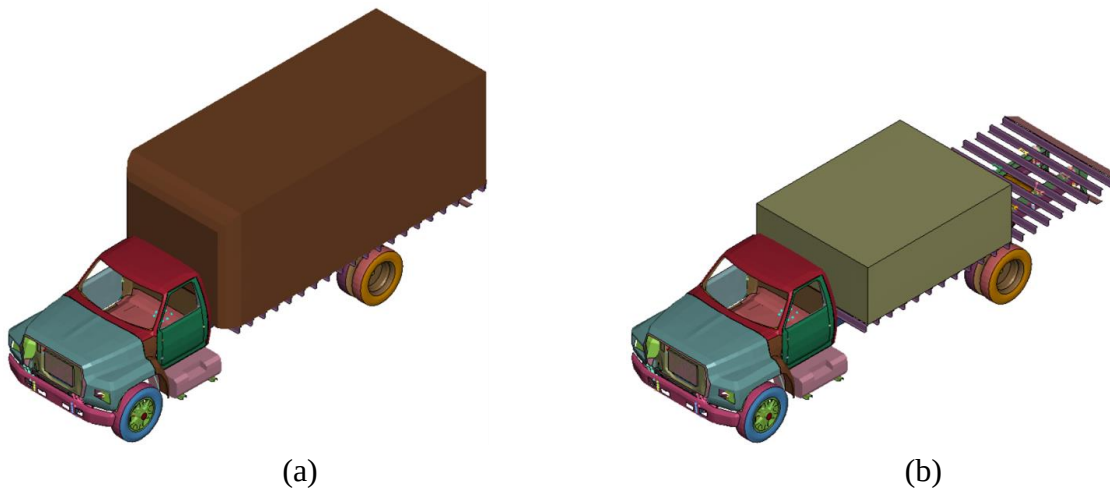
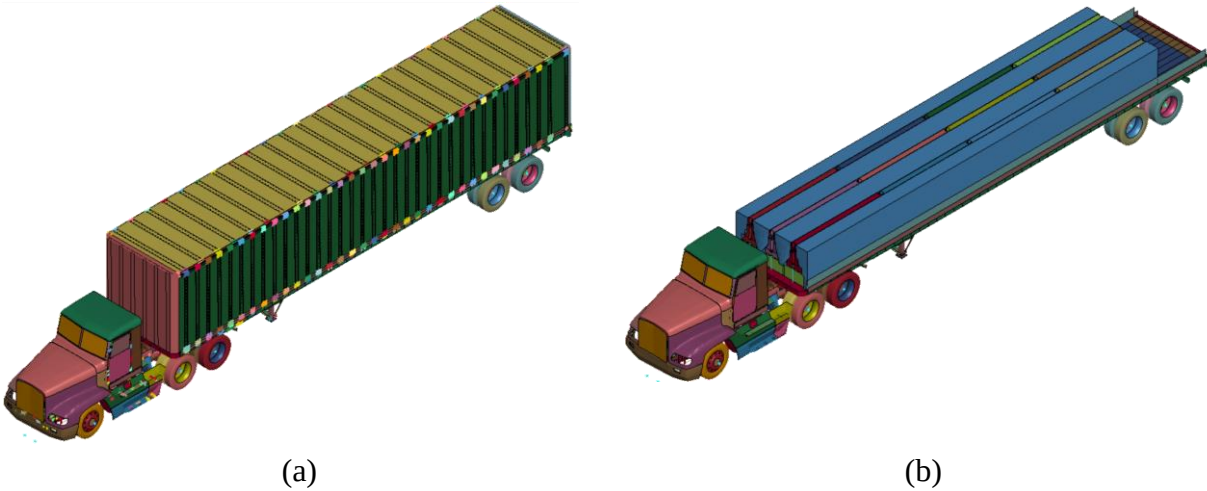


Figure 3-1. (a) Ford F800 SUT finite element model (b) Added mass in the cargo area

3.3.2 Tractor-Semi Trailer (TST)

The TST finite element model used in this study is the simulation of the GMC 1991 vehicle used in test No. TL5CMB-2 [26] and shown in **Figure 3-2 (a)**. The total inertial mass is 36,700 kg including the added ballast shown in **Figure 3-2 (b)** that has its CG located at a height of 1950 mm. The only modification implied was the impact velocity which was decreased from 85 to 80 km/h to comply with the current MASH TL-5 conditions. The model contained 583 parts, 375,540 deformable elements out of which about 80% are shell elements, 20% solid elements and a

negligible percentage of beam elements. An accelerometer was located at the CG coordinates to measure the kinetic properties of the vehicle during the crash event.



(a) (b)
Figure 3-2. (a) GMC 1991 TST finite element model (b) Ballast load

3.3.3 Sub-standard New Jersey Barrier

The design of the barrier model used in this study was provided by Kansas Department of Transportation (KDOT) as in **Figure 3-3**. The barrier model is assembled from three parts: the concrete parapet, the transverse stirrups, and the longitudinal bars. The parapet part is modeled using a constant stress solid element section and the Continuous Surface Cap Model (CSCM) for the material (LS-DYNA MAT_159), [27]. The steel reinforcement has been represented with beam sections with Hughes-Liu element formulation and the piecewise linear plasticity material model (LS-DYNA MAT_024). The reinforcement was constrained within the concrete using a Lagrange-in-Solid constraint. The CSCM concrete material model considers strain rate effects.

To validate the behavior of the concrete material model, a single concrete element was modeled and subjected to tensile and compressive displacement-controlled loading at different rates. The ratio of the concrete dynamic strength to the static strength was compared with experimental studies by Ross et al. [28] for the tensile behavior and Bischoff and Perry [29] for the compressive behavior.

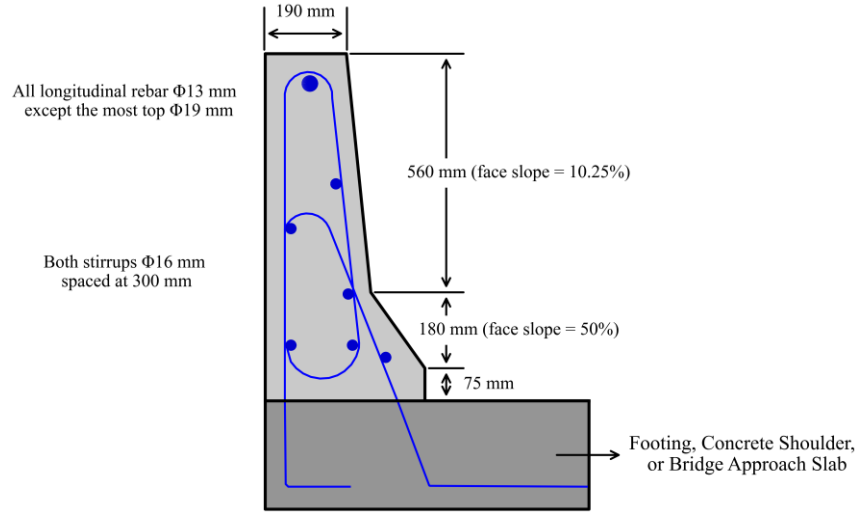


Figure 3-3. Cross-section and details of the barrier design provided by KDOT

The results showed that the default values accurately capture the response of the concrete up to a rate of 1 strain per second, after this, the model underestimates the concrete strength. Therefore, further calibration of tensile and compressive strain rate parameters (n_c and n_t) presented in LS-DYNA material manual [27] and explained in [30] was necessary to adjust the behavior at high strain rates. The concrete stress strain curves generated at various strain rates are plotted in **Figure 3-4 (a)** to illustrate the effect of this parameter. **Figure 3-4 (b)** shows the two banded ranges from the experimental scattered data points reported by Ross et al. [28] and Bischoff and Perry [29] enclosing the present tension and compression curves after calibration.

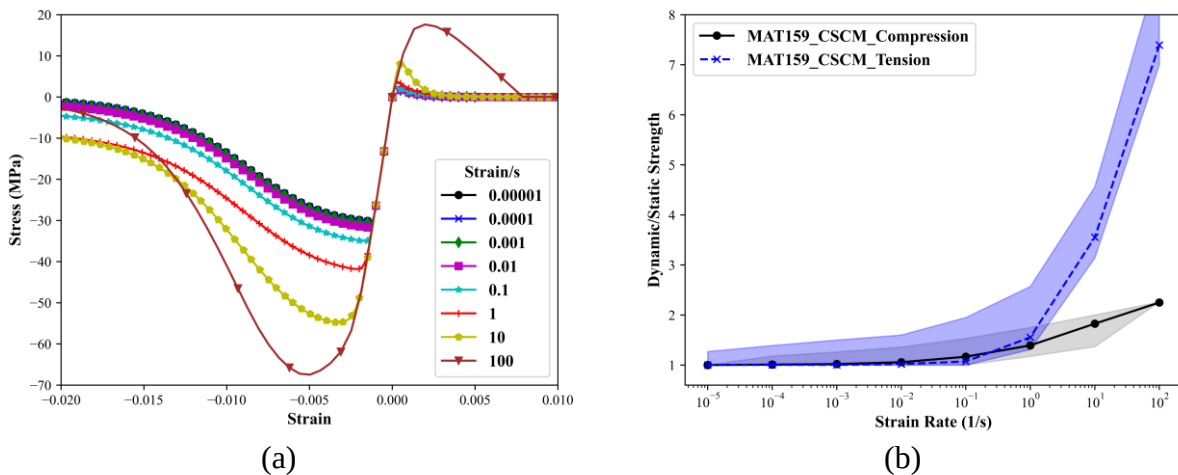


Figure 3-4. Concrete material model (a) strain rate effects (b) model calibration for strain rate

To validate the steel material model, the authors tested a steel coupon in tension. The resulting constitutive relation was then idealized and used to simulate the coupon in LS-DYNA. Complete matching in the constitutive behavior was obtained as shown in **Figure 3-5**. The properties of the barrier materials used in the LS-DYNA simulations are shown in **Table 3-1**.

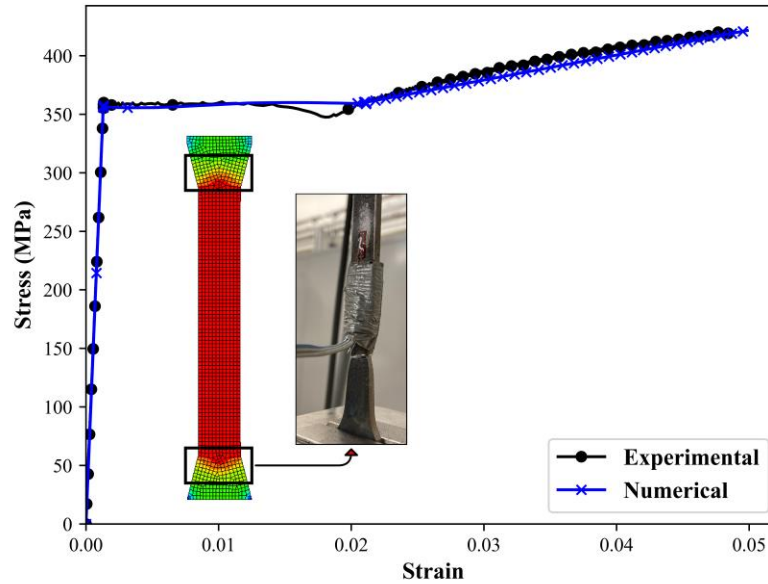


Figure 3-5. Validation of steel material

Two different boundary conditions were considered for the barrier to cover possible construction scenarios. Fixed boundary condition is achieved by fixing all the degrees of freedom of the dowels at the pavement level, **Figure 3-6 (a)**. The free boundary condition represents portable concrete barriers (PCB), where the barrier was modeled as one continuous unit, **Figure 3-6 (b)**, as well as segmented into 3.8 m (12.5 ft) units, **Figure 3-6 (c)**.

Table 3-1: Materials properties

Properties	Concrete	Steel
LS-DYNA model	* MAT_159	* MAT_024
Density (kg/m ³)	2430	7833
Elastic modulus (MPa)	25000	207000
Poisson's ratio	0.19	0.3
Compressive/Yield strength (MPa)	27	400

The connection details between the segments for typical barriers in Kansas were provided by KDOT and these connections were simulated by explicitly modelling rebar hooks with a rigid solid pin, as shown in **Figure 3-6 (c)**.

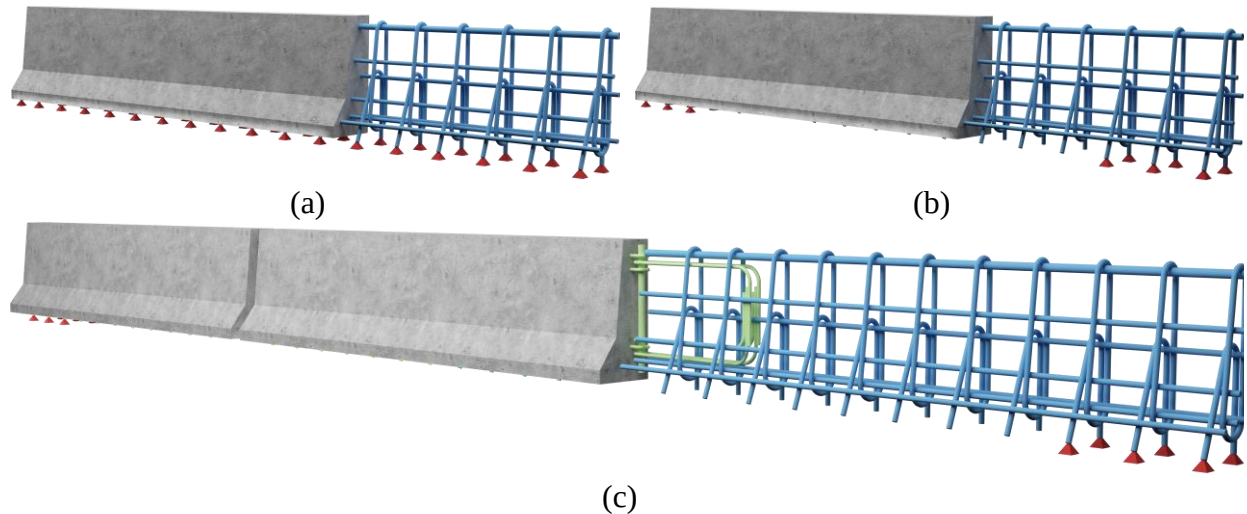


Figure 3-6. Modelling of the barrier's boundary condition (a) Fixed (b) Free (c) Segmented

The pin-to-hook connectivity was accomplished with the `CONSTRAINED_JOINT_REVOLUTE` option in LS-DYNA. The dowels within 1 m distance from each end in the free barriers were fixed at the pavement level to provide edge anchorage as done in the experimental program by [31]. The barrier was free to displace without friction between its bottom face and the ground. This reflects the critical case in terms of the lateral displacement. The location of the first impact point between the vehicle and the barrier corresponds to the location of the critical impact point (CIP), that is the location that maximizes the risk of barrier's failure. The barrier length and the CIP in this study adopted Sheikh et al. experiment [32] as 45 m and 7.5 m respectively. The segmented barrier was slightly longer than 45 m due to the connection distance between the different segments.

3.4 Model Validation and Solution Verification

The models were validated by comparing the results of a simulation against experimental testing and existing simulations in the literature (model-to-model validation). The results of the finite element analysis were verified by examining the system energy balance and energy conservation.

3.4.1 Experimental-Numerical Validation

In order to verify the reliability of the simulation and validate the numerical model, the barrier used in the experimental crash event reported in Sheikh et al. [32] was modeled and assigned the materials properties discussed in section 3.3.3 of this paper. A comparison in the sequence of the crashes at certain time steps (phenomenological events) is presented in **Figure 3-7**.

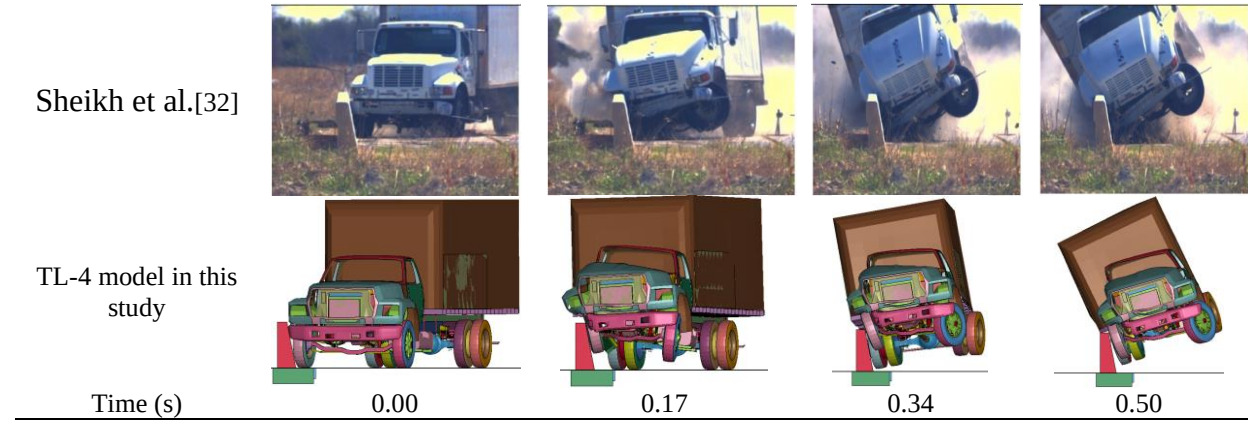


Figure 3-7. Comparing sequence of crash simulation against experimental crash event for TL-4

It can be observed that the vehicle in this study behaved qualitatively the same as in the experimental crash event at various time steps, **Figure 3-7**. The only exception is that, in the experiment, the front wheel stayed in contact with the barrier due to the high friction, this behavior was not fully captured in the present simulation. The friction value between the tires and the barrier was 0.75 as recommended by Trajkovski et al. [33], and between the body of the truck and the barrier was 0.20 as used in Cao et al. [12]. To further support the validity of the simulation, the 50-ms moving average acceleration at the CG that was reported in the experimental crash is plotted against the numerical simulation as shown in **Figure 3-8**. Perfect matching is observed until the rear wheels hit. After that, slight difference was observed and might be attributed to the different characteristics of mass distribution in both vehicles.

Additionally, and to provide other means of validation, a model-to-model comparison is made against the numerical simulations of the experimental crash event that has been conducted by Cao

et al. [12]. It is worth mentioning that the vehicle in Cao et al. [12] simulation was not upgraded in terms of the mass to meet the current MASH requirements.

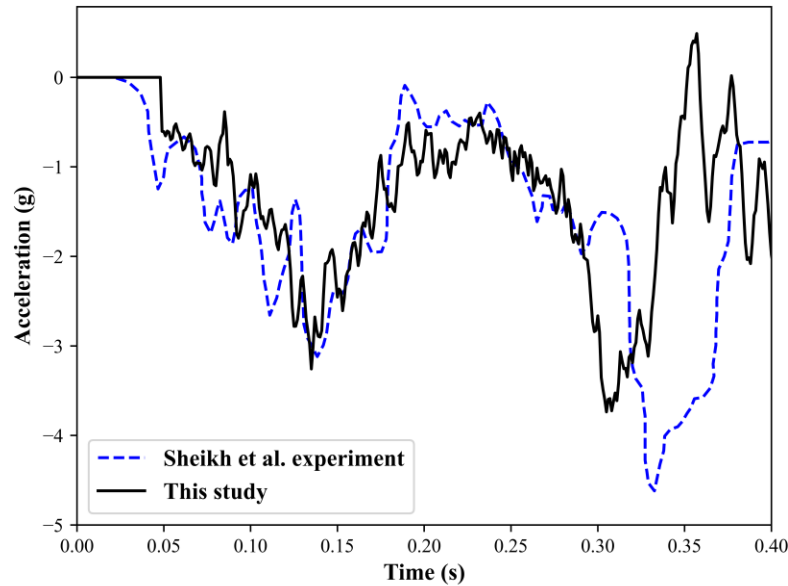


Figure 3-8. Acceleration readings at CG

The contact force resulting from the impact between the vehicle and the barrier for the simulation is shown in **Figure 3-9**. The comparison shows high consistency in the beginning of the crash event when the bumper and the front wheels hit the barrier. The difference in the middle segment (time = 0.1-0.15 s) is attributed to the difference in the vehicular mass. This study resulted in higher contact force when the rear impact occurs (about 700 kN as compared to 500 kN in Cao et al. [12] simulation), this is referred to the contact between the cargo area and the barrier. This contact was avoided in Cao et al. [12] simulation due to the lighter gravitational force, thus the cargo area was able to pass above the barrier. Generally, this simulation was accurately able to capture the peak forces and the overall crash behavior.

Finally, the barrier under consideration in this study and discussed in 3.3.3 was simulated under TL-4 crash event. The damage induced on the simulated barrier was and compared with experimental test for the same barrier as reported by Lance Bullard et al [18].

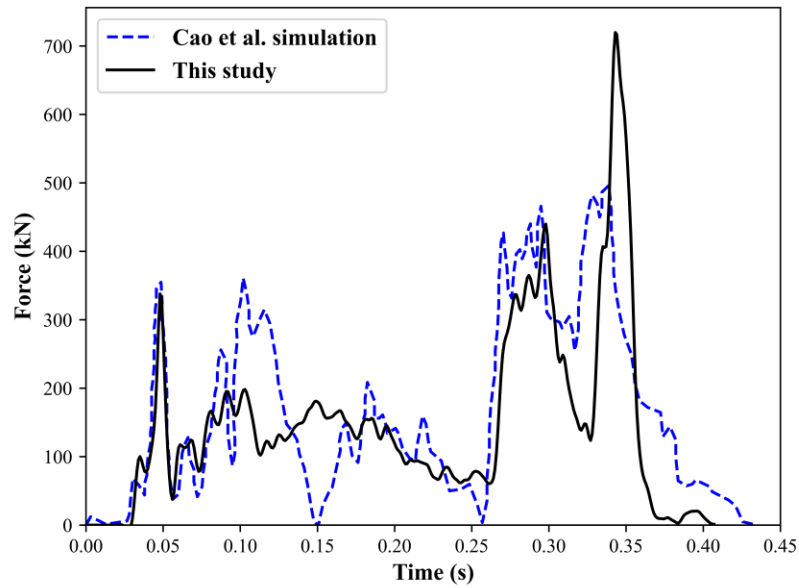


Figure 3-9. Validation by comparing model-to-model contact force results

The damage on the simulated barrier was presented by means of the effective plastic strain in LS-DYNA, this is the plastic component of the rate of deformation tensor. The comparison is shown in **Figure 3-10** and indicates high consistency in the damage locations across the barrier.

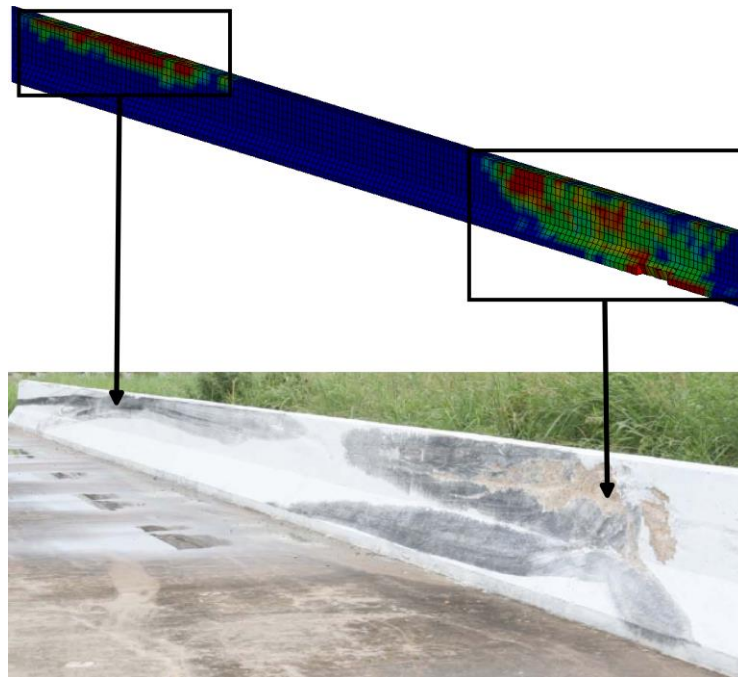


Figure 3-10. Experimental by L. Bullard et al. [18] vs. numerical damage in the barrier

3.4.2 Solution Verification

According to Consolazio et al. [34], monitoring energy components is considered a major step that helps identifying the presence of model errors and numerical defects that may occur in impact simulations. The context of energy monitoring that is used to verify this model is satisfying the energy conservation in the numerical simulation throughout the crash event. The total energy (TE) in the mechanical system exists in the form of kinetic energy (KE), internal energy (IE), hourglass energy (HGE), damping energy (DE), sliding energy (SE), and rigid wall energy (RWE). At every time step during the event, the total energy should be equal to the initial energy in the system (whether it is kinetic and/or internal) plus the external work (W). HGE is a fictitious form of energy that describes the energy consumed in distorting the elements. The ratio of HGE to the TE provides an indicator of whether a simulation is affected by artificial numerical effects, this ratio should be close to zero for an adequate mesh, [34].

Eq. 44 states the energy conservation in a simulated system. Dividing the left-hand side of **Eq. 44** by its right-hand side gives the energy ratio as shown in **Eq. 45**. This ratio should remain close to unity to indicate that the energy is conserved throughout the simulated event.

$$\underbrace{KE + IE + HGE + DE + SE + RWE}_{\text{Total Energy (TE)}} = \underbrace{KE_{\text{initial}} + IE_{\text{initial}}}_{\text{Initial Energy}} + \underbrace{W}_{\text{External Work}} \quad \text{Eq. 44}$$

$$\text{Energy ratio} = \frac{\text{Total Energy (TE)}}{\text{Initial Energy} + \text{External Work}} \quad \text{Eq. 45}$$

Figure 3-11 (a) shows the conservation of the different energy components in TL-4 simulations.

The vehicle starts with a kinetic energy of $KE_{\text{initial}} = \frac{1}{2}mv^2 = 3.2 \text{ MJ}$, where m is the model mass (10,000 kg) and v is the initial velocity (25,000 mm/s). Once the impact event started, the KE transformed to IE, SE, and a very minor part to HGE, DE and RWE. **Figure 3-11** (b) shows the energy ratio for this simulation. It can be seen that there is about 8% loss in the energy ratio

between 0.3-0.4s then the ratio picks up to unity again. This might be attributed to the high impact between the barrier and the rear wheels of the vehicle which happened during this time interval. Since the performance of the sub-standard barrier is also evaluated under TL-5 crash events, it was important to ensure the model accuracy of the TL-5 crash event.

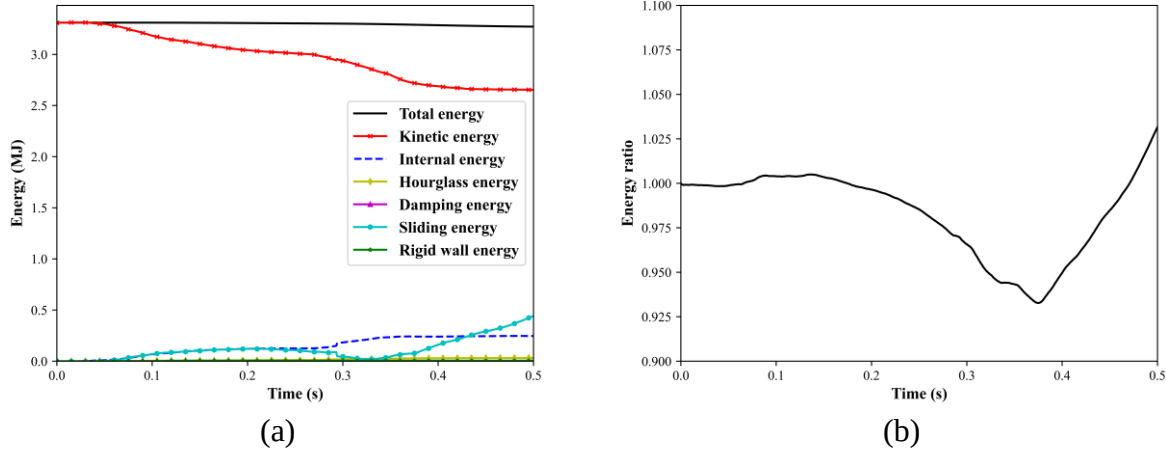


Figure 3-11. TL-4 validation by energy balance (a) Energy components (b) Energy ratio

Figure 3-12 (a) shows the conservation of the energy components in a TL-5 simulation, the initial kinetic energy is $KE_{initial} = \frac{1}{2}mv^2 = 9.1 \text{ MJ}$, where m is the model mass (36,700 kg) and v is the initial velocity (22,250 mm/s). **Figure 3-12** (b) shows the energy ratio for the same simulation which was kept around unity throughout the crash event. Mass scaling was utilized to help improve the overall time-step, and this mass is reflected in the energy calculations and caused $KE_{initial}$ to be slightly higher than 3.2 and 9.1 MJ for TL-4 and TL-5 respectively. Finally, the ratio of the HGE to the TE in both events is less than 1% which is a negligible value that proves the absence of numerical instabilities that might affect the results of this study.

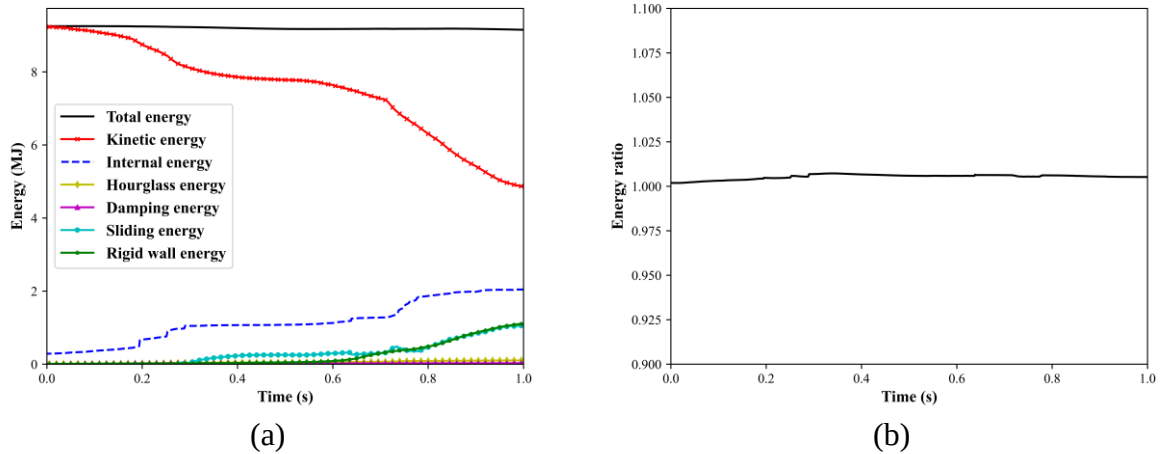


Figure 3-12. TL-5 validation by energy balance (a) Energy components (b) Energy ratio

3.5 Analysis Results and Discussion

3.5.1 Energy Dissipation

Figure 3-13 (a) and (b) show the performance of the sub-standard barrier in dissipating impact energy thus reducing the vehicle's KE and developing IE during TL-4 and TL-5 crash events respectively. In TL-4 crash event, **Figure 3-13** (a), it is obvious that the boundary condition plays a minor role in how much the barrier can reduce the KE. Fixed, and segmented barriers varied slightly and reduced the KE by about 20% and 22.7% respectively (from around 3.3 to about 2.65 and 2.55 MJ). Less reduction is caused by free barriers with about 16.4% (from around 3.3 to about 2.77 MJ). The small amount of KE reduction and the slight difference between the different boundary conditions might be attributed to the less engagement of the barrier with the impacting vehicle. In other words, the body of the vehicle is small and travelling at high velocity causing the vehicle to start overturning and rolling over the barrier before a complete response of the barrier is observed. Therefore, the characteristics of the energy reduction depend mostly on the initial response of the barrier right after the impact. The behavior with respect to the boundary conditions is observed in the TL-5 crash event, **Figure 3-13** (b).

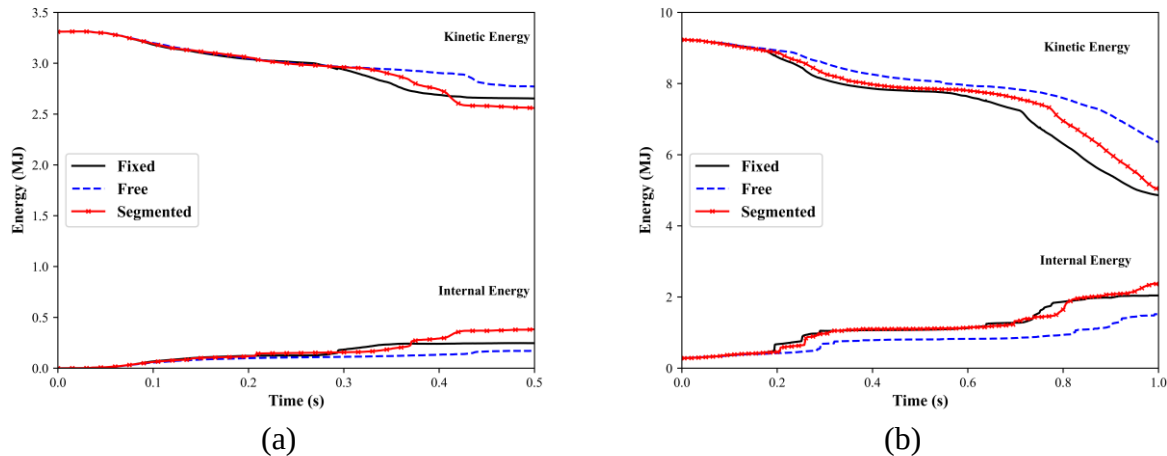


Figure 3-13. Performance in dissipating impact energy during (a) TL-4 (b) TL5 crash events

Fixed and segmented barriers behaved similarly; both reduced the vehicle's KE by about 45% (from around 9.2 to about 5.20 MJ) as compared to about 30% for the free barrier. However, higher percentage in the KE reduction was associated with TL-5 crash event as compared to TL-4, this is referred to the more engagement of the barrier in the crash as the vehicle is larger and travels at lower velocity. Finally, in both test levels, the free boundary condition is the least to reduce the KE. This might be attributed to the high deformability of the free barriers when compared to the fixed ones. Although segmented barriers share the deformability feature with free barriers, they still have rigid connections between the segments that provide additional stiffness for the barrier and absorb more energy. All the reductions in the KE is accompanied with an increase in the IE as a form of energy conservation that was discussed previously and shown in **Figure 3-11** and **Figure 3-12**.

3.5.2 Velocity and Acceleration Reduction

Figure 3-14 (a) and (b) show the performance of the considered sub-standard barrier in reducing the vehicle's impact velocity during TL-4 and TL-5 crash events. Fixed and segmented barriers were able to reduce the initial velocity by about 17% (from 90 to about 75 km/h), and about 14% reduction caused by free barriers during TL-4 event, **Figure 3-14** (a). On the other hand, the initial

velocity was reduced by about 30% (from 80 to about 56 km/h) by fixed and segmented barriers and about 22% by free barriers during TL-5 event, **Figure 3-14 (b)**. This similar trend as observed in the energy dissipation is also attributed to the same reason, that is the stiffness provided by the dowels and the side connections in the case of fixed and segmented barriers respectively. Finally, since the acceleration is the derivative of the velocity, the barrier's performance in reducing the vehicle's acceleration reflects similar behavior. Graphs showing such trends are not included for brevity.

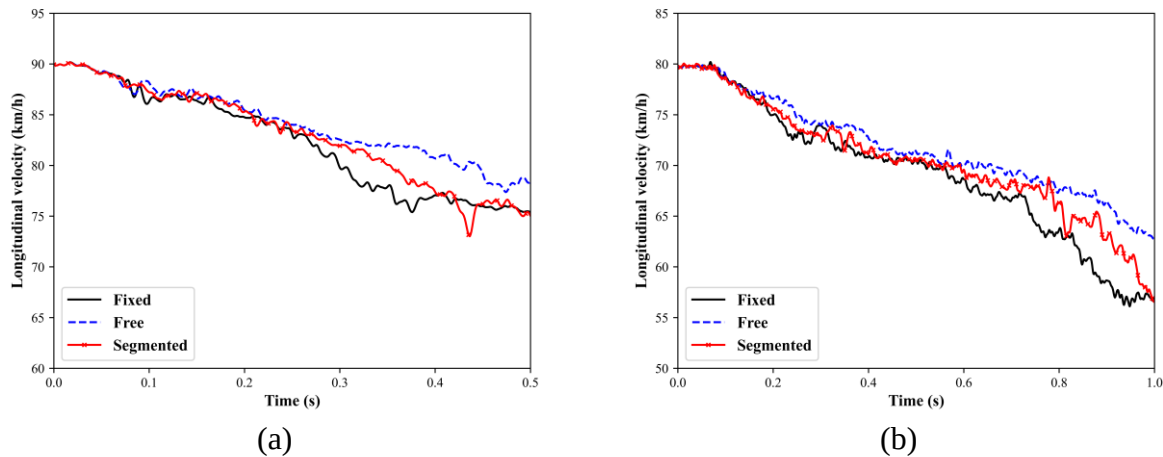


Figure 3-14. Performance in reducing impact velocity during (a) TL-4 (b) TL5 crash events

3.5.3 Lateral Displacement

The displacement history for each node at the upper back edge of the barrier were extracted and the maximum displacement value that was experienced by every node location was recorded to obtain the maximum displacement profile along the barrier. Small negative displacements (displacements toward the vehicle side) were neglected and recorded as zero. **Figure 3-15 - Figure 3-17** show the maximum lateral displacement values and their corresponding locations from the barrier's starting point during TL-4 and TL-5 events.

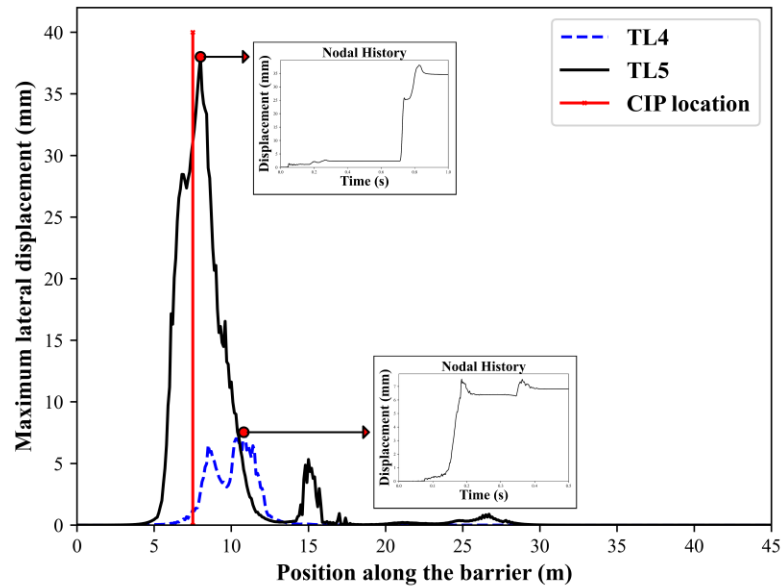


Figure 3-15. Lateral displacement profile along the fixed barrier

The reported locations of the maximum displacement are associated with the CIP of 7.5 m as discussed in section 3.3.3. While the location of impact point in a real crash event is unpredictable, the authors suggest that the location of the maximum displacement along the barrier is relative to that of the CIP. Therefore, it might be useful for engineers to have information about the possible location of the maximum barrier lateral displacement along the barrier length with respect to the impact point as shown in **Figure 3-15 - Figure 3-17**.

It is very clear that the boundary condition has a significant influence on the maximum lateral displacement value and location. For the fixed barrier, **Figure 3-15**, the maximum lateral displacement was about 8 and 38 mm for TL-4 and TL-5 respectively. The location of the maximum lateral displacement for the fixed barrier was at about 11 and 8 m from the edge of the barrier in TL-4 and TL-5 respectively, (3.5 and 0.5 m from the CIP location). The nodal displacement histories for each test level are indicated as sub-plots. For TL-4, it is obvious that the 11 m location was affected by the first contact of the front bumper/wheel, then another similar peak happened when the rear wheels hit the barrier. For the TL-5 event, the maximum

displacement happened at the 8 m location due to the rear wheel hit that happened at late stage of the crash.

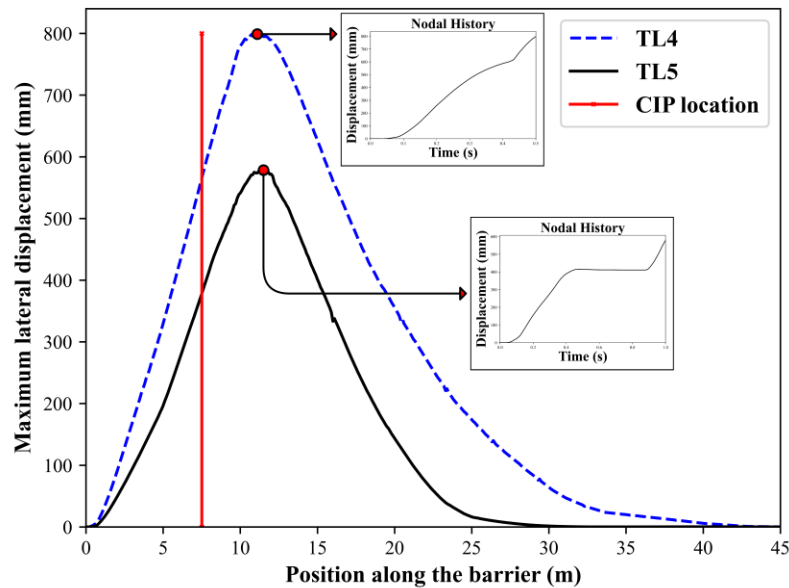


Figure 3-16. Lateral displacement profile along the free barrier

Free barriers displaced much more than fixed barriers with about 800 and 580 mm for TL-4 and TL-5 respectively as shown in **Figure 3-16**. The location of the maximum lateral displacement for both test levels was similar, 11.1 and 11.5 m (3.6 and 4 m from the CIP) for TL-4 and TL-5 respectively. The nodal time histories indicate that as the crashing vehicle keeps in contact with the barrier, the lateral displacement continues to increase. This is expected and attributed to the free boundary condition that allows the barrier to deform.

Finally, segmented barriers were between fixed and free barriers in terms of the lateral displacement with about 400 and 300 mm for TL-4 and TL-5 respectively. The gridlines in **Figure 3-17** represent the segments of the barrier. It can be observed that in TL-4 crash event, since the first segment includes fixed dowels at its end, the location of the maximum displacement occurred at the beginning of the second segment (3.9 m). Also, another peak happened within the third segment due to the rear wheel hit, but this hit produced less displacement. The reason why the

location of the maximum lateral displacement is closer than that of the first impact point (3.9 m compared to 7.5 m) refers to the segmentation of the barrier. This allowed a door-like displacement mechanism at the first connection between the segments. The same behavior was observed in the TL-5 crash event. The only difference is that the rear wheel hit that happened within the third segment (at 9.6 m) was significant to produce higher displacement than that at the beginning of the second segment.

It is important to mention that the reported displacements were a result of an event that has a duration of 0.5 and 1.0 second for TL-4 and TL-5 respectively. While longer simulations are informative, they are hard to achieve due to computational cost and model instability as the crash progresses. However, the selected durations were enough to extract the information about the quantities of interest in this study and commonly used by other researchers and experiments.

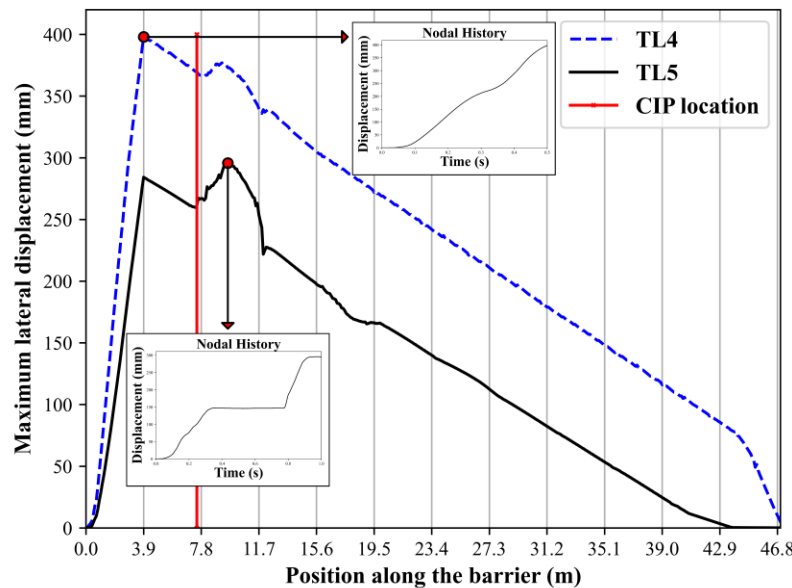


Figure 3-17. Lateral displacement profile along the segmented barrier

3.5.4 Contact Force

This section presents the barrier lateral contact force due to an impact event. This force is in the direction perpendicular to the face of the barrier summed over all the nodes involved in the contact. In other words, it represents the force demand due to contact with the impacting vehicle. This analysis helps to quantify the VCF and design the barriers to be structurally capable of meeting the required demand. AASHTO describes this force in Table A13.2-1 as the transverse force (F_t), [7]. However, the contact force in this section is a result of dynamic simulations, and the design forces of the traffic barriers in Table A13.2-1 are static forces. Estimating the equivalent static force (ESF) that represents the peak impact force (PIF) has been considered by several researchers such as El-Tawil et al. [35], Zhou et al. [36], and Roy et al. [37], to mention some. The transverse VCF in this section is filtered using SAE 60-Hz filter [38]. This filter removes the force spikes that happened over a very short duration due to the high-frequency numerical noise, [12].

Figure 3-18 (a) and (b) show the lateral contact force exerted on the sub-standard barrier during TL-4 and TL-5 crash events respectively. The discussion of the contact force in this section will be categorized into two main events: the first event is when the front wheel/bumper hit the barrier and the second one corresponds to the rear wheel hit. For TL-4, **Figure 3-18** (a), it can be noticed that the boundary condition has minor influence on the value of the force resulting from the first contact event. The barrier was subjected to about 300, 255, and 265 kN in fixed, free, and segmented barriers respectively. This slight difference is attributed to the more stiffness provided by fixed barriers. The same observation is supposed to appear in the second event. However, due to the deformability of the free and segmented barriers, the vehicle maintained lower elevation during the impact which resulted in the rear step bar to come in contact with the barrier and caused spike in the contact force in the case of the free and segmented boundary conditions. **Figure 3-18**

(a) shows pictures of the extra deformation of the step bar in the segmented barrier compared to the fixed one which caused the force spikes. Since this phenomenon is vehicle dependent and might not appear with other crashing vehicles that share the same inertial characteristics but higher CG, it is reasonable to exclude these spikes from the comparison. Thus, one can see that the fixed and segmented barriers resulted in approximately the same contact force peaks of about 310 kN, and much less in the free barriers of about 150 kN.

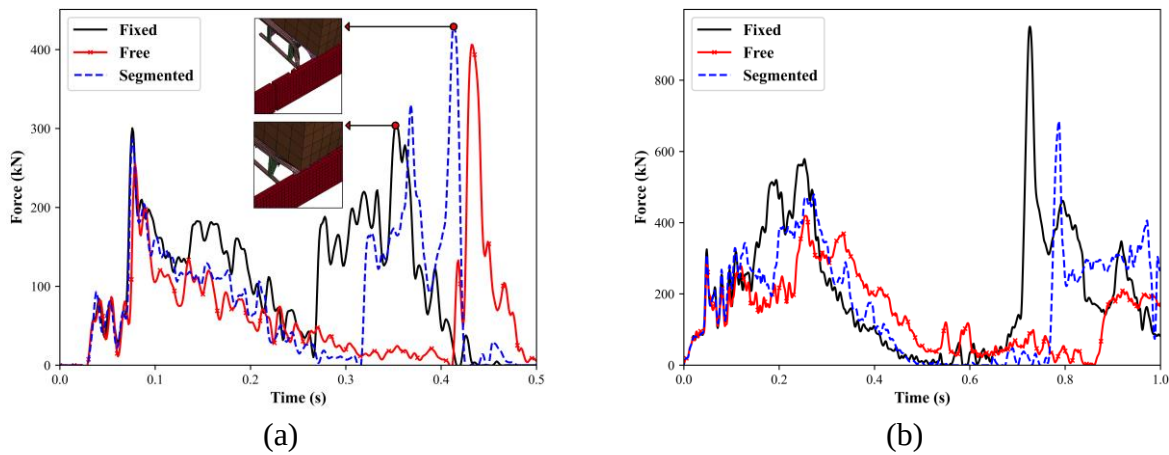


Figure 3-18. Lateral VCF during (a) TL-4 (b) TL5 crash events

Similar behavior but different force magnitudes resulted in TL-5 crash events, **Figure 3-18 (b)**. Fixed, free, and segmented barriers were subjected to about 580, 420, and 480 kN respectively during the first contact event. Since the TST vehicle has higher CG elevation than the SUT, the second contact event resulted in expected forces with respect to the different boundary conditions. Fixed, free, and segmented barriers experienced peak forces of about 950, 210, and 685 kN respectively. This indicates the more influence of the boundary condition for TL-5 crash event as compared to TL-4. Finally, it is also observed in both test levels that the more the available flexibility, the longer it takes for the second impact event to happen.

3.5.5 Structural Adequacy

The MASH summarizes the structural adequacy criterion in Table 5-1A with discussion in Section 5.2.1 [8]. The evaluation criteria emphasize the ability of the test article (the sub-standard barrier) to contain and redirect the vehicle or bring it to a controlled stop. Also, the impacting vehicle should not penetrate or override the installation with acceptable controlled lateral displacement in the test article.

In this study, the simulations showed that fixed, free and segmented barriers subjected to TL-4 and TL-5 crash events were able to redirect the impacting vehicle without any penetration, but with some overriding/rollover at later stages of the impact event. Also, the lateral displacements were controlled in the case of fixed barriers during TL-4 crash event. In all other cases, excessive displacements have been observed. Therefore, judging the effectiveness of sub-standard barriers to protect bridge piers will mainly depend on the location of the pier behind the barrier. AASHTO specifications in Section 3.6.5 specifies a minimum distance of 1 m (3.25 ft) between the top edge of the traffic face of the barrier and the face of the pier component being protected [7]. This means that when there is a tendency of overriding the barrier, and the pier is located within a close distance from the barrier, there will be a possible contact between the cargo of the impacting vehicle and the pier. Also, excessive displacements, especially in free boundary conditions, might reach the piers behind causing another source of loading due to contact with the deformed barrier. To further investigate the effectiveness of the sub-standard barriers as protective devices to bridge piers against VCF, the next section presents a case study of a pier system subjected to TL-4 crash event.

3.6 Case Study

The intent of this section is to provide an insight about the behavior of a sub-standard barrier when compared to a standard one for the same test level, namely TL-4 in this case study. Further research

might be conducted to include higher test levels such as TL-5. A pier system of an existing bridge provided by KDOT has been modeled and placed behind barriers in two TL-4 crash events. The first event included the sub-standard barrier considered in this study and discussed in section 3.3.3. The other event included the standard TL-4 barrier tested in Sheikh et al. [32].

Since the behavior of the segmented boundary condition in terms of the QOI is between fixed and free, only the two extreme boundary conditions were considered in the case study. The distance between the face of the pier and the traffic face of the barrier is 1 m (3.25 ft) as per AASHTO. The center of first pier in the system corresponded to the location of the maximum lateral displacement depending on the boundary condition as discussed in section 3.5.3. A comparison between the effectiveness of each barrier in protecting the piers behind is presented in terms of the damage induced on the piers and the resulting contact force.

The numerical model of the pier system is shown in **Figure 3-19**. It contained the same material properties discussed in section 3.3.3. The translational stiffness transverse to the bridge orientation and the torsional stiffnesses were obtained from the steel girders and deck slab and represented by translational and rotational springs as indicated in **Figure 3-19**. The calculations of the translational and rotational springs stiffnesses are shown in **Figure 3-20** and **Eq. 46 - Eq. 51**. Since the translational stiffness in the direction of the bridge orientation is very high, rollers were placed across the depth of the pier beam to restrict the motion in that direction. The base of the piers was fixed to model the pier piles and the stiff foundations.

Finally, only the dead load calculated from the tributary length of the bridge was uniformly applied as a surface load on the pier beam. The duration of the simulations was increased to 0.8 second to cover potential damages at advanced stages in the crash event. The damage induced on the barriers and the piers was investigated through the effective plastic strain fringe in LS-DYNA.

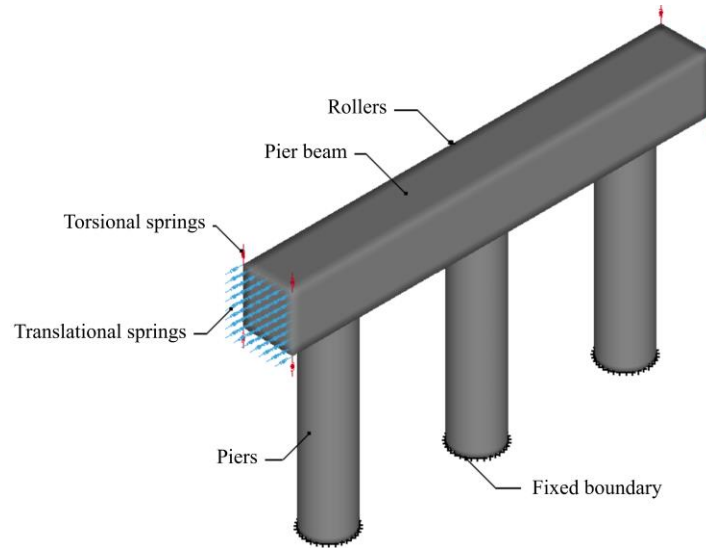


Figure 3-19. Numerical model of the pier system

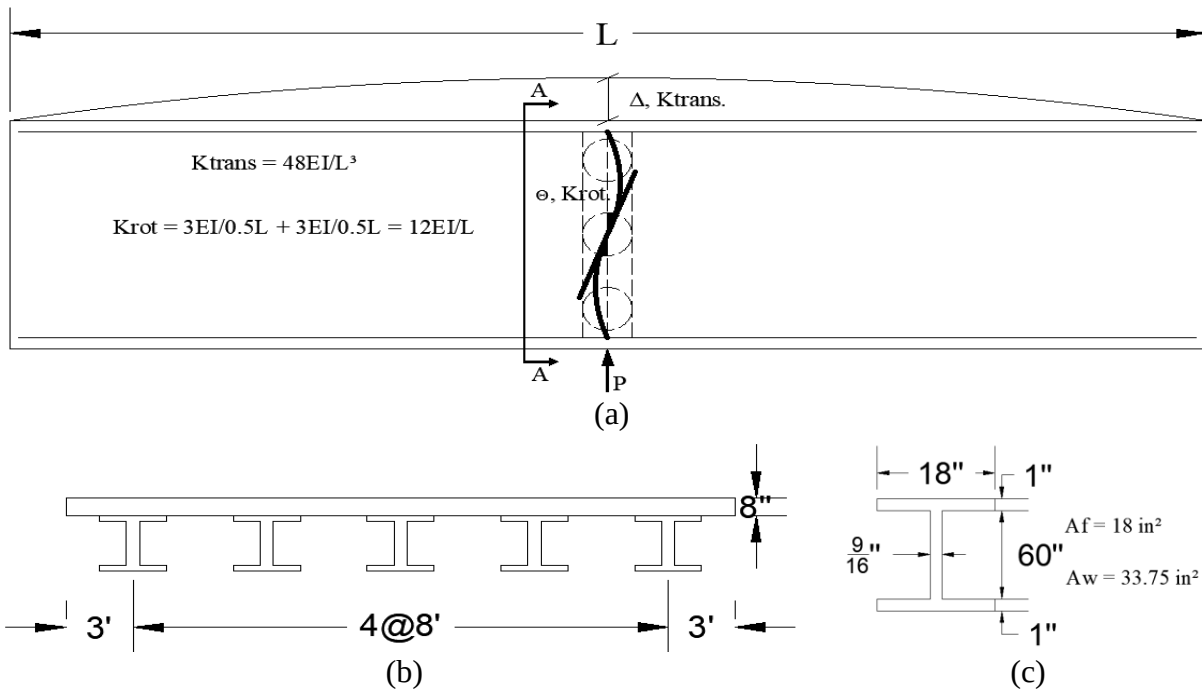


Figure 3-20. Stiffness calculations for numerical model (a) bridge layout (b) section A-A (c) girder detail

$$E_c = 57\sqrt{4000} = 3605 \text{ ksi} \quad \text{Eq. 46}$$

$$I_c = \frac{8 \times 456^3}{12} = 63212544 \text{ in}^4 \quad \text{Eq. 47}$$

$$I_s = \left(\frac{18^3}{12} \times 10 + 4 \times 18 \times 96^2 + 4 \times 18 \times 192^2 + 2 \times 33.75 \times 96^2 + 2 \times 33.75 \times 192^2 \right) = 6433020 \text{ in}^4 \quad \text{Eq. 48}$$

$$EI = EI_c + EI_s = 3605 \times 63212544 + 29000 \times 6433020 = 4.144 \times 10^{11} \text{ k.in}^2 \quad \text{Eq. 49}$$

$$K_{trans} = \frac{48 \times 4.144 \times 10^{11}}{3720^3} = 386.4 \frac{\text{k}}{\text{in}} = 67669 \frac{\text{kN}}{\text{m}} \quad \text{Eq. 50}$$

$$K_{rot} = \frac{12 \times 4.144 \times 10^{11}}{3720} = 1.33 \times 10^9 \frac{\text{k.in}}{\text{rad}} = 150 \times 10^6 \frac{\text{kN.m}}{\text{rad}} \quad \text{Eq. 51}$$

Starting with fixed barriers, **Figure 3-21**, it can be seen that the sub-standard barrier, **Figure 3-21** (a), was heavily damaged at the beginning of the crash event and caused the vehicle's cargo to approach the first pier. This contact was avoided in the case of the standard barrier shown in **Figure 3-21** (b). At a later stage in the event and as the vehicle tends to rollover the barriers, the last pier was hit by the vehicle in both barriers.

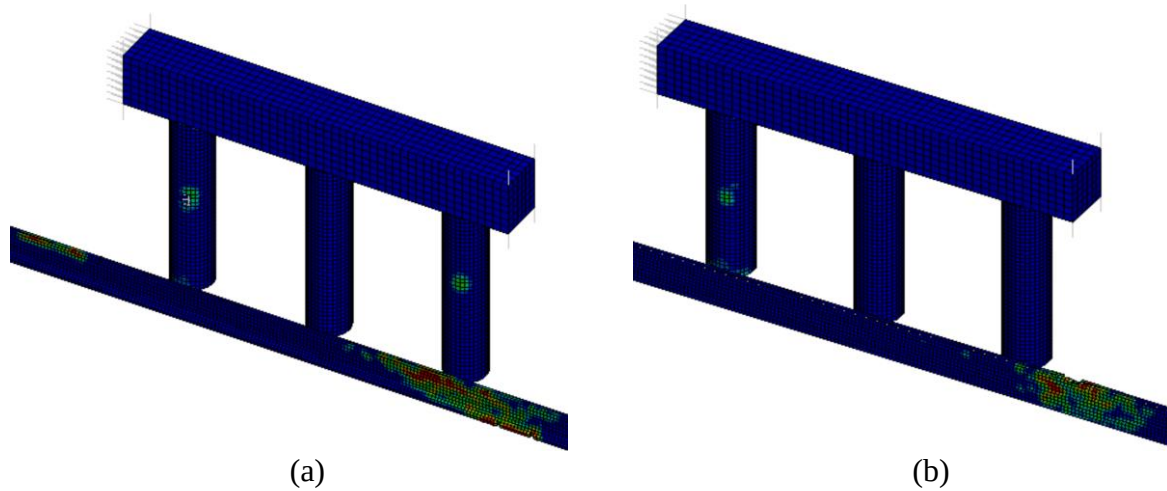


Figure 3-21. Damage on piers protected by (a) fixed sub-standard barrier (b) fixed standard barrier
The VCF demand on the pier system resulted from the fixed barriers is shown in **Figure 3-22**.

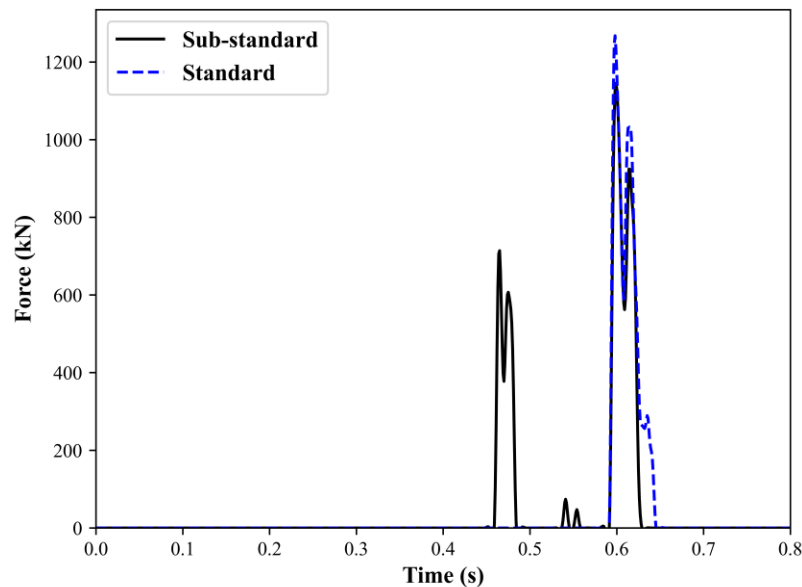


Figure 3-22. VCF on bridge piers protected by fixed barriers

It is shown that about 700 kN was applied on the first pier when protected by the sub-standard barrier and avoided in the case of the standard one. Also, almost around 1200 kN was applied on the third pier in both barriers. In the case of the free boundary condition, both barriers deformed during the crash and the vehicle was able to reach the first pier as shown in **Figure 3-23** (a) and (b). However, the standard barrier was able to reduce the damage on the middle pier when compared to the sub-standard one. The VCF result was about 800 kN in the first pier for both barriers and about 1200 and 750 kN in the middle pier in the presence of the sub-standard and the standard barriers respectively, **Figure 3-24**. The vehicle model was not shown in **Figure 3-21** and **Figure 3-23** to better observe the damage of the elements under consideration.

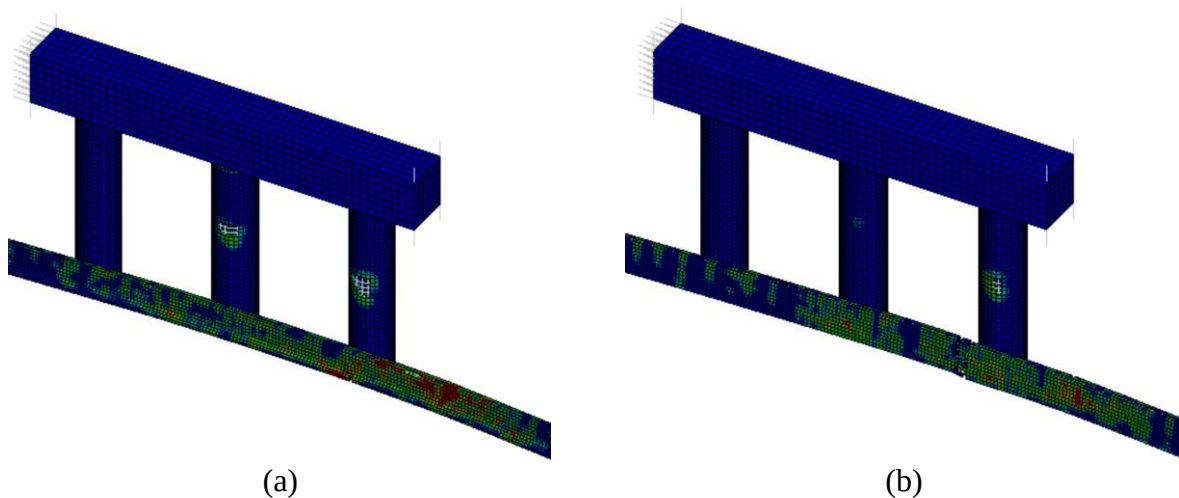


Figure 3-23. Damage on piers protected by (a) free sub-standard barrier (b) free standard barrier

The previous VCF values experienced by the piers are due to TL-4 crash event. Based on the finding of this case study, it is anticipated that the sub-standard barrier will transfer higher VCF values to the piers during TL-5 crash event. On the other hand, when a standard TL-5 barrier is used, the vehicle might clear the pier system with minor to no VCF transferred to the piers, which supports the current requirements of AASHTO specifications (Section 3.6.5). However, although the considered sub-standard barrier is not structurally adequate to solely protect bridge piers and exempt them from being designed or retrofitted to withstand a VCF of 2670 kN as per AASHTO

specifications (Section 3.6.5), the presence of such a barrier will contribute to considerably reducing the required demand on the pier and achieve feasible retrofiting solutions in older bridge infrastructures.

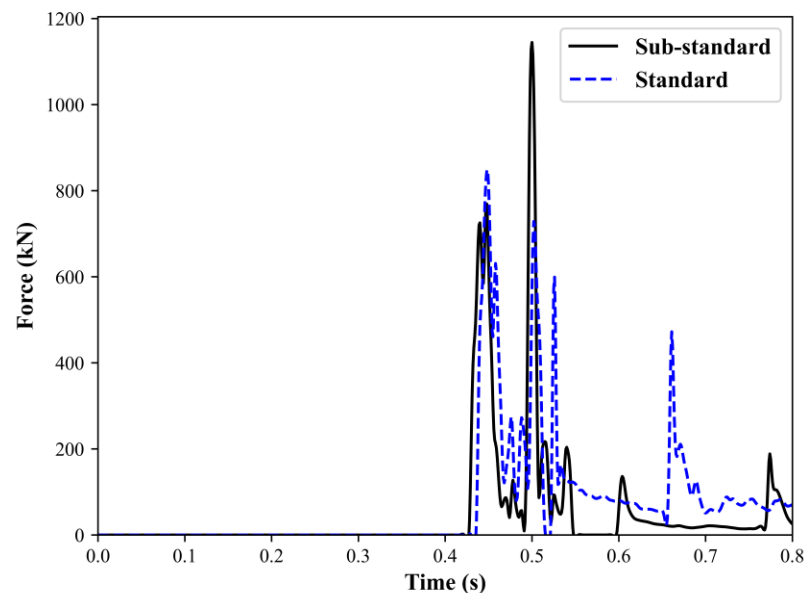


Figure 3-24. VCF on bridge piers protected by free barriers

The peak VCF experienced due to TL-4 crash event was about 1200 kN, that is about 45% of the currently required ESF. This percentage is expected to be higher under TL-5 crash event, but a reduction might still exist, which will give some margin of flexibility for engineers and decision makers in the highway inventory.

3.7 Conclusions and Recommendations

In this study, sub-standard barriers which are available abundantly on the highway inventory were examined to investigate their ability to lessen the level of impact damage and protect bridge piers from VCF. An extensive and detailed simulation matrix revealed very interesting findings regarding the reduction in kinetic energy, velocity of the impacting vehicle, lateral displacement as well as the contact force demand on the considered sub-standard barrier. A case study of pier system was also simulated in the presence of the sub-standard barrier as compared to a standard

TL-4 barrier to examine the effectiveness in reducing the impact effect on the piers behind during TL-4 crash event. The following conclusions maybe drawn:

- 1 The base boundary condition of the barriers plays an important role in influencing some important performance parameters. Specifically, the fixed base boundary condition performs more efficiently than the free base boundary condition in reducing the lateral displacement, while increasing the contact force demand on the barrier and being less economically efficient.
- 2 It has been observed in both test levels that the examined sub-standard barrier redirects the impacting vehicle without complete penetration. However, some overriding and rolling over existed at late stages of the crash event. According to MASH, this type of barrier is classified as structurally inadequate for TL-4 and TL-5 crash events.
- 3 As an intervening structure, being structurally inadequate does not imply the loss of ability to absorb impact forces. Based on the findings of this study, it is recommended that AASHTO specification accepts sub-standard barriers to achieve some level of protection as a contribution to reduce the currently required lateral resistance of bridge piers to withstand VCF.
- 4 It is recommended in future research to extend the current study to involve bridge piers during TL-5 crash events so that a better judgment on the critical VCF transferred to bridge piers in the presence of sub-standard barriers is achieved. Also, varying the distance between the face of the pier and the traffic face of the barrier gives better ideas about the recommended clear distance that minimizes the VCF on bridge piers.

Chapter 4 - Estimation of Vehicular Collision Force on Bridge Piers in the Presence of Sub-standard Intervening Concrete Barriers

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4.1 Abstract

Critical infrastructures such as bridges should be resilient to extreme loading events. Among the potential hazardous events, vehicle collisions are becoming increasingly common. Modern American Association for State Highway Transportation Officials (AASHTO) bridge design specifications consider a limit state to account for vehicular Collision Force (VCF) and require addressing it when bridge piers are within the clear zone defined by AASHTO's Roadside Design Guide. AASHTO specifies two methods to address this limit state: either by providing an intervening structure that is capable of absorbing the VCF and/or redirecting the colliding vehicle or designing the piers to resist a lateral load equivalent to 2670 kN (600 kips). Many old bridges that were designed prior to recognizing the VCF as a limit state fail to meet the modern specifications because either the piers were designed to resist a lower lateral load than the specified limit or the installed intervening concrete barrier is not a standard one that qualifies to solely protect the piers against VCF. This paper introduces a new approach to address the requirements of the modern standards. The approach is based on blended contribution between the available sub-standard barrier and the under designed bridge pier. The proposed approach is based on a

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matrix of dynamic simulations using LS-DYNA and considering different crash scenarios. The results showed that in the presence of the sub-standard barrier, the estimated VCF can be at least reduced by about 25%, leaving 75% to be resisted by the existing pier.

Keywords: Vehicular collision force, Bridge piers, RC barriers, Dynamic FEA, AASHTO

4.2 Introduction and Research Significance

Bridge infrastructures play a major role in connecting communities and providing passage over whatever the bridge is designed to cross. When the purpose of the bridge is to cross roadways, the bridge piers are subject to extreme loading events represented by collision with the vehicles that use the underneath roadway. The severity in the consequences of such an event depends on multiple factors such as the type and velocity of the vehicle involved in the crash as well as the condition of the bridge infrastructure. Over the years, some collision events caused severe damage to the bridges such as pier fracture and complete collapse. Other events caused slight damage such as concrete cracking or spalling at the location of impact [36].

According to the Federal Highway Administration (FHWA), the number of the bridges in the United States in 2022 is 620,699 distributed across the fifty states. The condition of these bridges is classified as good, fair, and poor. This classification is based on the condition of the bridge components including the sub-structural elements such as bridge piers. An estimated 66% of the bridges in the USA are classified as poor-fair condition [22]. This fact increases the risk of bridge failure when the piers are subjected to loading events such as Vehicular Collision Force (VCF).

VCF was first addressed by the American Association for State Highway Transportation Officials (AASHTO) in the 2nd edition of the bridge design specification published in 1998. In that edition, it was required designing bridge piers to structurally resist a lateral static force of 1780 kN (400 kips) to account for VCF [4]. Several studies have then considered evaluating the VCF on

structural elements in an attempt to quantify an equivalent static force (ESF) that can be used for analysis and design purposes. El-Tawil et al. [35] conducted finite element analysis (FEA) of vehicles colliding with a bridge pier and proposed that the 1780 kN is insufficient to represent the impact force in some crash scenarios.

After studying a collection of crash data, analyzing bridge pier collision scenarios through FEA and collecting empirical data of the forces experienced during full-scale collision tests, the research team at Texas Transportation Institute (TTI) recommended increasing the VCF to 2670 kN (600 kips) [5,6]. This value is reflected in the latest AASHTO bridge design specification (Section 3.6.5)[7]. Thus, any upgrades or changes to an existing bridge require addressing the VCF as per the latest AASHTO requirements. Old bridges with poor-fair sub-structures have piers that are often inadequate to satisfy the strength requirement. Such piers are hereby referred to as under-designed piers. Therefore, addressing the VCF by providing structural resistance might require expensive rehabilitations and repairs, and in many cases might not be the most feasible solution.

The alternative solution suggested by the AASHTO provisions (Section 3.6.5.1) is redirecting or absorbing the VCF through providing protection to bridge piers. One of the methods stated by AASHTO to protect bridge piers is the use of a 1070 mm (42 in.) high intervening structure that has its traffic face located more than 1 m (3.25 ft) from the component being protected. Also, the used structure should be able to survive Test Level-5 (TL-5) crash event as specified in the guidelines of the AASHTO's Manual for assessing safety hardware (MASH). The different test levels are indicated in

Table 2-1. While this solution might be convenient for newly constructed bridges, it is very common to find existing bridges that have their piers either located within 1 m distance from the barrier, or the installed barrier is not in accordance with the specifications to pass the acceptance

criteria for TL-5 crash event, which is hereby referred to as sub-standard barrier, or a combination of the two cases (the piers within 1m distance and the installed barrier is sub-standard) as shown in **Figure 4-1**. In this case, both solutions (the structural resistance and the intervening structure) fail to address the VCF as per the recent AASHTO requirements.

Therefore, this study proposes a new approach to address the VCF through a blended contribution between the existing sub-standard barrier and the under-designed pier. This approach is developed and validated through dynamic simulation of an envelope of crash events varying key parameters to investigate the worst-case scenario. Crash events corresponding to TL-4 for single-unit truck (SUT) and TL-5 for tractor-semi trailer (TST) shown in

Table 2-1 are considered in this study. The simulations are conducted using the general-purpose FEA program LS-DYNA.



Figure 4-1. KS 177 + Interstate 70 (S-W) (captured from google maps [39.062, -96.5385])

4.3 Simulation Matrix and Study Phases

The simulation matrix consists of various parameters. Starting with two different vehicles, the single unit truck (SUT) travelling at 90 km/h (56 mph) to represent TL-4 crash event and the tractor semi-trailer (TST) travelling at 80 km/h (50 mph) to represent TL-5 crash event. Then, two barrier boundary conditions were considered, fixed base barrier to represent fixed installation by extending the vertical reinforcing bars into the pavement in addition to free base barriers as in the

case of portable concrete barriers (PCB). The barrier segment length is assumed to be 30 m (100 ft) to comply with an experimental study by the National Cooperative Highway Research Program (NCHRP) - test (4-12) for a similar barrier type [18]. The vehicles were directed to attack the barrier at an angle of 15°. This angle is stated in MASH guidelines shown in

Table 2-1 for TL-4 and TL-5 crash events. Also, AASHTO provisions recommend that the VCF to be applied on bridge piers at angle of up to 15°, [7]. However, the authors decided to expand the simulation matrix to include 25° angle of attack. While the later angle is less likely to happen, specially under bridges, simulating steeper angles is beneficial to compare the load demands on piers relative to AASHTO's 15° scenario to observe the severity of the consequences. Finally, three locations of the pier along the barrier length were considered to investigate the worst possible position that attracts the highest contact force. Since the dynamic simulations demand high computational cost and consume time, and in order to keep the simulation matrix reasonable and achievable, the variation in the pier location along the barrier's transverse direction was not considered in this study. Only the case that represents the worst loading scenario on the pier was considered. Therefore, the pier position along the barrier's transverse direction was set to zero (the pier is just behind the barrier). The first vehicle-barrier contact point was at 2.1 m away from the barrier's edge, and the three positions of the pier were at 1.0, 1.5 and 2.0 m away from the first vehicle-barrier contact point along the barrier length. The parameters of the simulation matrix are shown in **Table 4-1**.

Table 4-1. Parameters of the simulation matrix

Vehicle type	SUT				TST			
Barrier boundary	Fixed		Free		Fixed		Free	
Barrier segment length (m)	30							
Angle of vehicle attack	15°	25°	15°	25°	15°	25°	15°	25°
Pier location with respect to the barrier edge (m)	3.1		3.1		3.1		3.1	
	3.6		3.6		3.6		3.6	
	4.1		4.1		4.1		4.1	

The study consists of two phases, in the first phase the simulation matrix is composed of 24 simulations. A simulation that has the name [SUT_Fixed_30_15_3100] represents a crash event that has the SUT vehicle impacting with a fixed base 30 m long barrier at an angle of 15° with a pier behind the barrier located at 3100 mm from the barrier's edge, see **Figure 4-2**. The objective of this phase is to find the critical dynamic impact force (DIF) based on which an ESF is proposed to account for the VCF in the presence of the sub-standard barrier. In the second phase, the obtained ESF is applied laterally to obtain the internal forces in the pier which will be compared with the pier actual capacity.

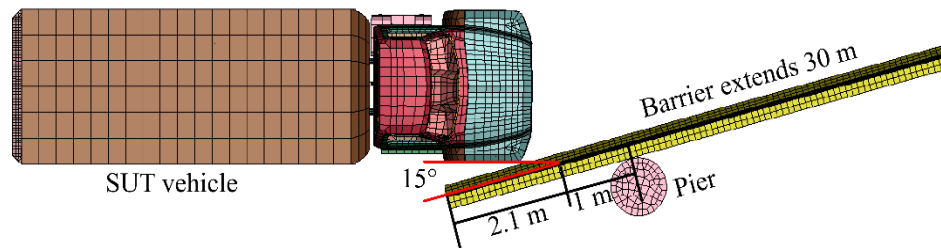


Figure 4-2. Simulation matrix parameters

4.4 Phase 1: Modelling and Results

4.4.1 Single-Unit Truck (SUT)

The finite element model of the SUT vehicle used in this study is Ford F800 shown in **Figure 3-1** (a). It was developed by the FHWA at the National Crash Analysis Center (NCAC) and downloaded from the National Transportation Research Center Inc. (NTRCI) website [25]. The model was modified by the author to comply with MASH specifications for TL-4 as shown in **Table 2-1**. The modifications include increasing the vehicle inertial mass from 8,000 to 10,000 kg and the impact velocity from 80 to 90 km/h. The added mass was through a ballast concealed within the cargo area located directly behind the truck cab as shown in **Figure 3-1** (b). The center of gravity (CG) of the ballast is at a height of 1700 mm above the ground. The modified model contained 154 parts, 35,000 deformable elements out of which 96% are shell elements, 2.5% solid

elements and about 1.5% beam elements. An accelerometer was located at the CG coordinates to measure the kinetic properties of the vehicle during the crash event.

4.4.2 Tractor-Semi Trailer (TST)

The TST finite element model used in this study is the simulation of the GMC 1991 vehicle used in test No. TL5CMB-2 [26] and shown in **Figure 3-2** (a). The total inertial mass is 36,140 kg including the added ballast that has its CG located at a height of 1950 mm above the ground and shown in **Figure 3-2** (b). The only modification implied was the impact velocity which was decreased from 85 to 80 km/h to comply with MASH TL-5 conditions shown in

Table 2-1. The model contained 583 parts, 375,540 deformable elements out of which about 80% are shell elements, 20% solid elements and a negligible percentage of beam elements. An accelerometer was located at the CG coordinates to measure the kinetic properties of the vehicle during the crash event. The vehicle was given 0.3 seconds to run before impacting the barrier so that it can be stabilized under the generated velocity as recommended by the model developers.

4.4.3 Sub-Standard Jersey Barrier and Under-Designed Pier

4.4.3.1 Geometry and Materials

The barrier and the pier models used in this study are designed according to Kansas Department of Transportation (KDOT) as shown in **Figure 4-3** (a) and (b). Karagozian & Case Concrete Model-Release 3 (LS-DYNA *MAT_072R3) was selected for the concrete material model. The Simplified Johnson Cook material model (LS-DYNA *MAT_098) was selected for the steel rebar. All rebars are explicitly modeled as beam elements and the concrete as solid elements. The reinforcement is constrained within the concrete barrier using a Lagrange-in-Solid constraint. Parameters related to the used material models are in the material's LS_DYNA user manual [27].

Details on the concrete material model validation and calibration available in Agrawal et al. [39].

The concrete and steel properties used in the simulations are shown in **Table 4-2**.

Table 4-2. Material properties

Properties	Concrete	Steel
LS-DYNA model	* MAT_072R3	* MAT_098
Density (kg/m ³)	2430	7833
Elastic modulus (MPa)	25000	207000
Poisson's ratio	0.19	0.3
Compressive/Yield strength (MPa)	27	400

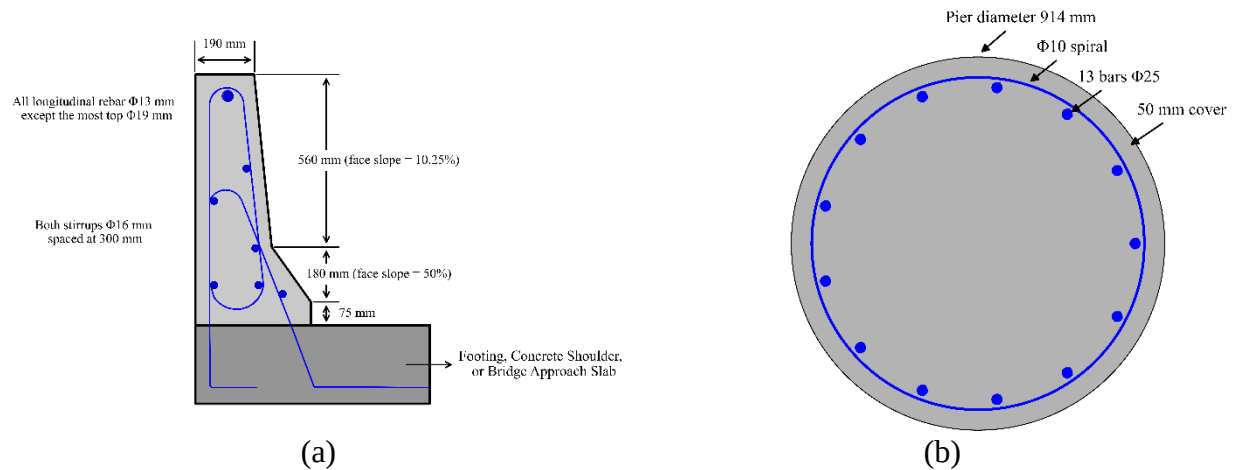


Figure 4-3. Cross-section and details of (a) the sub-standard barrier (b) the under-designed pier as provided by KDOT

4.4.3.2 Boundary Conditions

Two boundary conditions were considered for the barrier, the fixed base boundary is achieved by fixing all the dowels nodes at the base level **Figure 4-4** (a), meanwhile in the free base boundary only the dowels within 1 m distance from the barrier's edges were fixed to provide sort of barrier anchorage, **Figure 4-4** (b). The pier was fixed on both ends to achieve the highest stiffness so that it attracts higher contact force as shown in **Figure 4-4** (c).

4.4.3.3 Standard Requirements

The barrier is considered sub-standard because its clear height is 913 mm (32 in.) which is less than 1067 mm (42 in.) as specified by the AASHTO provisions (Section 3.6.5.1). The pier is subjected to the ESF of 2670 kN (600 kips) as suggested by AASHTO [40].

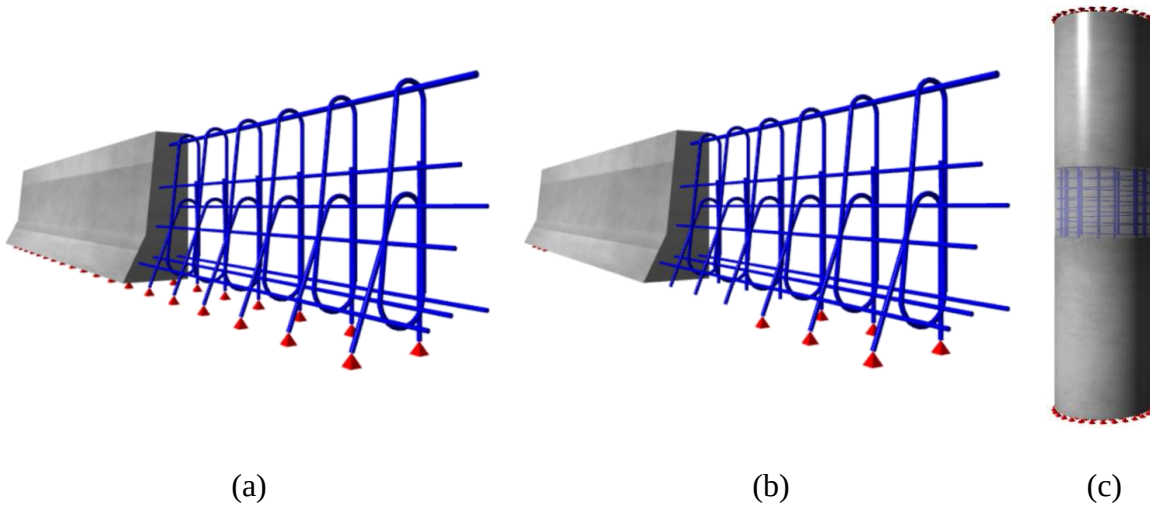


Figure 4-4. Boundary condition (a) Fixed barrier (b) Free barrier (c) Fixed pier

The provisions indicate that this force should be applied at 0.6-1.5 m (2.0-5.0 ft) from the ground level, **Figure 4-5**. The end moments and shears for fixed-fixed boundary shown in **Figure 4-5** are found by **Eq. 52 - Eq. 55** and calculated in **Table 4-3**.

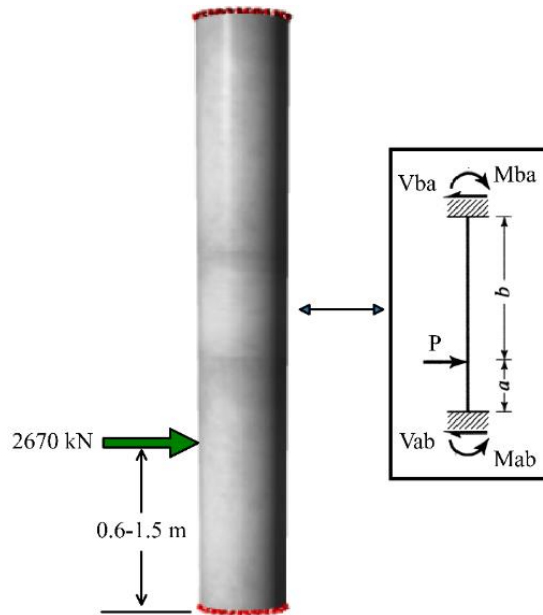


Figure 4-5. End forces in the pier

$$V_{ab} = \frac{Pab^2}{L^2} \quad \text{Eq. 52}$$

$$M_{ab} = \frac{Pb^2(3a + b)}{L^3} \quad \text{Eq. 53}$$

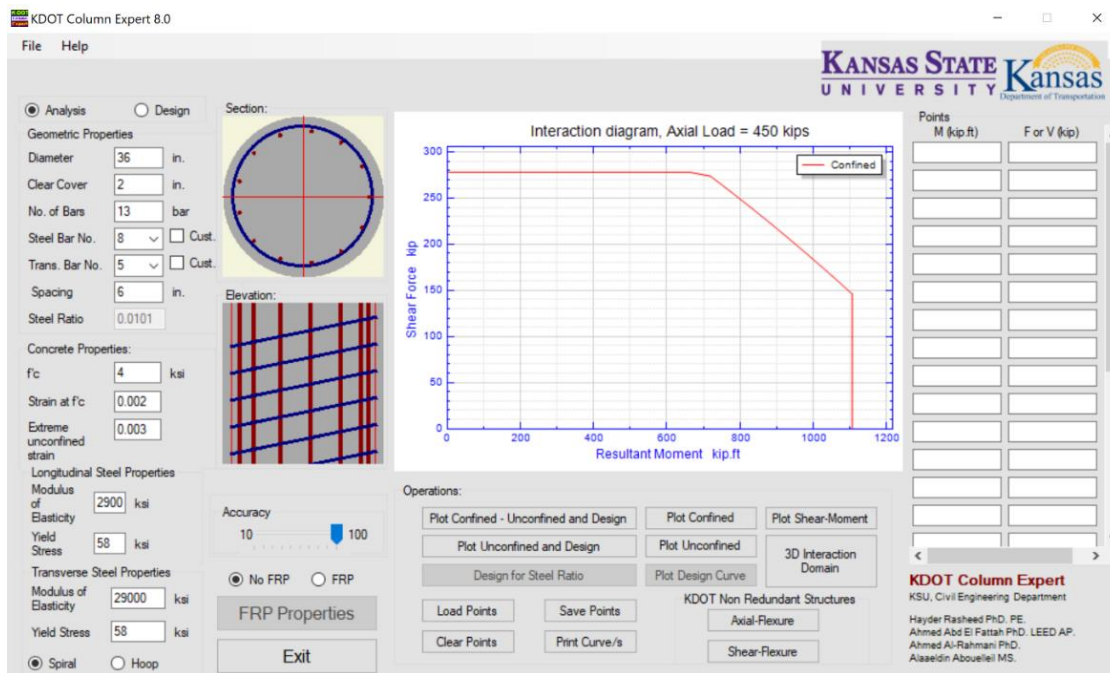
$$V_{ba} = \frac{Pba^2}{L^2} \quad \text{Eq. 54}$$

$$M_{ba} = \frac{Pa^2(a + 3b)}{L^3} \quad \text{Eq. 55}$$

Table 4-3. The internal forces in the pier due to 2670 kN (600 kips) VCF

a (See Figure 4-5)	Moment			Shear		
	End	Kip.ft	kN.m	End	Kips	kN
0.6 m (2 ft)	Mab	954.57	1294.8	Vab	580.48	2583.13
	Mba	115.70	156.94	Vba	19.52	86.87
1.5 m (5 ft)	Mab	1597.52	2166.0	Vab	492.21	2189.50
	Mba	591.67	802.22	Vba	107.79	479.50

The capacity of the pier section is calculated using a software called KDOT column expert, this tool adapts the model developed by Rasheed and Abouelleil [41] and provides a shear-moment capacity interaction diagram at a specific axial force. The user interface and the input parameters used in KDOT column expert for the pier under consideration are shown in **Figure 4-6**.

**Figure 4-6. KDOT column expert user interface and input parameters**

The capacity of the pier section is calculated with the pier subjected to an axial load represented by the dead load effects coming from the bridge beam, girders, and the deck. The calculations of the dead load are based on the tributary length of the bridge segment above the pier beam. This length is equal to half the distance to the abutments from each side in case the bridge has a single piers line, or it is equal to half the distance to the abutment plus half the distance to the next pier

as in the case of the exterior piers line for the bridge shown in **Figure 4-1**. Based on this, and considering a tributary length of 38 m, the approximated axial load is about 2000 kN (450 kips) assuming a two-pier transfer beam. The capacity interaction diagram for the pier under the given estimated axial loading is shown in **Figure 4-7**. It is obvious that the pier will not resist the 2670 kN ESF suggested by AASHTO, it is therefore considered an under-designed pier. The next sections will consider the blended contribution approach by proposing a reduced ESF to represent the VCF in the presence of sub-standard concrete barrier.

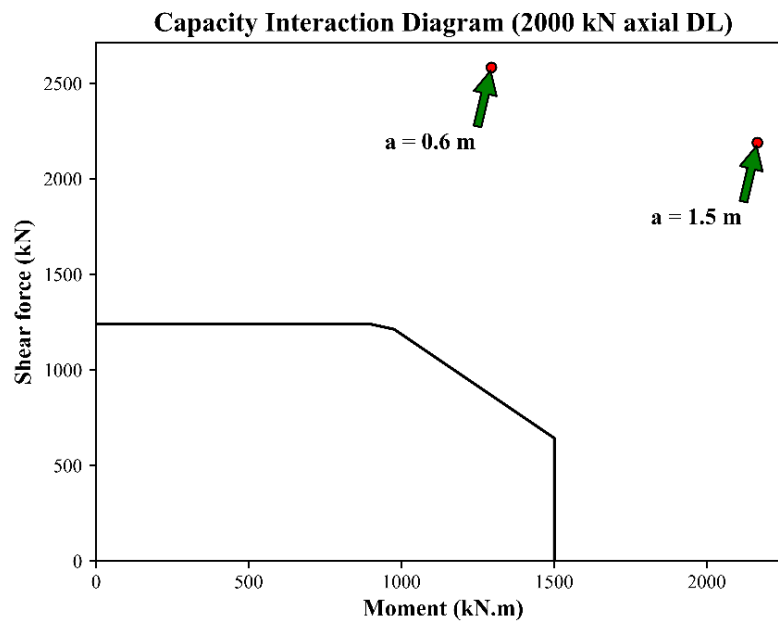


Figure 4-7. Capacity interaction diagram for the pier

4.5 Simulation Results

This section describes the contact force demand on the pier behind the barrier. This force is a result of the vehicle-pier contact and the barrier-pier contact. The vehicle-barrier contact was not reported as it has no influence on the load demand on the pier. **Figure 4-8** (a-d) shows the results of the SUT vehicle when attacking fixed and free barriers at angles of 15° and 25°. The results are summarized in **Table 4-4**. It can be seen that the barrier boundary did not have major influence on the DIF. The peak impact force (PIF) and the critical location of the pier was similar in both

boundary conditions in the two angles of attack. This might be attributed to the direct engagement of the pier in the impact since it is close to the first contact point. It is expected that if the pier is further from the contact point, higher forces will be absorbed by the barrier, yielding lower contact forces for the pier. However, the intent of this study is to place the pier in the critical location that resists higher impact forces rather than investigating the effect of the boundary conditions of the barrier on absorbing contact forces. The authors conducted another study considering the performance of the sub-standard barriers in absorbing VCF and protecting bridge piers.

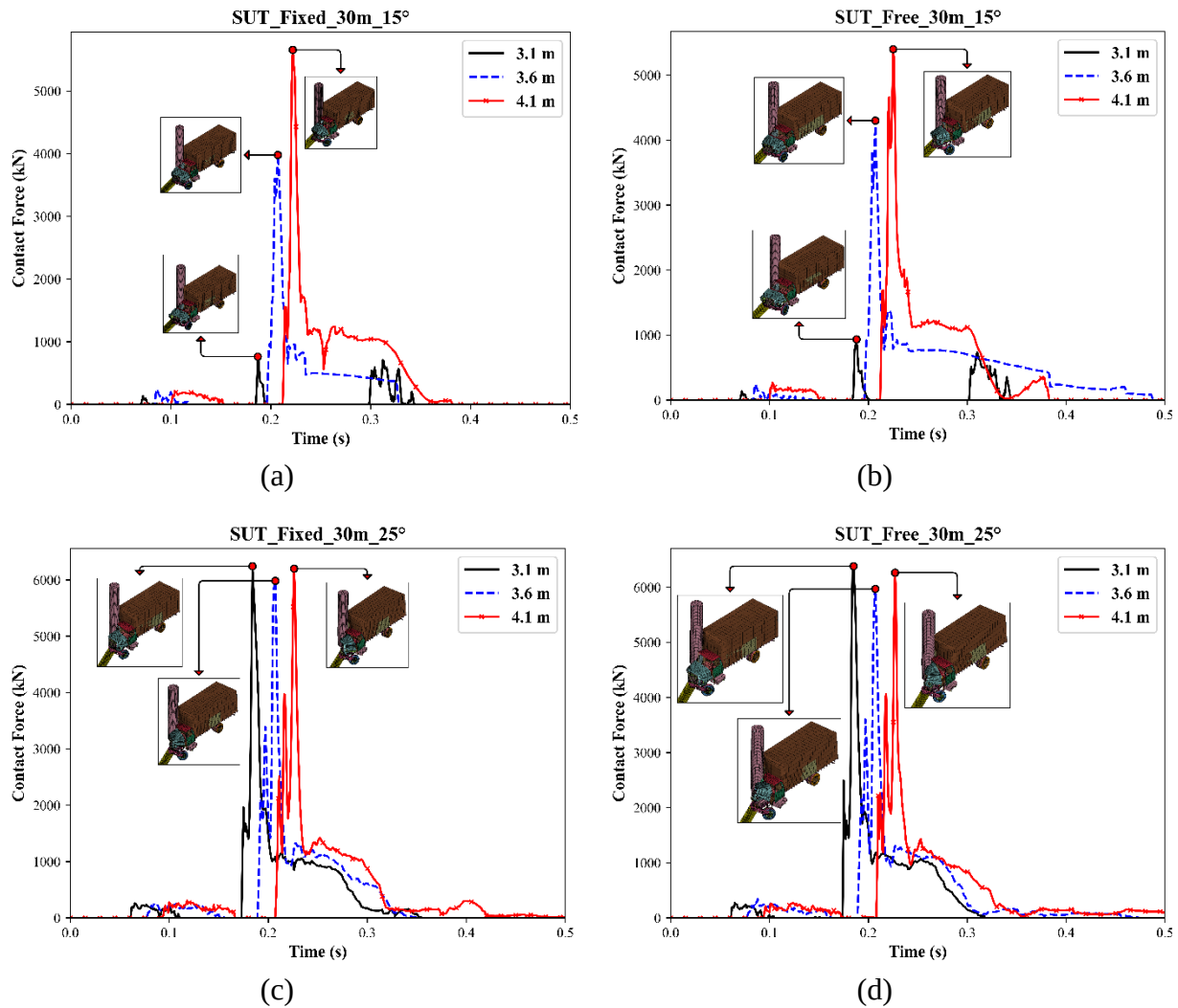


Figure 4-8. Dynamic impact force due to TL-4 vehicle

Another observation is that higher impact forces resulted when the angle of attack is 25° , which is an expected outcome since the impacting vehicle tends to penetrate the barrier rather than deviate and regain stability.

For the TST vehicle, **Figure 4-9** (a) and (b) reflect similar trend to that of the SUT in terms of the effect of the boundary condition on the DIF when the angle of attack is 15° with a slightly higher PIF when the barrier has free base.

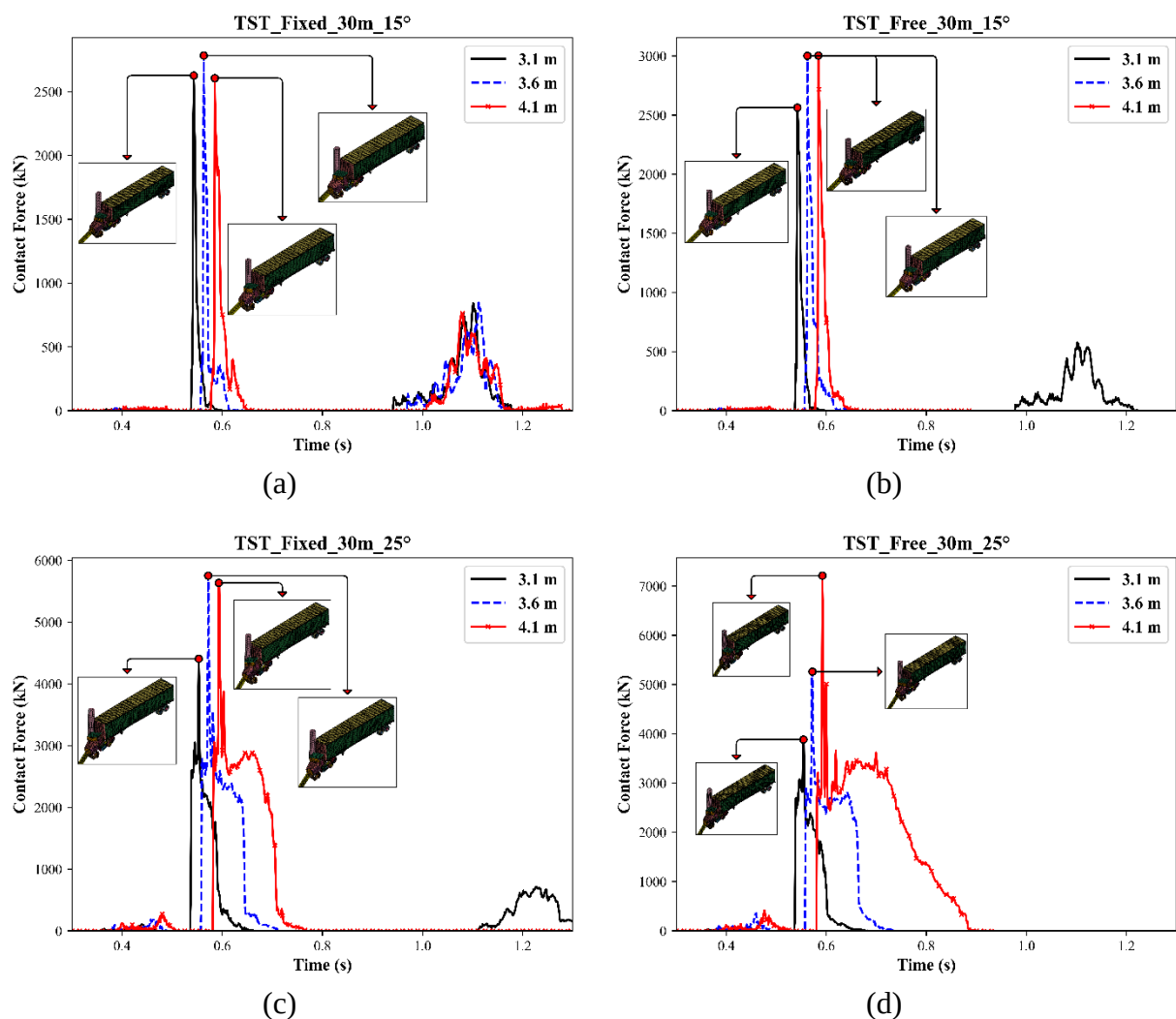


Figure 4-9. Dynamic impact force due to TL-5 vehicle

For the 25° angle of attack, the effect of the boundary conditions appears to be more influential, where the free base barriers produced higher contact forces. This might be attributed to the less

energy absorption capacity of the free barrier compared with the fixed ones, resulting in higher forces transferred to the pier. The PIF resulting from each simulation is reported in **Table 4-4**.

Table 4-4. PIF for the simulation matrix

Vehicle type	SUT				TST			
Barrier boundary	Fixed		Free		Fixed		Free	
Barrier segment length (m)	30							
Angle of vehicle attack	15°	25°	15°	25°	15°	25°	15°	25°
The critical pier location	4.1	3.1	4.1	3.1	3.6	3.6	4.1	4.1
PIF (kN)	5657	6242	5400	6382	2783	5753	3004	7210

4.6 The Dynamic Impact Force (DIF) vs. The Equivalent Static Force (ESF)

Accuracy in estimating the applied loads for the purpose of analyzing and designing structural elements is crucial. Dynamic loads are of special concern due to the high fluctuation of the load values during the loading event. Therefore, existing research and guidelines tend to suggest an ESF to simplify the analysis and design process. The method of reflecting the dynamic effect through the estimated ESF depends on the type of the dynamic load. For instance, highway sign structures subjected to wind loading are analyzed and designed by applying an estimated static pressure equivalent to the wind load, however the dynamic nature of the wind is reflected through amplifying the applied static pressure using a dynamic amplification factor (DAF) that is a function of the dynamic properties of the structure [42]. The same applies when applying seismic loads through the equivalent static method or the response spectrum [43].

Sharma et al. [44] showed that the dynamic shear force applied on piers from VCF is greater than the static counterpart and also varies depending on the impact scenario. Therefore, several studies considered evaluating the VCF on structural elements in an attempt to quantify a conservative ESF that can be used for analysis and design purposes as well as determining the impact capacity of the structural member subjected to collision. Roy et al. [37] conducted a state-of-the-art review of the studies on the damage induced on bridge piers due to VCF. Several models on obtaining the ESF

from VCF were summarized. Three models will be presented in this study to analyze and compare. Eventually, one model will be adopted to build the findings on.

4.6.1 Model 1: The Modified Global ESF

This model is a modified version of the model presented by Zhou et al. [36]. The original model to find the global equivalent static force (GESF) is given in **Eq. 56**.

$$GESF = \frac{\int_0^t p(i) di}{t} \quad \text{Eq. 56}$$

where $p(i)$ is the force at instant i and t is the impact duration. The modification implied to this model is through dropping the instants when the contact between the vehicle and the pier is lost; that is points of zero contact force. Thus, producing a new time history including the effective impact time. Then the contact forces are averaged over the continuous period of contact. **Figure 4-10** shows an original and modified contact force graph. The ESF is the integration of the filled area divided by the effective impact time.

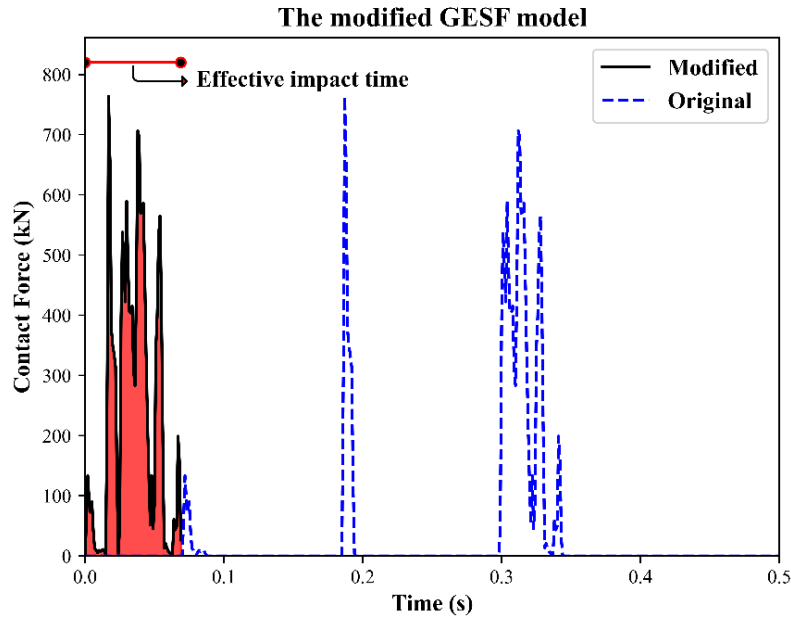


Figure 4-10. ESF averaged over the effective impact time based on model 1

4.6.2 Model 2: The Local ESF

This model is presented by Zhou and Li [45] and shown in **Eq. 57**. It is based on averaging the DIF in the local zone of the PIF over a 50-ms time window centered at the PIF point to get the local equivalent static force (LESF) as shown in **Figure 4-11**.

$$LESF = \frac{\int_{tp-25}^{tp+25} p(i) di}{50} \quad \text{Eq. 57}$$

where $p(i)$ is the force at instant i and tp is the time corresponding to the PIF. The ESF is the integration of the filled area divided by 50-ms.

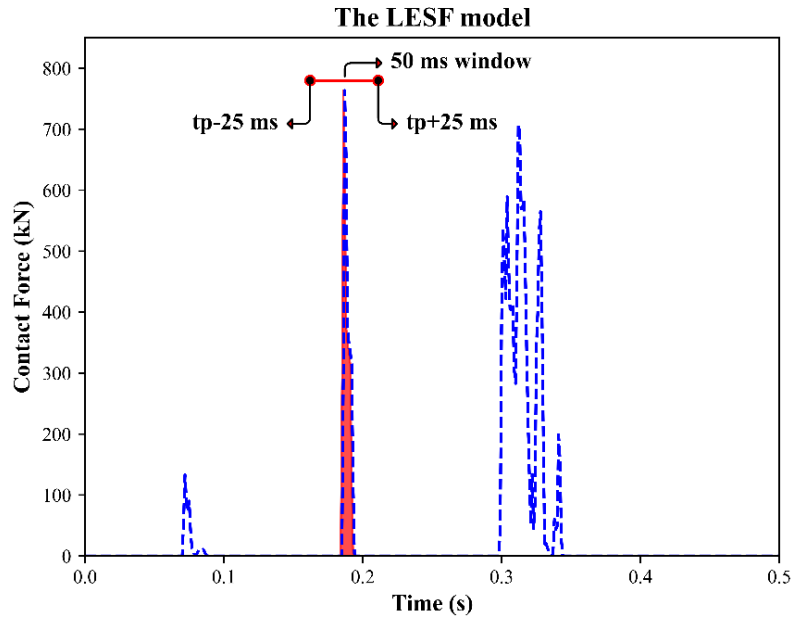


Figure 4-11. ESF averaged over 50-ms window based on model 2

Although this model reduces the effect of non-critical impact instances but fixing a 50-ms time window allows having non-critical instances that will significantly reduce the ESF as in **Figure 4-11**. Therefore, Abdelkarim and ElGawady [46] suggested a model considering the peak of 25-ms moving average, in this study the same concept is adopted with changing the averaging window to 50-ms as discussed next.

4.6.3 Model 3: The Peak of Fifty Milli-Second Moving Average (PFMSA)

This model is able to capture the PIF throughout the impact time history. It gives less influence to dynamic impact forces acting over very short durations (as in **Figure 4-11** time 0.18-0.2 second) and higher influence to those acting over longer durations (as in **Figure 4-11** time 0.3-0.35 second). A comparison between original and model 3 contact force graph is shown in **Figure 4-12**. The ESF is the PFMSA. The results of the three models are summarized in **Table 4-5** below. It is obvious that model 1 yielded a significant reduction of the PIF in almost all the simulations. Also, models 2 and 3 are reasonably comparable. Therefore, the ESF obtained from model 3 will be used in phase 2.

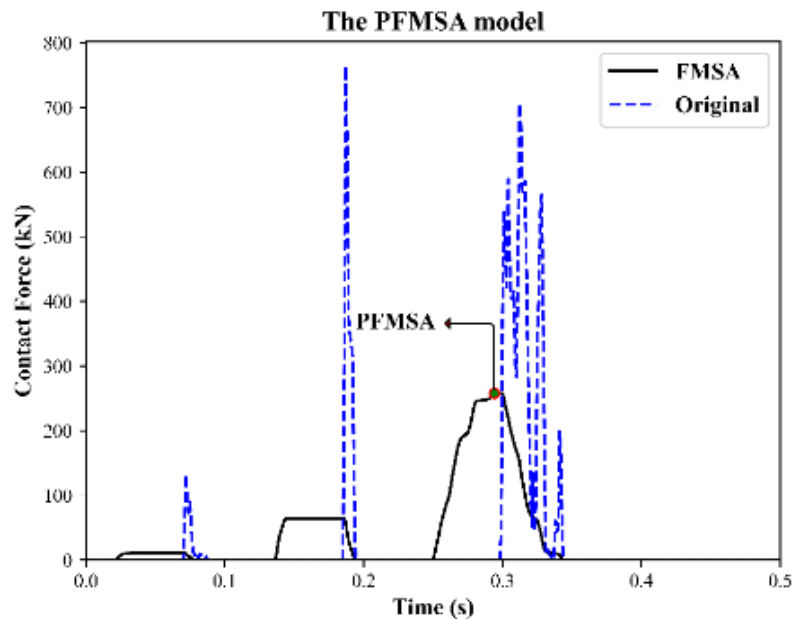


Figure 4-12. ESF as the peak of 50-ms moving average based on model 3

Since the boundary conditions did not have significant influence on the PIF, a uniform ESF will be selected to govern both boundaries. The selected ESFs for SUT attacking at 15° and 25° are 2000 kN and 2100 kN respectively. For the TST, the selected ESFs are 750 kN and 3350 kN for the two angles of attack respectively. Since the AASHTO specifications require the VCF to be acting in a direction of 0-15°, comparing the proposed values for this angle of attack to the current

standards 2670 kN (600 kips) yields a reduction of 25% and 72% for the SUT and TST respectively.

Table 4-5. ESF based on the three models

Vehicle type	SUT				TST			
Barrier boundary	Fixed		Free		Fixed		Free	
Barrier segment length (m)	30							
Angle of vehicle attack	15°	25°	15°	25°	15°	25°	15°	25°
PIF (kN)	5657	6242	5400	6382	2783	5753	3004	7210
ESF1 (kN)	790	766	794	903	204	1161	220	1485
ESF2 (kN)	1460	1906	1678	1892	680	2222	718	2399
ESF3 (kN)	1814	2133	2002	2093	770	3013	777	3372
ESF1/PIF	0.14	0.12	0.15	0.14	0.07	0.20	0.07	0.21
ESF2/PIF	0.26	0.31	0.31	0.30	0.24	0.39	0.24	0.33
ESF3/PIF	0.32	0.34	0.37	0.33	0.28	0.52	0.26	0.47

4.7 Phase 2: Application of the Modified VCF

In this section, information about the location of the first impact point between the colliding vehicle and the pier is obtained from the dynamic analysis. Based on this location, the ESF that was proposed in the previous section is applied, the internal forces in the pier are found and compared to the pier capacity. **Figure 4-13** (a) and (b) show two simulations of the SUT corresponding to 15° and 25° angles of attack respectively. It is obvious that the location of the first impact point is about 1.2-1.3 m (about 4 ft) from the ground. For the TST simulations, this location might reach up to 4.1 m (13.5 ft) from the ground as shown in **Figure 4-13** (c) and (d) for 15° and 25° respectively. **Table 4-6** shows the end moments and shears corresponding to the ESF applied at the locations defined in **Figure 4-13** for the corresponding representative simulations.

Table 4-6. Critical internal forces based on the proposed ESF

Vehicle	Angle	ESF (kN)	a (m)	Moment (kN.m)		Shear (kN)	
				Mab	Mba	Vab	Vba
SUT	15°	2000	1.2	1488	403	1767	234
	25°	2100	1.3	1617	485	1817	284
TST	15°	750	4.1	230	611	138	613
	25°	3350	4.1	1024	2726	613	2738

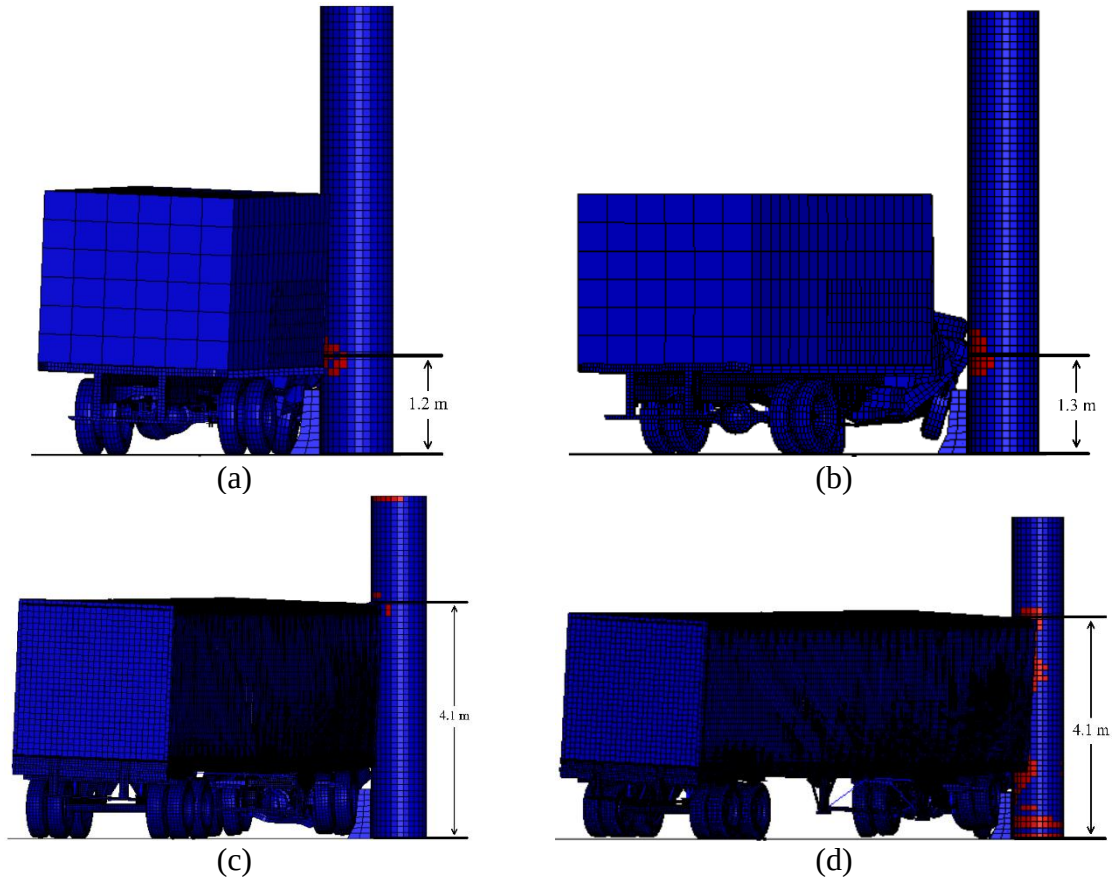


Figure 4-13. Location of ESF application

By plotting the shear and moment diagrams, one may observe that the critical shear and moment at one of the ends is the critical internal shear and moment in the pier. These values will be projected on the capacity interaction diagram as shown in **Figure 4-14**.

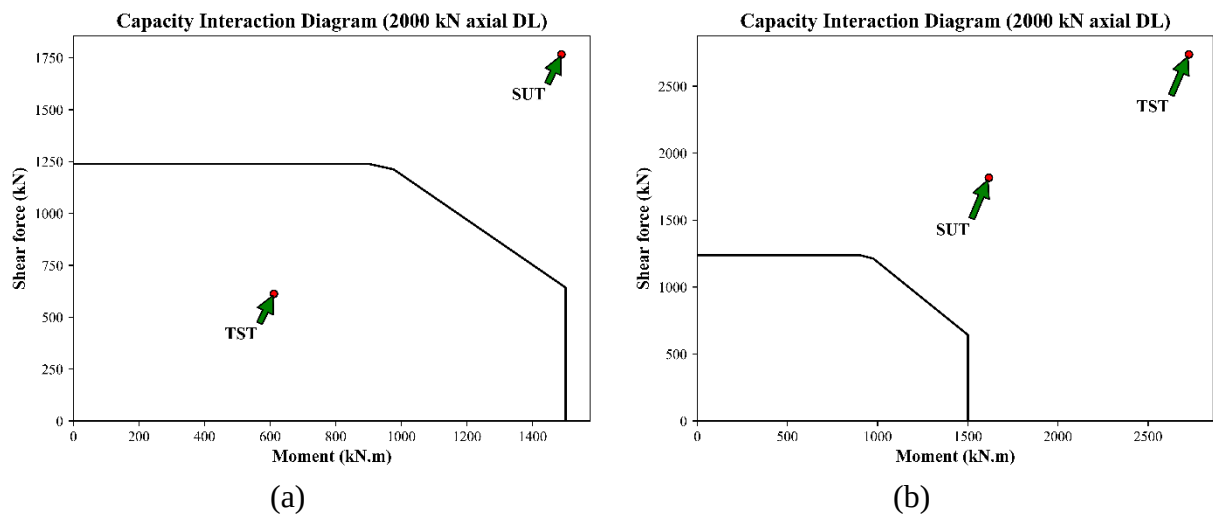


Figure 4-14. Adequacy of the pier under the proposed VCF (a) 15° angle of attack (b) 25° angle of attack

It can be seen that when the angle of attack is 15° , the TST vehicle impact produced internal forces that can be resisted by the considered pier. However, the SUT vehicle produced high shear values beyond the capacity of the pier. This might be attributed to the fact that the mass concealed in the back of the SUT vehicle was in direct contact with the pier and thus produced high inertial forces. Although the TST vehicle is the one used to describe TL-5 crash events, designers should be aware that impacts with lumped masses travelling at high speed will yield high shear demands. When the angle of attack is 25° , the considered pier fails to meet the shear and moment demands caused by the impact even in the presence of the sub-standard barriers. In this case strengthening techniques should be used to upgrade the pier capacity against VCF. However, attacking at this angle is considered a crash scenario that is less likely to happen as discussed before.

4.8 Conclusion

In this study, a comprehensive dynamic finite element parametric study is conducted to assess the effect of sub-standard barriers used as intervening structures in contributing to the reduction of the VCF on bridge piers. The main parameters studied are the type of the colliding vehicle, the angle of attack, the boundary conditions of the intervening sub-standard barrier and the pier location along the barrier's length. A specific sub-standard barrier and pier design were implemented from KDOT drawings for this parametric study. The pier was found to be deficient under the required AASHTO's ESF to address the VCF. On the other hand, the parametric simulations resulted in a range of DIF time histories. By extracting the PIF from these time histories and properly converting them into ESF values, the proposed reduction in the VCF is evaluated. For the angle of attack specified by AASHTO, the TL-5 crash scenario was found to be adequately within the interaction capacity of the examined pier design. However, this was not the case for the TL-4 crash scenario. Extending the angle of attack to 25° showed that the TL-4 and the TL-5 cases definitely require

strengthening of this bridge pier. In addition, it is important to report that the centroid of the impact force in case of the SUT was within the range of heights adopted by AASHTO. This was not the case for the TST where the centroid location of the impact took place at much higher height level. Accordingly, the authors suggest implementing some of these important findings in future editions of AASHTO bridge design specifications to reduce the demand on the piers in the presence of sub-standard barriers.

Chapter 5 - Conclusions and Recommendations

The aim of this research was to assess the performance of sub-standard RC barriers as protective structures to bridge piers against VCF and quantify their contribution in reducing the ESF that piers are required to resist. To achieve this aim, several objectives have been set for investigation throughout this research.

The first objective was to explore the currently used methodologies to obtain the lateral capacity of RC barriers and propose accurate ones that can predict the actual transverse capacity of RC barriers. The investigation for this objective showed that the currently used AASHTO's procedure of YLA results in a significant underestimation of the transverse capacity of RC barriers. Also, a detailed YLA procedure was developed and proposed along with an equivalent truss technique as alternative methodologies for AASHTO's YLA for estimating the transverse capacity of RC barriers. A case study of a sub-standard barrier was analyzed using the previously mentioned methods and verified against FEA using Abaqus. The results showed agreement between the FEA and the proposed methods. Also, it was shown that AASHTO's YLA underestimated the capacity of the sub-standard barrier by about 50%.

The second objective was to assess the performance of sub-standard barriers for the purposes of bridge piers protection. The intent of this assessment was to obtain comprehensive idea about the behavior of these barriers during high TLs crash events represented by TL-4 and TL-5. An extensive and detailed simulation matrix has been conducted considering different quantities of interest (QoI). The QoI included kinetic energy, velocity of the impacting vehicle, lateral displacement as well as the contact force demand on the sub-standard barrier under consideration. The investigation for this objective confirmed that the reason for disqualifying the sub-standard barrier is due to vehicle rolling over. However, in both TLs, the examined sub-standard barrier

redirected the impacting vehicle without penetration. Also, it was concluded that the base boundary condition of the barriers plays an important role in influencing some important performance parameters. Specifically, the fixed base boundary condition performs more efficiently than the free base boundary condition in reducing the lateral displacement, while increasing the contact force demand on the barrier and being less economically efficient.

The last objective of this research was to assess the effect of sub-standard barriers used as intervening structures in contributing to the reduction of the VCF on bridge piers. A bridge pier that was found to be deficient to address the VCF under the required AASHTO's ESF is used in a simulation matrix to obtain a range of DIF time histories. By obtaining the ESF from these time histories, the proposed reduction in the VCF is quantified to be at least 25%.

Based on the results of this research, it is recommended that:

1. A distinction in the structural adequacy criterion to be made between the barrier's load resistance and its geometrical adequacy of vehicle redirection. This recommendation is based on the fact that sub-standard barriers can structurally withstand impact loads of high TLs. However, the failure of the sub-standard barrier in satisfying the structural adequacy criterion is referred to its geometrical inadequacy to redirect the impacting vehicle.
2. AASHTO specification accepts sub-standard barriers to achieve some level of protection as a contribution to reduce the currently required ESF of bridge piers to withstand VCF.
3. A consideration to be given to the location of the ESF application point. The dynamic simulations showed that, in the presence of sub-standard barriers, the location of the PIF was within the range of heights adopted by AASHTO in case of the SUT. However, this was not the case for the TST where the centroid location of the impact took place at much higher height level due to the impact with the trailer.

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