2016

Vulnerability Assessment of Steel Bridges Due to On-Deck Blasts

Justus Frenz
University of Rhode Island, justus.frenz@web.de

Follow this and additional works at: <https://digitalcommons.uri.edu/theses>

Recommended Citation

Frenz, Justus, "Vulnerability Assessment of Steel Bridges Due to On-Deck Blasts" (2016). *Open Access Master's Theses*. Paper 894.
<https://digitalcommons.uri.edu/theses/894>

This Thesis is brought to you for free and open access by DigitalCommons@URI. It has been accepted for inclusion in Open Access Master's Theses by an authorized administrator of DigitalCommons@URI. For more information, please contact digitalcommons-group@uri.edu.

VULNERABILITY ASSESSMENT OF STEEL BRIDGES
DUE TO ON-DECK BLASTS

BY
JUSTUS FRENZ

A THESIS SUBMITTED IN PARTIAL FULFILLMENT OF THE
REQUIREMENTS FOR THE DEGREE OF
MASTER OF SCIENCE
IN
CIVIL AND ENVIRONMENTAL ENGINEERING

UNIVERSITY OF RHODE ISLAND

2016

MASTER OF SCIENCE THESIS
OF
JUSTUS FRENZ

APPROVED:

Thesis Committee:

Major Professor	Mayrai Gindy
	George Tsiatas
	Arun Shukla

Nasser H. Zawia
DEAN OF THE GRADUATE SCHOOL

UNIVERSITY OF RHODE ISLAND
2016

ABSTRACT

Highway bridges are a critical element in the infrastructure for personal transportation and movement of goods, yet they are constantly exposed to a number of impacts and risks. One of these possible threats is an accidental or intentional explosion on top of the bridge deck.

In this thesis, the effects of deterioration (in the form of section loss) of the superstructure subjected to a blast load are analyzed for an example bridge. The software ABAQUS and its CONWEP model were utilized to run a different scenarios of section thicknesses reductions of the steel elements, varying thicknesses of the concrete slab deck and the locations of the blast source.

The analysis output suggests that, for the blast load assumed in this thesis, only small parts of the concrete deck structurally fail while the rest of the bridge remains intact but permanently deformed in a way that a replacement of the bridge superstructure after the explosion appears to be inevitable. Increasing section loss obviously does have an impact on the deformations. However, the differences to the initial bridge structure observed in the analysis are only minor as the zones of structural failure and permanent deformations grow slightly but the overall stability does not change fundamentally. So, even though the initial bridge performs better when exposed to a blast event, initial and deteriorated bridges sustain permanent deformations making bridge replacement necessary in both cases.

ACKNOWLEDGMENTS

I would first like to thank my thesis advisor Professor Mayrai Gindy of the Civil and Environmental Engineering Department at the University of Rhode Island for helping me to find a topic and her encouragement during the time of research and writing of the thesis.

Besides my advisor, I would especially like to thank Professor George Tsiatas of the Civil and Environmental Engineering Department at the University of Rhode Island for answering questions, pointing to very interesting sources and providing inspiration throughout the entire process.

My sincere thank also goes to Professor Arun Shukla of the Mechanical, Industrial and Systems Engineering Department at the University of Rhode Island for serving on my thesis committee as an outside committee member and Professor Frederick J. Vetter of the Department of Electrical, Computer and Biomedical Engineering at the University of Rhode Island for being the Chair of my Defense Committee.

Finally, I would like to thank my family for providing me with unfailing support and continuous encouragement throughout my years of study, especially the times spent abroad.

Justus Frenz

TABLE OF CONTENTS

ABSTRACT	ii
ACKNOWLEDGMENTS	iii
TABLE OF CONTENTS.....	iv
LIST OF TABLES	vii
LIST OF FIGURES	ix
CHAPTER 1: INTRODUCTION.....	1
CHAPTER 2: REVIEW OF LITERATURE.....	3
2.1 Blast Loading.....	3
2.1.1 Explosive Attack.....	4
2.1.2 Event Location	4
2.1.3 Explosive Materials	7
2.1.4 Blast event and Blast Wave Phenomena	9
2.1.5 Shock Loading	13
2.1.6 Fragments	15
2.1.7 Example Bridge	16
2.2 Structural Response.....	16
2.2.1 Structural System Behavior	17
2.2.2 Element Response	19
2.2.3 Material Properties and Strain Rate Effects	21
2.2.4 Bridge Design Specifications	26
2.3 Simulation	27
2.3.1 Simulation Techniques for Impulse Loading.....	28
2.3.2 Software.....	32
2.3.3 Verification/ Validation.....	32
CHAPTER 3: METHODOLOGY	34
3.1 Research Approach.....	34
3.1.1 Structural Steel Section Deterioration.....	34

3.1.2 Concrete Deck Deterioration	36
3.2 AASHTO LRFD Guide Example Bridge	38
3.2.1 General Information	39
3.2.2 Concrete Deck.....	40
3.2.3 Steel Girder	43
3.2.5 Assumptions	52
3.3 Material Properties.....	53
3.3.1 Concrete.....	54
3.3.2 Reinforcement Steel	56
3.3.3 Structural Steel	57
3.4 Bridge Loading and Boundary Conditions	59
3.4.1 Extreme Event II.....	59
3.4.2 Permanent Load.....	60
3.4.3 Transient Loads.....	61
3.4.4 Blast Loading (BL).....	61
3.4.5 Boundary Conditions	62
3.5 Simulation Input	63
3.5.1 Analysis Type	63
3.5.2 Analysis Duration	63
CHAPTER 4: FINDINGS	65
4.1 Initial Bridge (No Deterioration)	65
4.1.1 Concrete Deck.....	66
4.1.2 Steel Girder Superstructure	74
4.1.3 Conclusion.....	85
4.2 Deteriorated Bridges	85
4.2.1 Deck Deterioration.....	86
4.2.2 Steel Deterioration	93
4.3 Validation of the Model.....	99
4.3.1 System Deformation	99
4.3.2 Blast Pressure	100
CHAPTER 5: CONCLUSION	104

5.1 Effects of Section Reduction on the Structural Response	104
5.2 Model accuracy.....	105
5.2.1 Simulation	105
5.2.2 Explosive Charge	105
5.2.3 Deterioration Assumptions.....	106
5.3 Conclusion.....	106
5.4 Further Research	107
APPENDICES	108
A.1 Static Deformation of the Example Bridge	108
A.2 Deck Reinforcement System Input.....	112
BIBLIOGRAPHY	114

LIST OF TABLES

TABLE	PAGE
Table 1: Concrete and Reinforcement Steel DIF values	23
Table 2: Structural Steel DIF values	26
Table 3: Steel Deterioration Cases 0 to 4	35
Table 4: Steel Deterioration Cases 5, 10 and 20	35
Table 5: Deck Deterioration Cases Overview	38
Table 6: Positive Moment Deck Reinforcement	41
Table 7: Negative Moment Deck Reinforcement	41
Table 8: Web, Stiffener and Cross Frame Thicknesses	46
Table 9: Bottom Flange Thicknesses	48
Table 10: Top Flange Thicknesses	49
Table 11: Top Flange Widths	49
Table 12: Concrete General Material Properties	54
Table 13: Concrete Elastic Behavior	54
Table 14: Concrete Plasticity General	54
Table 15: Concrete Plasticity Compressive Behavior	55
Table 16: Concrete Plasticity Tensile Behavior	55
Table 17: Reinforcement Steel General Properties	56
Table 18: Reinforcement Steel Elastic Behavior	56
Table 19: Reinforcement Steel Property Modification	57
Table 20: Reinforcement Steel Plasticity Input	57
Table 21: Structural Steel General Material Properties	58

Table 22: Structural Steel Elastic Behavior	58
Table 23: Structural Steel Material Modification	58
Table 24: Structural Steel Plasticity Input	59
Table 25: Load Combination - Extreme Event II (AASHTO (2012))	59
Table 26: Blast Loading Cases.....	62
Table 27: Deck Reinforcement, Negative Moment, No Deterioration	112
Table 28: Deck Reinforcement, Positive Moment, No Deterioration.....	112
Table 29: Deck Reinforcement, Negative Moment, Mild Deterioration	112
Table 30: Deck Reinforcement, Positive Moment, Mild Deterioration.....	113
Table 31: Deck Reinforcement, Negative Moment, Severe Deterioration	113
Table 32: Deck Reinforcement, Positive Moment, Severe Deterioration.....	113

LIST OF FIGURES

FIGURE	PAGE
Figure 1: Blast Location, Side View	6
Figure 2: Blast Location, Plan View	6
Figure 3: Pressure-time variation for Free-Air Burst (<i>UFC 3-340-02</i> , Department of Defense (2014)).....	10
Figure 4: Free-air burst blast environment (Department of Defense (2014))	11
Figure 5: Structural Steel Stress-Strain Curve from <i>UFC 3-340-02</i> , Department of Defense (2014)	25
Figure 6: Possible Analysis Method Combinations (from NCHRP (2010))	29
Figure 7: Deck Deterioration, one Patch.....	36
Figure 8: Deck Deterioration, two Patches	37
Figure 9: Deck Deterioration, four Patches.....	37
Figure 10: Deck Deterioration, six Patches.....	37
Figure 11: Static System, Example Bridge (FHWA, 2003).....	39
Figure 12: Superstructure Cross Section, Example Bridge (FHWA, 2003)	39
Figure 13: Positive and Negative Deck Moment Regions	41
Figure 14: Steel Girder Properties, Example Bridge (FHWA, 2003)	44
Figure 15: Steel Girder Properties, Model Input.....	45
Figure 16: Placement of Intermediate Stiffeners	46
Figure 17: Location of Intermediate Stiffeners	47
Figure 18: Model of Steel Elements.....	50

Figure 19: Bearing Stiffener and Cross Frame Model	51
Figure 20: Cross-Frame Model	51
Figure 21: Bearing Stiffener at Abutment Support	51
Figure 22: Bearing Stiffener at Pier Support	51
Figure 23: Girder Naming for Analysis	67
Figure 24: Vertical Deformation of Deck (Section 18.5m), Blast Case 1	67
Figure 25: Deformation Control Locations.....	68
Figure 26: Vertical Deformation of Span for two Control Points at time step 0.1 sec., Blast Case 1	69
Figure 27: Length of Deck Deformation.....	69
Figure 28: Difference of Vertical Deck Deformation for Case: Steel 0 Deck 0 Blast 1	70
Figure 29: Deck Failure Criteria Sketch	70
Figure 30: Vertical Deformation of Deck Section 36.8 m at Time Step 0.1 sec. for Blast Case 2.....	72
Figure 31: Difference of Vertical Deck Deformation for two Control Points at Time Step 0.1 sec.	73
Figure 32: Failure Criteria Case: Steel 0 Deck 0 Blast 2	73
Figure 33: Vertical Deformation of Girders at Time Step 0.005 sec.	75
Figure 34: Girder Location Naming [m]	75
Figure 35: Vertical Deformation of Girders at Time Step 0.1 sec.	76
Figure 36: Vertical Deformation of Girders at Location 18.5 m	77
Figure 37: Absolute Rotation of Girder +3 at Abutment Support	78

Figure 38: Vertical Deformation of Girders at Time Step 0.15 sec.	78
Figure 39: Vertical Deformation of Girders at Time Step 0.2 sec.	79
Figure 40: Vertical Deformation of Girders at Time Step 0.05 sec. (Blast Case 2)	80
Figure 41: Vertical Deformation of Girders at Time Step 0.075 sec. (Blast Case 2) ..	81
Figure 42: Vertical Deformation of Girders at Time Step 0.1 sec. (Blast Case 2)	81
Figure 43: Vertical Deformation of Girders at Time Step 0.125 sec. (Blast Case 2) ..	82
Figure 44: Vertical Deformation of Girders at Time Step 0.2 sec. (Blast Case 2)	82
Figure 45: Vertical Deformation of Girder +3 at Location 36.8 m (Blast Case 2)	83
Figure 46: Vertical Deformations of Girder +3 Case: Steel 0 Deck 0 Blast 1 - Long Analysis (10 sec)	84
Figure 47: Vertical Deformations of the Deck, Cases Steel 0 Deck 11, 12, 14 & 16..	86
Figure 48: Deformation Differences for Cases Steel 0 Deck 11, 12, 14 & 16.....	87
Figure 49: Vertical Deck Deformations for Cases Steel 0 Deck 21, 22, 24 & 26	88
Figure 50: Deformation Differences for Cases Steel 0 Deck 21, 22, 24 & 26.....	88
Figure 51: Vertical Deformations of Girder +3 at Location 18.5m for Cases Steel 0 Deck 11, 16, 21 & 26	89
Figure 52: Vertical Deck Deformation at Section 36.8 m for Cases Steel 0 Deck 11, 12 & 14 (Blast Case 2)	91
Figure 53: Deformation Differences for Cases Steel 0 Deck 11, 12, 14, 21, 22 & 24 (Blast Case 2)	91
Figure 54: Vertical Deformation of Deck for Combined Deterioration (Blast Case 1)92	
Figure 55: Vertical Deformation of Girder +3 for Cases Steel 0, 1, 2, 3, 4, 5 & 10 Deck 0	93

Figure 56: Maximum Vertical Deformation of Girder +3 for Cases Steel 0, 1, 2, 3, 4, 5 & 10 Deck 0	94
Figure 57: Vertical Deformation of Girder +3 at Time Step 0.092 sec for Cases Steel 0, 1, 2, 3, 4, 5 & 10.....	95
Figure 58: Vertical Deformation of Girder +3 at Location 18.5m for Cases Steel 0, 5, 10 & 20 Deck 0	96
Figure 59: Vertical Deformations of Girder +3 at Location 18.5m (Blast Case 2)	97
Figure 60: Vertical Deformation of Girder +3 at Location 36.8m for Cases Steel 0, 1, 2, 3 & 4 Deck 0 (Blast Case 2).....	98
Figure 61: Vertical Deformation of Girder +3 at Location 36.8 m for Cases Steel 0, 5 & 20 Deck 0 (Blast Case 2).....	99
Figure 62: ABAQUS Result of Static Analysis	99
Figure 63: Blast Loading from CONWEP model for Point located under Blast Source	101
Figure 64: CONWEP model loading at 0.0008 seconds.....	101
Figure 65: CONWEP model loading at time steps 0.0013 sec. and 0.0035 sec.	102

CHAPTER 1: INTRODUCTION

Highway bridges are a vital part of the national infrastructure as they enable transportation and traffic to cross rivers, valleys and other obstacles without diversion or deceleration of the traffic flow. This outstanding function naturally creates a bottleneck situation in the transportation network. Any restraints on this small network element automatically result in major and far-reaching annoyance throughout larger parts of the network. Bridge closures cause long diversions for road users and overloading of other roadways. Therefore, the serviceability of bridges is of great economic importance and high public interest.

To provide the desired level of functionality, bridges have to be designed and maintained to resist a variety of impacts from traffic and wind loading, but also weathering and aging. To continuously guarantee a certain level of safety, they have to be surveyed and assessed on a regular basis. Depending on a variety of factors, officials schedule recurring condition surveys of their assets so that the structural health of the structure is known at all times and reevaluation of load allowance, maintenance, or replacement can be carried out as necessary.

One imaginable impact to every highway bridge is an on-deck blast event. Explosions on roadway bridges could be caused inadvertently in accidents or intentionally as part of an attack trying to disrupt the transportation network. Bridges are critical infrastructure assets and choke points of traffic flow, yet very accessible and difficult to protect which could make them a potential target.

Currently, the on-deck blast scenario is not routinely considered during the design process of regular highway bridges. Manuals and recommendations exist for blast resistant design of bridge substructures (namely pier columns) and, of course, for protective structures, but not for bridge decks and superstructures. The sixth edition of the AASHTO ‘LRFD Bridge - Design Specifications’ (2012) only provides a set of references for blast loading and analysis but no specific requirements. Likewise, no information is available to examine, how bridge deterioration affects the bridge resistance capacity to blast loading.

In this thesis, the effect of bridge deterioration (namely cross section reduction) to blast loading resistance is examined.

Upon review of current literature on blast loading, structural response and blast simulation in Chapter 2, the methodology of the analysis is discussed in Chapter 3. Here, the research approach, the software inputs such as material characterization, the example bridge, and analysis options are presented.

Chapter 4 comprehends the findings of the analysis outputs in three categories. In a first step, the original bridge is exposed to blast loadings at different blast locations to identify a characteristic bridge behavior after the detonation. In a second step, bridges with deteriorated elements are exposed to the same blast loadings. Here, the effects of the deteriorations by location can be assessed. Lastly, bridges with deterioration combinations in more than one structural element are studied.

In the final chapter (Chapter 5), conclusions from the findings of Chapter 4 are discussed.

CHAPTER 2: REVIEW OF LITERATURE

This chapter presents a comprehensive review of both current and established literature relevant for the topic of on-deck blast loading. The literature review is composed of a sequential presentation:

Section 2.1 examines the explosion and blast loading, characterizing the explosive attack, comparing different explosive materials, discussing the propagation of the shock wave, and shock phenomena on structures.

Section 2.2 subsequently describes the structural response to such shock loadings. The sub-categories include structural system behavior, element response, material properties with strain-rate effects, and failure criteria.

In Section 2.3, the simulation of the problem is presented. Here the simulation techniques, utilized software and the validation of the analysis output are discussed.

2.1 Blast Loading

In this section, a blast scenario is characterized and parameters necessary to simulate the event are presented. In a previous step, possible blast scenarios have to be identified in a risk assessment for every individual bridge. The topic ‘blast risk assessment’ will not be presented in this thesis (a step-by-step risk assessment process can, for example, be found in ‘*Guide to Highway Vulnerability Assessment for Critical Asset Identification and Protection*’, SAIC (2002)).

2.1.1 Explosive Attack

The output of the Risk Assessment is ultimately the identification of one or more possible attack scenarios that the bridge might be exposed to. It has to be checked whether those events compromise bridge's structural integrity and/or serviceability in an extent deemed unacceptable by decision makers. In the following sections, the qualitative determination of possible threats is converted into quantifiable impacts to the structural model.

The most relevant characteristics of the explosion include:

- the weight of the explosive charge
- the distance between the explosive and the structure
- the angle of impact
- the impact of fragments

2.1.2 Event Location

A variety of several blast locations might be critical for the bridge, depending on the bridge type.

For many bridge types, explosions under the span are highly concerning. Most of all, the blast impact from underneath possibly leads to a load reversal in many structural members which can lead to element failure as well as components being lifted out of their usual bearing positions. Also, fatally compromising the substructure and foundation inevitably results in a collapse of the superstructure. Furthermore, underside explosions are somewhat confined so that reflections of the blast wave can

significantly magnify the impact, for example in small spaces near abutments or in between girders (Williamson and Winget 2005).

For most cable bridges, locations around or especially inside of the tower are very critical as a tower failure would lead to a collapse of the entire bridge. Unless explosions are especially set up to cut a number of cables, on-deck blast is not likely to cut cables accidentally due to their extremely small area and round shape as well as their flexibility (Williamson and Winget 2005).

For box girders, internal explosions are the most critical as the confinement pressure could lead to failure (Williamson and Winget 2005).

Nevertheless, one scenario applicable to all bridge types is the on-deck blast event examined in this report. Many of the aforementioned attacks can be avoided by simple limitation of accessibility. For example, the access to bridge towers and the inside of the box girder can be controlled and monitored. Explosions from below the bridge may also be controlled by avoiding parking spaces under the bridges, restricting access paths or for the case of a highway bridge, the lower road can be channeled away from the bridge supports and substructure by permanent barriers making the critical points difficult to access. The very controlling parameter ‘standoff distance’ between the charge and the object can thereby be increased significantly. In contrast, on-deck access to bridges cannot be restricted without major interference of the bridges original purpose of serving as a roadway (e.g. weight restrictions for cars and trucks). Risk mitigation methods can also include car and truck searches before they are allowed onto the bridge. This extreme method however heavily impacts the flow of the traffic.

As this thesis focusses on on-deck explosions, the standoff distance is very small and only consists of the distance between the center of the charge and the bridge deck surface. For the following report, two location scenarios will be considered:

Location 1 is a charge located at midspan with a height of 6 ft. 7 in. (2.0 m) for a one-span collapse (Blast Case 1).

Location 2 is a charge placed directly above the center pier at 6 ft. 7 in. (2.0 m) possibly leading to substructure failure and/or a two span collapse (Blast Case 2).

Figure 1: Blast Location, Side View

Figure 2: Blast Location, Plan View

The Example Bridge will be presented thoroughly Chapter 3 (Section 3.2).

2.1.3 Explosive Materials

Explosive materials can be classified by their physical state (solid, liquid or gasiform) with different blast pressure environments being produced by each material during the explosion. Blast effects generated by high-explosive, solid materials are best known. Their blast pressure distributions, impulse loading, characteristic loading durations and other effects of the explosion are well established (Department of Defense, 2014). In the following, these characteristics are presented for the solid explosive material TNT (Trinitrotoluene).

2.1.3.1 TNT Equivalency

Across the reviewed literature, TNT was the one material chosen for the analysis. Even though other materials might actually explode in the scenario, efforts were made to convert the amount of this other material into the amount of TNT that would release the same heat and therefore has a similar effect on the structure and the surroundings. The explosive charges are characterized by the TNT-equivalent weight ([lb] or [kg]) of the explosive. However, since condensed high explosive produce similar characteristic blast waves, with an adjustment of the weight, the effects of different explosives can be modeled sufficiently with the TNT approach. The TNT equivalent charge weight is determined by (Conrath et al. (1999)):

$$W = \left(\frac{\Delta H_{EXP}}{\Delta H_{TNT}} \right) * W_{EXP}$$

ΔH_{EXP}	Heat of detonation of explosive in question
ΔH_{TNT}	Heat of detonation of TNT
W_{EXP}	Weight of explosive in question

In the following, all charge weights will always be referred to as TNT-equivalent weights.

2.1.3.2 Charge Weights

All explosions analyzed in this thesis propagate through air, the detonation location has been identified at a standoff distance of 6 ft. 7 in. above the roadway (the change in location on the bridge does only effect the blast in an uncoupled analysis (see Section 2.3.1.2)), the type of explosive is always TNT (other explosive materials are expressed as a TNT equivalent weight charge) and the casement will be neglected in this study.

Thus, the next critical detonation parameter is the amount of explosive as described in the charge weight. Conrath et al. (1999) describe the weight limits that can be assumed by designers for different kind of aggressors. For instance, a handheld explosive, that is carried by the attacker and placed near the structure, can be assumed as a 50 lbm (23 kg) charge. For the vehicle mode of attack in which a vehicle such as a car or truck is driven onto or abandoned near the critical structure, the weight is only limited by the carrying capacity of the vehicle. The Blue Ribbon Panel on Bridge and

Tunnel Security’s recommendations (2003) include a table of possible magnitudes of threats. For conventional explosives, the ‘Highest Probability’ car bomb size is 500 lbs. The ‘Largest Possible’ threat, which could pass onto the bridge unnoticed, is quantified as a truck bomb with a weight of 20,000 lbs. In this report, the most probable value is assumed:

Example Charge Weight:	500 lbs. (Blast Cases 1 & 2)
------------------------	------------------------------

Instead of using a safety factor associated with impacts and resistance during the check, the Department of Defense (2014) recommends to increase the TNT equivalent charge weight by 20 percent into an effective charge weight. This is to compensate for any kinds of unknowns.

As the charge weight is a very critical design parameter in the determination of the blast impact on the structure, this recommendation will be applied. The effective charge weight is (Blast Cases 1 & 2):

	Charge Weight	Effective Design Charge Weight
Example Charge:	500 lbs. .	600 lbs. (272.6 kg)

2.1.4 Blast event and Blast Wave Phenomena

Detonations are described as sudden, violent release of energy in a comparatively small area. The explosive material is being converted into very high pressure gas at very high temperatures leading to a pressure front that propagates outwards spherically into the surrounding atmosphere until it is disturbed by any kind of confinement or barrier.

The pressure front released during the explosion, or blast wave, travels away from the burst point and is characterized by (Department of Defense, 2014):

- a time t_A as the duration of travel of the blast wave front through the medium until it impacts the surface,
- a positive peak pressure (much) greater than the ambient pressure P_{S0} ,
- a pressure decay back to the ambient pressure and
- a negative pressure phase with a pressure below the ambient pressure which is usually a lot longer than the positive phase but less important for the design.

The incident impulse for both the positive and negative phase can be determined by integrating the area under the curve of the respective phase.

Figure 3: Pressure-time variation for Free-Air Burst (*UFC 3-340-02*, Department of Defense (2014))

Depending upon boundary conditions such as detonation location and confinement, the parameters vary greatly. The scenario of on-deck blast can be characterized as an unconfined explosion because the pressure wave can diffuse freely in three directions (radially and upwards), so that only the original blast wave travelling from the burst point impacts the deck. No reflected wave or pressure build-ups are expected to occur at the bridge deck surface.

UFC 3-340-02, Department of Defense (2014) defines ‘Free Air Bursts’ as detonations that occur “adjacent to and above a protective structure such that no amplification of the initial shock wave occurs between the source and the protective structure”.

Figure 4: Free-air burst blast environment (Department of Defense (2014))

In this blast environment, the point of the surface located normal under the burst point has to sustain the greatest normal incident pressure and impulse. For all other points on the bridge deck, the peak pressure and impulse have to be modified with

regards to the increasing distance from the burst point and the angle of incidence. The modification reduces the size of the impact.

For some design applications, the negative shock phase is also implemented in the loading-time function of the structural response to the blast load. In steel structures such as frames, the overall motion is affected by this phase. In more rigid structures, (namely) reinforced concrete, the effects of this phase are not of high significance (*UFC 3-340-02*, Department of Defense (2014)). In this analysis, the CONWEP model includes the negative phase automatically. However, the phase is also relevant for the behavior as the structural steel components of the composite bridge have a significant influence on the overall load bearing characteristic of the structure.

Other types of Unconfined Explosions include ‘air bursts’ and ‘surface bursts’ (*UFC 3-340-02*, Department of Defense (2014)). Both of these types include reflected blast waves from either the ground or other object faces in proximity of the blast event in the analysis and focus on horizontal surfaces. They are therefore not applicable for this analysis.

For large distances between the charge and the surface of the structure, a few simplifying assumptions can be made. For example, the wave front can be assumed as planar so that the entire surface is impinged by the same characteristic blast wave pressure-time impact allowing for a faster analysis. (*UFC 3-340-02*, Department of Defense (2014))

For close-in explosions such as in this analysis, this assumption of a constant pressure-time impact along the entire surface is not acceptable and a significant overestimation of the impact would yield unrealistic, very conservative results. Also,

the difference of arrival time of the shock front cannot be disregarded. Therefore, close-in explosion accuracy has to be employed (*UFC 3-340-02*, Department of Defense (2014)). Again, this is automatically accomplished by the CONWEP model (Section 2.3.2) utilized in this analysis.

2.1.5 Shock Loading

Blast loading differs significantly from other impacts in civil engineering design. Among the differences are the duration of the loading process (sudden/ impulsive instead of static) and the duration of the load being present on the structure. The blast duration is measured in milliseconds, which, as a scale, is magnitudes shorter than the unit of seconds during wind impact analysis or very long time spans for quasi-static loads.

2.1.5.1 Loading Types

Gündel et al. (2010) describe three different characteristic types of loading. Dead load such as gravity, but also certain live loads can be treated as ‘static’ or ‘quasi-static’. These loads are not subject to fast changes and remain on the structure for a relatively long time, so only the absolute value of the load is required during the design process. The duration of these loads is greater than three times the natural period of the structural element.

Quasi-static loading: $t_d/T > 3$

In contrary, wind or earthquakes cause dynamic loading to the structure. For the design process, the load-time distribution is relevant and necessary, as this impact has a high time dependency and the response is partly governed by the dynamic characteristics of the element.

Dynamic loading: $0.3 < t_d/T < 3$

In this analysis, short-duration dynamic loads are being studied. The structure is being exposed to the nonoscillatory pulse loads of an explosion that only last milliseconds. In this domain of load durations much smaller than the elements natural period, inertia has to be taken into account. Thus, other principles and checks have to be applied to control the structural response to this type of impact.

Impulse loading: $0.3 > t_d/T$

2.1.5.2 Detonation Loading

Conrath et al. (1999) identify three principal products of a detonation, all of which are dependent on standoff distance, media through which the blast propagates, casement, charge weight, and type of explosive.

- Total Impulse Delivered
- Peak Pressure Delivered
- Delivered Velocity, Distribution and Mass of Fragments

As stated previously, a time-dependent pressure variation must be established for the design process. For structural loading, the dynamic pressure is the controlling

input. It can be determined from charts (e.g. Figure 2-3, *UFC 3-340-02*, Department of Defense (2014)) as relationship between the incident shock pressure (Section 2.1.4) and the dynamic pressure is established. In this analysis, the CONWEP model (Section 2.3.2) will be used to determine blast pressures on the structure instead of hand calculations. The figures from *UFC 3-340-02*, Department of Defense (2014) will be utilized to for the validation of the CONWEP model input to check the impact to the structure.

2.1.6 Fragments

Fragments are another important product of explosions. They can be divided into two categories (*UFC 3-340-02*, Department of Defense (2014)).

- Primary Fragments are produced by the casing or objects in intimate contact with the explosive. These fragments are very small but travel at very high velocities.
- Secondary Fragments (or “debris”) produced by the blast wave interaction with objects and structures in close proximity of the explosive source.

For structural analysis, Conrath et al. (1999) suggest to either neglect fragments as the blast wave has the governing impacts throughout the design or implement an assumption for a pre-damaged concrete surface (exposed concrete face is assumed with spalls or craters) when the blast wave arrives at the structural surface.

Both types of fragments are not considered in this analysis as they only impact the upper surface of the concrete road deck and are assumed to have only a subsidiary impact.

2.1.7 Example Bridge

For this analysis of an explosion on top a steel girder bridge and the effects of structural deterioration on the bridge's resistance, a typical bridge has to be identified. In the following, the LRFD design example bridge will be used as it is specified in FHWA NHI-04-041 (2003). This assumption eliminates the need for an assessment of typical bridges and the exemplary design of a bridge for this report. The bridge implemented in the model is described in Chapter 3.

The blast cases have been identified through this literature review and not through an exemplary risk assessment as no parameters are available and too much input data would have to be assumed. The blast loading has been characterized in Sections 2.1.2 and 2.1.3.2.

2.2 Structural Response

Blast impacts present unique challenges for structures. Very short, highly impulsive impacts do not occur during any of the other loading types the structure was initially designed for. However, the structural response is very dependent on the rate of loading.

2.2.1 Structural System Behavior

Explosions present an extreme loading event with a great amount of energy to be dissipated by the structure in order to prevent failure or total collapse. Therefore, an elastic and inelastic response with large deformations are to be expected until the kinetic energy of the impact is dissipated with strain energy of deflection and/or partial or total collapse occurs due to fragmentation of concrete. Design provisions such as span length, element height and detailing of reinforcement determine the deflection capabilities of a reinforced concrete structural element. *UFC 3-340-02* (Department of Defense (2014)) describes the following structural behavior:

At a deflection associated with 2° (degree) support rotation, tension reinforcement has yielded and compressed concrete may begin to crush. Reinforced concrete elements without shear reinforcement will fail at this level of deflection.

Shear-reinforced structural elements have the capacity to transfer the compressive force to the compression rebar thus preventing failure until such members fail at about 6° (degree) support rotation.

If truss action can develop, the failure can be prevented until a support rotation of approximately 12° (degrees) is reached. This is only possible if lateral restraint is available to develop sufficient in-plane tensile forces.

Additionally, the shear capacity has to be adequate to prevent abrupt failure at lower loading levels due to shear so that the aforementioned flexural failure resistance can be utilized. The structural nonlinear response is also heavily dependent on the redundancy of the system, but no universal solution can be identified. Indeterminate structures have advantages with redistributing loads with their inherent capability to

create alternate load paths. However, they usually also restrain deformation and therefore might have a smaller capacity to dissipate energy with plastic deformations. Therefore, structures with greater structural ductility, larger spans and mass have an advantage. According to *UFC 3-340-02*, Department of Defense (2014), two modes of structural behavior can be identified: Ductile (associated with large deflections without complete collapse) and Brittle (partial failure or total collapse of the element).

In structural behavior to blast, ductile elements have a substantial advantage over brittle. Brittle materials fail abruptly while ductile members are capable of developing plastic hinges in regions of maximum moment. In reinforced concrete for example, a plastic hinge can develop, when the tension reinforcement yields and then compressed concrete is crushed and/ or compression reinforcement buckles *UFC 3-340-02*, Department of Defense (2014).

2.2.1.1 Failure Criteria

A variety of physical parameters are available to monitor the structural health of a structure during the process of blast loading. Commonly, either rotations or deflections are utilized as system performance indicators as they are easily obtainable from a structural simulation.

In this analysis, 2° support rotation as identified by *UFC 3-340-02*, Department of Defense (2014) is used as the failure criteria during the assessment for both concrete slabs and steel girders.

In addition, compression failure criteria of steel columns as suggested by Conrath et al. (1999) will be used during the analysis evaluation. ‘Light’, ‘Moderate’ and

‘Severe’ damage of the steel column are characterized by height shortening under compression of 2 %, 4% and 8 % respectively.

2.2.2 Element Response

Individual structural elements may have different response times than the overall system. While long-span elements are able to deform, short or stiff elements might fail abruptly. Flexural modes of element response are more favorable, the greater flexural ductility can dissipate more energy than shear deformations.

During the analysis of an element, two steps are of importance. First, the effects of the detonation on the particular member have to be understood and checked. And regardless of the response of the single member is (deflection, failure, etc.), the impact on adjoining elements and the effect on the overall structural integrity have to be assessed. The failure of a single element might lead to failure of one or more adjoining elements that did not structurally fail during the blast loading.

In this analysis, a full bridge model so that interactions between the individual elements are included automatically.

2.2.2.1 Global Response

Detonations at great standoff distances are expected to cause global element responses. This means that the design case can be associated with a set of assumptions for the structural design process. Elements subjected to such impacts are expected to resist with flexural response mechanism of the full length of the elements. Hinges

(either plastic hinges or support bearings) have to be available to allow for the adequate rotations.

A flexural response mechanism is very desirable as it is ductile and has the capability of dissipating great amounts of energy. Bending is also associated with lower strain rates thus demanding less ultimate resistance of elements. Damage is mainly induced by wave propagation resulting in concrete spalling on the far side and concrete crushing on the impacted face of the element.

However, great attention has to be paid to connection detailing as hinge support or beam slips from support can occur at lower loading level than shear or flexural failure. (Conrath et al. (1999))

2.2.2.2 Local Response

Close-in detonations are associated with a different element response type. Structural members subjected to more localized, non-uniform impacts have to resist localized direct (dynamic) shear resulting in punching shear response types. These impacts are typically linked with higher strain rates.

Other than from small standoff distances, local responses are also observed at points of geometric or load discontinuity such as cross-section variation or discrete ends of protective shielding of the element.

Elements subjected to close-in impacts have to be designed for large deflection and need specialized, advanced reinforcement detailing to ensure a ductile response. If this is not the case, the element is likely to fail due to the high-pressure concentrations in a brittle failure mode.

Concrete tends to react with spalling, cratering, scabbing as well as direct shear. Also, the hinges and supports have to be capable to resist the sudden shear stress induced by the detonation. (Conrath et al. (1999))

2.2.3 Material Properties and Strain Rate Effects

At high strain rates, material properties change considerably from the characteristic values under static loading. However, techniques are available to consider those complex strain rate effects and calculate the strength increase at a given strain rate. Neglecting these phenomena would be a source of inaccuracy because material ductility and strength are very important parameters in the bridge response and behavior to blast effects. Also, the strength increase contributes to a more economic design result.

UFC 3-340-02, Department of Defense (2014), propagates the introduction of two different factors for both concrete and steel.

The Dynamic Increase Factor (DIF) is defined as the ratio of Dynamic Material Strength to Static Material Strength. As strain rates vary significantly between static loading and those present in blast events, this factor adjusts material properties to the impulse loading case.

Strength Increase Factors (SIF) account for realistic material properties under dynamic loads. Additionally, Age Increase Factors can be used to account for concrete gaining strength beyond the nominal capacity at 28 days.

2.2.3.1 Reinforced Concrete

Even though concrete has to be classified as a brittle material, reinforced concrete demonstrates ductile behavior so that it can be considered as a ductile construction method. This performance is dependent on reinforcement bars being tied and anchored sufficiently.

The concrete strength has a great influence on the behavior of element, namely the shear capacity. In stronger concrete materials, shear reinforcement may be reduced. Also, at larger support rotations of elements, stronger concrete exhibits less cracking and crushing. Especially for elements with large support rotations of more than 2 degrees, the concrete strength only has a minor influence on ultimate strength of elements.

UFC 3-340-02, Department of Defense (2014), provides recommendations for minimum material qualities to be used for structural building materials in blast design. For concrete, a minimal compressive strength of $\min f'_c = 4$ ksi is recommended. Under no circumstances, the concrete material strength should be less than $f'_c = 3$ ksi. Research by Yi et al. (2013) shows, that higher concrete strength exhibits fewer numbers of failure mechanisms, however the likelihood of brittle failure during simulations at higher loads increases. Brittle failure should be avoided on a general principle. During the analysis, a compromise between high material strength and danger of brittle failure has to be identified.

For in-situ applications, performance testing of the existing concrete might be desirable as the in-place strength could be greater than the nominal strength. This increase in resistance is very beneficial for the analysis of the structural capacity.

A estimate of DIFs for design purposes are given in *UFC 3-340-02*, Department of Defense (2014). These values are more conservative for shear and bond than for bending and compression. This can be justified by the brittle failure mechanism associated with shear and bond failure which is less desirable and more dangerous thus a reduced resistance capacity can be accepted. For a preliminary design, DIF values can be obtained from the Table 4-1(*UFC 3-340-02*, Department of Defense (2014)). In the table below, the values for ‘Bending’ in the ‘Close-In’ design range are presented:

Table 1: Concrete and Reinforcement Steel DIF values

Type of Stress	Close-In Design Range		
	Reinforcing Bars		Concrete
	f_{dy}/f_y	f_{du}/f_u	f'_{dc}/f'_c
Bending	1.23	1.05	1.25

For more accurate values, the concrete DIF can be adjusted with respect to the actual strain rate. A higher strain rate results in a higher ultimate compressive strength of the concrete (for example, Figure 4-9 from *UFC 3-340-02*, Department of Defense (2014), not applied in this analysis).

No Strength Increase Factor (SIF) is applied: $SIF = 1.0$.

2.2.3.2 Reinforcement Steel

Reinforcement should be of Grade 60 with bars smaller than No. 11 bars for ductility purposes. Larger bars are undesirable because of the spacing and anchorage requirements. On a general basis, ductility and thereby resistance is reduced at bends and splices. Anchorages should not be avoided near the points of maximum stress.

For accurate calculations of ultimate resistance of an element, stress-strain relationships for the reinforcement should be known. Characteristic values for Grade 60 reinforcement (for ASTM A 615) are:

$$F_y = 60 \text{ ksi and } F_u = 90 \text{ ksi}$$

The SIF for the yield stress can be assumed as $SIF = 1.1$ so that the recommended design values are

$$F_y = 66 \text{ ksi and } F_u = 90 \text{ ksi}$$

The DIF values for the design process can be found in the table in Section 2.2.3.1. Again, for a more accurate and detailed analysis, strain rate dependent DIF values can be obtained (for example Figure 4-10 from *UFC 3-340-02*, Department of Defense (2014), not used in this analysis).

2.2.3.3 Structural Steel

Just as reinforcement bars, structural steel is strong and ductile. It is characterized by yield stress, ultimate tensile strength, elongation at rupture and modulus of elasticity. All of these properties vary with the duration of the impact compared to the natural frequency of the element thus being subject to strain rate effects.

Like in reinforcement, a yield strength increase of 10 % can be assumed for steel other than high strength steel, for which this assumption might be unconservative (UFC 3-340-02, Department of Defense (2014)).

$$SIF = 1.1 \quad \text{for } F_y = 50 \text{ ksi or less}$$

$$SIF = 1.0 \quad \text{otherwise}$$

The mechanical properties are illustrated in the plot shown below. The yield point increases significantly while the ultimate tensile strength only increases slightly. This is captured in two different DIFs for yield strength and ultimate tensile strength respectively. The modulus of elasticity and the elongation at rupture do not change significantly. The figure below presents the static and the dynamic stress-strain relationship.

Figure 5: Structural Steel Stress-Strain Curve from *UFC 3-340-02*, Department of Defense (2014)

The DIFs for yield strength and ultimate tensile strength for general design are tabulated for some materials. Table 5-2 from *UFC 3-340-02*, Department of Defense (2014) only lists Steel Materials A36, A588 and A514 while the design material values for the example bridge in this analysis suggest that a A572 Grade 50 has been used ($f_y = 50$ ksi and $f_u = 65$ ksi). The following dynamic increase factors (DIF) have been assumed for this analysis:

Table 2: Structural Steel DIF values

Material	Bending	Ultimate Stress DIF
	High Pressure	
	f_{dy} / f_y	f_{du}/f_u
A572 Grade 50	1.24	1.1

More accurate, strain-rate dependent DIFs for the yield strength of different steel materials can be taken from graph such as Figure 5-2 (UFC 3-340-02, Department of Defense (2014), not used for this analysis).

Generally, steel structures and members are not designed for excessive deflections so that strain-hardening of steel should not be occurring in the element.

Just as for concrete, a stronger material does not automatically result in better blast resistance. Son et al. (2012) have identified that medium-strength steel actually exhibits better behavior against blast loading than high-strength.

2.2.4 Bridge Design Specifications

The 6th Edition of the AASHTO *LRFD Bridge Design Specification* (2012) features a section on the loading scenario ‘Blast’. However, the manual only provides a qualitative list of relevant aspects during the design without quantitative provisions, suggestions or regulations. It states that results obtained from an equivalent static analysis should not be used for the design. A list of references is featured in the manual, for example:

- AASHTO’s Bridge Security Guidelines (2011)

- Baker et. al. (1983) ‘Explosion Hazards and Evaluation’
- Biggs’ (1964) ‘Introduction to Structural Dynamics’. This work presents dynamic analysis approaches for a large number of systems ranging from one-degree to multidegree systems and design applications. The chapter on ‘Blast-resistant Design’ focusses on Nuclear Explosions and Protective Structures.
- Bounds (1998) ‘Concrete and Blast Effects’
- Bulson (1997) ‘Explosive Loading of Engineering Structures’
- Conrath, et al. (1999) ‘Structural Design for Physical Security: State of Practice’
- Department of the Army, 1986 and 1990. The manual unfortunately does not specify these referenced documents any further. They could refer to publications ‘TM 5-855-1 : Fundamentals of Protective Design for Conventional Weapons’ (1986) and ‘TM 5-1300 : Structures to Resist the Effects of Accidental Explosions’ (1990, which has since been superseded by UFC 3-340-02 (Department of Defense, 2014)).

Another available design manual is ‘Blast-Resistant Highway Bridges: Design and Detailing Guidelines’ (NCHRP Report 645, 2010). It focusses solely on the substructure of the bridge, namely columns, but not the superstructure.

2.3 Simulation

To analyze blast events on structures, equations and/or numerical simulations are commonly utilized since large scale testing is very expensive and often times not

desirable or available. In the process, two tasks have to be accomplished. First, the blast event has to be simulated resulting in an energy and pressure wave output. This output then has to be applied to the structure as a load. The techniques to accomplish the simulation are outlined in the first part of this section.

Subsequently, this section introduces the software used for the numerical analysis and the approach for the validation of the model.

2.3.1 Simulation Techniques for Impulse Loading

For the simulation of blast-structure-interaction, many basic principal approaches are available. Each approach has an inherent level of the input data requirement, complexity of simulation and quality of output results. According to NCHRP Report 645 (2010), the approaches can be classified by three basic characteristics:

- Coupled/ Uncoupled Analysis
- Static or Dynamic Analysis
- Number of Degrees of Freedom (DOF)

The chart below compares different types of analysis type combinations. While a coupled, dynamic analysis with multiple degrees of freedom promises the most accurate simulation results, it is also the most complex, extensive and costly approach. This results in long computation durations and comprehensive input requirements. An uncoupled, static analysis with only one degree of freedom, in contrast, is simple and quicker, however the results must be expected to be less accurate.

Figure 6: Possible Analysis Method Combinations (from NCHRP (2010))

Categorically, it has to be stated, that every simulation regardless of its degree of sophistication can only be as good as the input data and the calibration. Therefore, a more complex analysis does not automatically yield better results. It is for the design engineer to decide on a good compromise of simulation complexity and simulation cost. For limited and uncertain input data available, a simple analysis might be more sensible since extensive analysis cannot guarantee better results.

2.3.1.1 First Principle/ Empirical Models

First principle methods solve problems with the use of basic laws of physics and materials. This has several limitations, because for an accurate analysis, comprehensive input data has to be specified which is very difficult. Data for atmospheric conditions, exact boundary conditions, material inhomogeneity and rates of reaction, among even more parameters, are challenging to assume for a design

purpose. Additionally, to establish confidence in the software, validation of the simulations should occur, which again is difficult.

In contrast, semi-empirical approaches establish a simple relationship between physical entities, require less computational effort and work well for the application they have been derived for during testing and experimenting. However, for applications outside of the calibration range, semi-empirical should not be utilized as their accuracy is highly questionable (NCHRP Report 645 (2010)).

Gündel et al. (2010) suggest that semi-empirical equations are oftentimes sufficient to predict the behavior of a structure.

2.3.1.2 Coupled/ Uncoupled Analysis

Uncoupled analysis means that blast wave and structural response are determined separately. The propagation of the wave is not effected by the structural response leading to conservative predictions of blast loads on structures. Structures are assumed rigid during the blast pressure calculation neglecting vent and pressure redistribution effects because of deflection or failure of individual members.

With a coupled analysis, smaller and more realistic pressures can be identified as local failure and deformation during loading are accounted for. However, this type of analysis requires more resources and experience as the implementation of the simulation is very complex and requires more computational resources (NCHRP Report 645 (2010)).

2.3.1.3 Static / Dynamic Analysis

Static analysis reduces the blast wave impact to an ‘equivalent static pressure’ that is being applied to the structure with no regards to time-history and inertia effects of the blast impact. Of course, this approach is very simple and can be completed by regular software utilized during the normal structural design process. However, the equivalent static pressure of a blast wave cannot be easily obtained as it depends on a large amount of factors such as type of explosive, geometric boundary conditions of the structure and the surrounding, blast wave reflections or material properties. Also, very limited historical data is available so that a pressure cannot be identified with confidence thus the result of this analysis must be interpreted and commented.

Dynamic analysis uses a time-varying blast load to design the structural element taking time-history effects into account. With this assumption, strain rate, inertia and mass effects can be considered during the analysis (NCHRP Report 645 (2010)).

2.3.1.4 Single / Multiple DOF

A single degree of freedom system reduces a complex system such as a beam to a single spring-mass-damper system. This system can be analyzed easily and results can be back-calculated to the original structure so that predictions for the system behavior can be made.

Multiple degree of freedom analysis ranges from simple 2-D frames to complex 3-D finite element systems. The level of sophistication is only limited by the computational capacities (NCHRP Report 645 (2010)).

2.3.2 Software

For this analysis, the software ABAQUS (Version 6.14 & 6.16, Dassault Systems) are used. The models were built in the 6.14-Version. Since a large number of simulations had to be run, the actual analysis was performed with the teaching suite of ABAQUS 6.16. Therefore, all the results are determined using the 6.16 Version.

For modelling of the blast loading, the CONWEP model was used as it is embedded in the ABAQUS software as one type of ‘shock loadings’. For the definition of an air blast scenario, an incident shock wave using the CONWEP model can be implemented. This model uses empirical data to determine a shock wave which is applied to structural surfaces from the mass of the explosive in TNT equivalence weight and the specification of the three-dimensional location of the center of the explosive charge. The total pressure on the structural surface is determined from the incident pressure, the reflected pressure and the angle of incident of the blast wave on the structure.

The simulation in this analysis can therefore be characterized as an empirical, uncoupled, dynamic analysis with multiple degrees of freedom.

The actual analysis inputs are described in Chapter 3.

2.3.3 Verification/ Validation

In order to check the output data obtained from a simulation, the calibration should be tested with experimental data or other proven and tested methods. Without any verification available, the exactness of the simulation results for a unique and

unusual application like blast design cannot be assumed and the reliability of the numerical results has to be commented.

Therefore, it is of high importance to identify a method of validating output data in order to establish confidence in the basis for the conclusions and recommendation.

2.3.3.1 Displacement

In order to check the model and boundary condition, the displacement obtained from the simulation software has to be checked. Since no historical/ experimental data is available and it difficult to check the dynamic displacements due to the impulse load without the development of another model or other simplifications, only the displacement caused by the static load (deadweight and lane load) will be checked with hand calculation equations. This will be presented in Chapter 4 and commented in Chapter 5.

2.3.3.2 Blast Loading

A very critical model input is the actual blast loading output of the CONWEP model in the ABAQUS software. In Chapter 4, the loading of the structure will be compared to hand-calculation approximations for blast as presented by *UFC 3-340-02* (Department of Defense, 2014). The software output is expected to produce loadings of similar magnitude.

CHAPTER 3: METHODOLOGY

In this chapter, the analysis approach, example bridge, material properties and simulation inputs are presented.

3.1 Research Approach

To investigate the effects of bridge deterioration on its blast resistance, the finite element simulation software ABAQUS is used. To assess the explicit effects of progressive section loss, a number of test models are generated varying only in section thickness of deck and superstructure elements. As none of the other input values change, observed differences in structural response can be directly interpreted as the impact of bridge deterioration on the blast resistance.

3.1.1 Structural Steel Section Deterioration

The *AASHTO Bridge Element Inspection Guide Manual* (2010) identifies steel section loss due to corrosion as the main result of deterioration in structural steel bridge elements. This is the only steel element characteristic considered in this research. Other likely phenomena observable in aging structures include fatigue, cracking and the worsening state of the connections.

In the numerical analysis, the following deterioration scenarios are examined. The first five cases (STEEL 0 to 4) are drawn up to understand the effects of localized deterioration as well as to understand relative importance of each section area towards the overall structural resistance.

Table 3: Steel Deterioration Cases 0 to 4

CASE	Deterioration
STEEL 0:	no section loss
STEEL 1:	10 % bottom flange, 5 % top flange section loss near the supports (3 ft zone around supports)
STEEL 2:	5 % web and bearing stiffener section loss near the supports (3 ft. zone around supports)
STEEL 3:	10 % bottom flange, 5 % top flange section loss in the span (zone located at least 3 ft. away from support)
STEEL 4:	5 % web and stiffener section loss in the span (zone located at least 3 ft. away from support)

The second set of deterioration scenarios (STEEL 5, 10 & 20) study bridges where different magnitudes of section loss have occurred along the entire girder. The objective of this case study is to quantify the progress of deterioration and study the effects of minor versus extensive deterioration.

Table 4: Steel Deterioration Cases 5, 10 and 20

CASE	Deterioration
STEEL 5	5 % bottom flange (entire girder length), 2.5 % top flange section loss (entire girder length) and 2.5 % web, stiffener, cross-frame and bearing stiffener section loss
STEEL 10	10 % bottom flange (entire girder length), 5 % top flange section loss (entire girder length) and 5 % web, stiffener, cross-frame and bearing stiffener section loss
STEEL 20	20 % bottom flange (entire girder length), 10 % top flange section loss (entire girder length) and 10 % web, stiffener, cross-frame and bearing stiffener section loss

3.1.2 Concrete Deck Deterioration

The *AASHTO Bridge Element Inspection Guide Manual* (2010) lists ‘cracking’, ‘spalls/ delamination/ patched areas’, ‘efflorescence’, and ‘load capacity’ for condition state assessment for ‘reinforced concrete deck/ slab’ elements.

In this study, the effects of spalls and delaminations will be studied. Three deterioration severity scenarios are examined. Deck 0 has no deterioration, Decks 11-16 have a one inch (11.8 % of 8.5 in. deck slab height) of section loss due to spalls and delamination from the top surface to represent a ‘Moderate’ defect as defined by the *AASHTO Bridge Element Inspection Guide Manual* (2010). Decks 21-26 are reduced by 2.5 inch (29.4 % of 8.5 in. slab thickness) due to spalling and delamination from the top. In this case the top reinforcement is exposed and the defect has to be characterized as ‘Severe’ according to *AASHTO Bridge Element Inspection Guide Manual* (2010).

Also, the extent of deterioration spreading across the bridge deck is varied. The first scenario (Deck Cases 11 & 21) is one patch of (10 x 50 ft.) around the center of the span and in the one traffic lane at the location of the blast loading. In this scenario, a total amount of 4.7 % of the total deck surface is assumed to have deteriorated.

Figure 7: Deck Deterioration, one Patch

The second scenario (Deck Cases 12 & 22) has 2 patches (same size as before), one located at the center of the span and around the central support pier location in the lane subjected to the blast load. This results in a total of 9.4 % deck area deterioration.

Figure 8: Deck Deterioration, two Patches

Deck Cases 14 & 24 have four patches, located at mid-span and above the central support in each of the traffic lanes (14.1 % of deck surface has deteriorated).

Figure 9: Deck Deterioration, four Patches

Deck Cases 16 & 26 have six patches of deterioration. The locations are at the center of both spans in both lanes (four patches) and above the pier (18.8 % of total area deteriorated).

Figure 10: Deck Deterioration, six Patches

Table 5: Deck Deterioration Cases Overview

CASE	Deck Deterioration
Initial Bridge	
DECK 0	no deterioration
‘Moderate’ Deterioration	
DECK 11	1 inch section loss in one patch as specified above.
DECK 12	1 inch section loss in two patches as specified above.
DECK 14	1 inch section loss in four patches as specified above.
DECK 16	1 inch section loss in six patches as specified above.
‘Severe’ Deterioration	
DECK 21	2.5 inches section loss in one patch as specified above
DECK 22	2.5 inches section loss in two patches as specified above
DECK 24	2.5 inches section loss in four patches as specified above
DECK 26	2.5 inches section loss in six patches as specified above

3.2 AASHTO LRFD Guide Example Bridge

For the study of impact of bridge deterioration on blast resistance, a realistic example bridge design is required with typical element dimensions. In the following analysis, the steel girder bridge as checked in the Federal Highway Administration’s Report FHWA NHI – 04-041 (‘LRFD Design Example for Steel Girder Superstructure Bridge’, 2003) is utilized. The FHWA report shows that the bridge design satisfies the AASHTO LRFD Bridge Design Specifications (Second Edition, 1998, including interims for 1999 through 2002). The design process and checks are not repeated in this report. Instead, the final design solution of the FHWA report is presented in the following subsections and implemented into the finite element analysis with some adaptations.

3.2.1 General Information

The example bridge is a two span, square, and continuous structure. Two 12-foot lanes and two 10-foot shoulders make up the bridge width of 44 feet (curb to curb). Each span is 120 feet, so the entire bridge length is roughly 240 feet (two 120-foot spans plus two 8-inches of bridge excess lengths beyond the two abutment bearings). Live load requirement HL-93 was assumed for the structural design.

Figure 11: Static System, Example Bridge (FHWA, 2003)

Figure 12: Superstructure Cross Section, Example Bridge (FHWA, 2003)

3.2.2 Concrete Deck

The road deck consists of a reinforced concrete slab spanning the bays between the 5 girders and cantilevering over the outer two girders supporting a parapet structure at both perimeters.

The slab has a design thickness of $t_s = 8.5$ in between the girders, the overhang thickness is $t_o = 9.0$ in. The deck top cover is $\text{Cover}_t = 2.5$ in, the concrete deck bottom cover is $\text{Cover}_b = 1.0$ in.

The reinforcement design distinguishes between positive and negative superstructure moment (composite cross section made up of both girder and deck concrete). Without conducting a thorough investigation to identify the exact location of the neutral axis and the exact load transfer between the girder and the deck, it can be approximated that in the positive moment region significant areas of the concrete deck are in compression while in the negative moment region, the concrete is located in the part of the composite section subjected to tension. Therefore, the longitudinal reinforcement specifications differ according to the aforementioned types of stress distributions along the length of the bridge. In the model, the negative moment reinforcement is assumed in an area of 60 feet around the central support. The two outer 60 feet area of the cross section are modeled with positive moment reinforcement specifications.

Figure 13: Positive and Negative Deck Moment Regions

Reinforcement in the positive moment superstructure deck region:

Table 6: Positive Moment Deck Reinforcement

Reinforcement Type/ Location	Bar Size	Spacing
Top, longitudinal (direction of girder span)	# 4	@ 10 in.
Top, transverse (direction of slab span)	# 5	@ 6 in.
Bottom, longitudinal	# 5	@ 10 in.
Bottom, transverse	# 5	@ 8 in.

Reinforcement in the negative moment superstructure deck region is models in a zone of 60 ft. around the central support.

Table 7: Negative Moment Deck Reinforcement

Location	Bar Size	Spacing
Top, longitudinal (direction of girder span)	# 5	@ 5 in.
Top, transverse (direction of slab span)	# 5	@ 6 in.
Bottom, longitudinal	# 5	@ 5 in.
Bottom, transverse	# 5	@ 8 in.

Of course, the deck also has positive and negative moment sections (perpendicular to the girder direction) as it continuously spans between girders and overhangs on both sides of the bridge. However, the structural deck resistance or rather the reinforcement specifications do not vary between positive and negative deck moment regions as they are in close proximity. An exception to this is the overhang design. Subjected to a bigger negative moment as (i) the parapet weight imposes a big load on the overhang, but even more, the (ii) different static model configuration of a cantilever, the deck overhang has not only a greater thickness but also additional top reinforcement (only orthogonal to the girder span direction). In the model, the parapet, the increase in slab thickness, and the additional reinforcement is neglected as the overhang configuration will not be studied in Chapters 4 and 5.

In the ABAQUS model, the concrete deck is modeled with 3D-deformable, planar shell elements (S4R) with reinforcement as specified below in tables below. The reinforcement position is measured from the center line of the shell thickness element.

DECK 0 consists of slab elements with a thickness of 8.5 in (216 mm), 'Concrete' material, and reinforcement in the negative and positive moment superstructure regions as specified in the tables of Appendix A.2.

DECK 11, 12, 14, and 16 are also modeled with slab elements, however, surface deterioration and therefore section loss has occurred at the top surface. The thickness of the slab is reduced by 1.0 in. (top concrete cover) to 7.5 inches (191 mm). The material and reinforcement do not change, however since the slab thickness is reduced moving the geometric center line, the specification of the reinforcement geometry has to be adjusted. Reinforcement input for positive and negative superstructure moment

regions is presented in tables of Appendix A.2. The fractions of the bridge deck surface which is assumed to have deteriorated in each of the different deck scenarios have been identified and listed in Section 3.1.2.

DECK 21, 22, 24, and 26 are slab elements as well, but the concrete cover on the road surface has deteriorated leaving the top reinforcement exposed. The thickness of the slab is reduced by 2.5 in. (top concrete cover) to 6 inches (152 mm). The material and reinforcement is the same as in DECK 0, but the reinforcement geometry has to be adjusted similarly to DECK 1. Reinforcement input for positive and negative superstructure moment regions is presented in tables of Appendix A.2. The location of the deteriorated patches has been presented in Section 3.1.2.

3.2.3 Steel Girder

The example bridge's superstructure consists of 5 steel girders, each with 2 spans of 120 spans. The web has constant dimensions along the entire girder length, the flange thickness increases around the pier support. The bridge is symmetrical about the centerline of the pier support.

Figure 14: Steel Girder Properties, Example Bridge (FHWA, 2003)

Between the top flange and the concrete deck, a haunch with a depth of 3.5 inch is specified. However, this haunch is neglected in the ABAQUS model, and the haunch depth is set to zero in the analysis.

Another simplification made for easier implementation of the model is in the girder top flange alignment. In the example bridge, the web has a constant height of 54.0 in. through the entire girder length while the top of the top flange varies in its geometrical height. In the gap between the top flange and the theoretical bottom face of the deck concrete, the haunch is located to cover the shear connectors. In the model, the height of the top surface of the top flange is kept constant. This means, that the deck can be positioned right on the top flange (neglecting the haunch and the small height difference between the top flange and concrete deck). But, the web height is therefore not constant along the length of the girders as it connects the bottom flange (constant height) to the bottom of the top flange (not constant as explained above). The girder implemented in the analysis model is presented in the figure below.

Figure 15: Steel Girder Properties, Model Input

3.2.3.1 Web and Stiffeners

The web and flange thicknesses for the different deterioration cases are implemented in the finite element model as assumed below. The thicknesses are given in inch [in.] in the first column and Millimeter [mm] in the columns below.

Table 8: Web, Stiffener and Cross Frame Thicknesses

Scenario	Web Thickness		Bearing Stiffener Thickness	Intermediate Stiffener Thickness	Cross Frame Thickness
	3 ft. zone around supports	Distance to support greater than 3 ft.			
STEEL 0	½ in.		11/16 in.	½ in.	
	12.7 mm		17.5 mm	12.7 mm	
STEEL 1	12.7 mm		17.5 mm	12.7 mm	
STEEL 2	12.1 (-5 %)	12.7	16.6 (-5 %)	12.7	
STEEL 3	12.7		17.5	12.7	
STEEL 4	12.7	12.1 (-5 %)	17.5	12.1 (-5 %)	
STEEL 5	12.4 (-2.5 %)		17.0 (-2.5 %)	12.4 (-2.5 %)	
STEEL 10	12.1 (-5 %)		16.6 (-5 %)	12.1 (-5 %)	
STEEL 20	11.4 (-10 %)		15.7 (-10 %)	11.4 (-10 %)	

The intermediate stiffeners are placed in accordance with *LRFD Design Example for Steel Girder Superstructure Bridge* (AASHTO, 2003). On the fascia girders, stiffeners are only placed on the interior side of the web. On the three internal girders, intermediate stiffeners are placed at both sides of the web.

Figure 16: Placement of Intermediate Stiffeners

At the spacing of 6'-9", 5 stiffeners are placed at either side of the central support (identifiable by the cross frame in the picture below). One Intermediate Stiffener is not in the display as it is part of a cross frame). Two more intermediate stiffeners are located 6'-9" inside of both abutment bearings. The outer girder is not displayed in the picture below so that the int. stiffeners can be identified.

Figure 17: Location of Intermediate Stiffeners

Bearing stiffeners and the cross-frame design are discussed in the Miscellaneous Steel Section 3.2.3.4.

3.2.3.2 Flanges

In the table below, the bottom flange thicknesses are listed. The flange of each span is divided into 5 sections. The flange named '84 ft.' is the section spanning from the abutment support. Flange '12 ft.' is located over and around the central pier support. Bottom flange '24 ft.' connects the aforementioned two sections as it placed between the two and is not adjacent to any support.

In the model, the bottom flange thickness is being reduced by geometrically reducing the distance between the bottom face and the bottom of the web. The top face

of the bottom face remains at a constant height throughout all deterioration scenarios in the assembly model. Thusly, the web does not theoretically get longer as the internal faces of the two flanges do not move.

Table 9: Bottom Flange Thicknesses

Scenario	Bottom Flange 84 ft		Bottom Flange 24 ft	Bottom Flange 12 ft	
	3 ft. zone around supports	Distance to support greater than 3 ft.		3 ft. zone around supports	Distance to support greater than 3 ft.
STEEL 0	7/8 in.		1.375 in.	2.75 in.	
	22.2 mm		34.9 mm	69.9 mm	
STEEL 1	20.0 (-10 %)	22.2	34.9	69.9	62.9 (-10 %)
STEEL 2	22.2		34.9	69.9	
STEEL 3	22.2	20.0 (-10 %)	31.4 (-10 %)	62.9 (-10 %)	69.9
STEEL 4	22.2		34.9	69.9	
STEEL 5	21.1 (-5 %)		33.1 (-5 %)	66.3 (-5 %)	
STEEL 10	20.0 (-10 %)		31.4 (-10 %)	62.9 (-10 %)	
STEEL 20	17.8 (-20 %)		27.9 (-20 %)	55.9 (-20 %)	

The top flange geometry assumptions are presented in the next two tables. The names are assigned in the same way as for the bottom flange. For this flange, section reduction is modeled as a flange width reduction in contrast to the thickness reduction (web, stiffeners and bottom flange). The top flange thickness remains constant

throughout all of the deterioration scenarios, the top flange widths are presented in table below.

Table 10: Top Flange Thicknesses

Scenario	Top Flange 84 ft	Top Flange 24 ft	Top Flange 12 ft
STEEL	5/8 in.	1.25 in.	2.5 in.
0, 1, 2, 3, 4, 5, 10, 20	15.9 mm	31.8 mm	63.5 mm

Table 11: Top Flange Widths

Scenario	Top Flange ‘84 ft’ Width		Top Flange ‘24 ft’ Width	Top Flange ‘12 ft’ Width	
	3 ft. zone around supports	Distance to support greater than 3 ft.		3 ft. zone around supports	Distance to support greater than 3 ft.
STEEL 0	14 in.				
	355.6 mm				
STEEL 1	337.8 (-5 %)	355.6		337.8 (-5 %)	
STEEL 2	355.6				
STEEL 3	355.6	337.8 (-5 %)		355.6	
STEEL 4	355.6				
STEEL 5	346.7 (-2.5 %)				
STEEL 10	337.8 (-5 %)				
STEEL 20	320.0 (-10 %)				

3.2.3.3 ABAQUS Model

In ABAQUS, the steel girders are modeled with solid elements (C3D8R). The geometry is rounded to the nearest 1/10 millimeter as presented in tables above.

Figure 18: Model of Steel Elements

The material is defined in the Structural Steel Section 3.3.3.

3.2.3.4 Miscellaneous Steel

Among other features, the FHWA report characterizes the cross-frame and bearing stiffeners as miscellaneous steel. The material ‘Structural Steel’ is used for these solid elements (C3D8R).

Figure 19: Bearing Stiffener and Cross Frame Model

Since no detailed cross-frame design is given in the FHWA report, a simplified plate connection is used in the model. The plate spans from stiffener to stiffener with a height of 0.50 m and a thickness of 12.7 mm. Five cross-frames are positioned in each of the spans at a spacing of 20 feet.

Figure 20: Cross-Frame Model

The bearing stiffener's thickness has already been listed above. They are located at both sides of the flange above each of the supports

Figure 21: Bearing Stiffener at Abutment Support

Figure 22: Bearing Stiffener at Pier Support

3.2.5 Assumptions

A number of assumptions were made during the assembly of the example bridge.

3.2.5.1 Parapet Structure

The parapet structure is neglected in the blast analysis bridge model. Therefore, mechanisms of the blast wave interacting with the parapet, which could possibly influence the side overhang of the bridge deck, are not considered. As described in the concrete deck Section 3.2.2, the overhang slab thickness and reinforcement increases were therefore neglected as well.

3.2.5.2 Connections

All connections between the elements are modeled with ‘ties’. For the structural steel elements of the girder, this is assumed to be valid as the connections are welded. The connection between the steel girder and the concrete deck is actually formed by shear stiffeners. Therefore, the assumption of a tie may not represent the load transfer very well.

Also, the example bridge has two bolted connection at either side of the central support. In the model, these connections are neglected and the girder is modeled as a continuous section. Likewise, the cross-frame possibly has bolted connections (no actual design provided in the *LRFD Design Example for Steel Girder Superstructure Bridge* (2003)).

3.2.5.3 Load Bearing and Substructure

Load bearing and substructure are not modeled in the ABAQUS input file. The support in the model only consists of displacement boundary conditions at the idealized bridge support locations. It can rotate freely at all supports.

At the center support, the movement of the structure is restricted in all three directions as it is held at the bottom of the bottom flange.

At the abutment bearing, vertical and sideways translations are restricted. However, the bridge is able to move in the bridge span direction.

3.3 Material Properties

The ABAQUS software does not provide an inherent materials database, instead materials have to be inputted manually by the user. The software offers a variety of characterization options so that a wide range of materials can be created and specified for the individual application and only material parameters relevant for the analysis need to be provided.

Three materials are used in the model for this study:

- Concrete ($f'_c = 4$ ksi)
- Reinforcement Steel (A615 Grade 60)
- Structural Steel (A572 Grade 50)

In the following sections, the material input into the software is outlined. For each material type, some general, elastic, and plastic properties are required. The material behavior during a blast event and the material property modifications have been presented in the literature review in Chapter 2.

3.3.1 Concrete

The concrete used in the example bridge is characterized by the Concrete 28-day compressive strength $f'_c = 4.0$ ksi. The material adjustment for high strain rates has been presented in Chapter 2.

General material properties

Table 12: Concrete General Material Properties

Material Property	US Units	ABAQUS (SI Units)
Density	150 kcf	2402.76945 kg/m ³

Elastic Behavior

Table 13: Concrete Elastic Behavior

Material Property	US Units	ABAQUS (SI Units)
Young's Modulus	4792.8 ksi	3.30 E+10 kg/m ³
Poisson's Ratio	0.2	0.2

Plastic Behavior

From the ABAQUS material specification options, Concrete Damaged Plasticity (CDP) is used for concrete modeling in the analysis. It has the capability to simulate inelastic compression and tension behavior.

Plasticity Input

Table 14: Concrete Plasticity General

Dilation Angle	Eccentricity	f_{b0}/f_{c0}	K	Viscosity Parameter
36	0.1	1.16	0.667	0

Compressive Behavior

Table 15: Concrete Plasticity Compressive Behavior

Yield Stress [Pa]	Inelastic Strain [-]
32750105.66	0
34473795.43	0.0025
344737.9543	0.02

Tensile Behavior

The tensile strength of concrete is expected to not influence the simulation results in a significant way. However, to provide a complete concrete model in the analysis, the concrete tensile strength is determined according to Wight and MacGregor (2009). For concrete $f'_c = 4.0$ ksi, it is found to be:

$$\overline{f_{ct}} = 6.4 * \sqrt{f'_c} = 6.4 * \sqrt{4000} = 0.405 \text{ ksi}$$

$$\overline{f_{ct}} = 0.405 \text{ ksi} = 2.79 * 10^6 \text{ Pa}$$

The cracking strain is assumed as 2 ‰, a linear stress strain relationship is assumed. The tensile stresses are not modified by dynamic or strength increase factors for the analysis.

Table 16: Concrete Plasticity Tensile Behavior

Yield Stress [Pa]	Cracking Strain [-]
2.79E+06	0
2.79E+03	0.002

The CDP model options of concrete compression and tension damage are not employed in the material characterization. The failure criteria are related to the maximum (first) dynamic deflection/ rotation after the detonation and do not depend on the damage which this reflection caused in the structure.

3.3.2 Reinforcement Steel

The reinforcement steel used for the concrete deck of the example bridge is characterized by a Yield Strength of $f_y = 60$ ksi. A tensile strength is not specified. In the following analysis, the steel A615 Grade 60 is assumed.

General material properties

Table 17: Reinforcement Steel General Properties

Material Property	US Units	ABAQUS (SI Units)
Density	490 kcf	7849.05 kg/m ³

Elastic Behavior

Table 18: Reinforcement Steel Elastic Behavior

Material Property	US Units	ABAQUS (SI Units)
Young's Modulus	29000 kcf	1.99948 E+11 Pa
Poisson's Ratio	0.3	0.3

Plastic Behavior

The focus of the study regarding the concrete slab is the support rotation for a close-in detonation. Therefore, dynamic increase factors (DIF) associated with bending (close-in case) are used for modifying the material properties. A strength increase factor (SIF) table was presented in the literature review. The property modification can be found in table below.

Table 19: Reinforcement Steel Property Modification

Property	Steel A615 – Grade 60	
	Yield Strength	Tensile Strength
F_y / F_u [ksi]	60	90
DIF	1.23	1.05
SIF	1.1	1
F_{dy} / F_{du} [ksi]	81.18	94.5
F_{dy} / F_{du} [Pa]	5.60 E+08	6.52 E+08

In ABAQUS, the ‘Plastic’ material input option was chosen. Here, the yield stress with regards to the plastic strain has to be specified. The adjusted material properties were used for the yield stress estimations.

Table 20: Reinforcement Steel Plasticity Input

Yield Stress [Pa]	Inelastic Strain [-]
554000000	0
560000000	0.017931034
652000000	0.15
586000000	0.197931034

3.3.3 Structural Steel

The structural steel used for superstructure elements such as girders, stiffeners and cross frames is characterized by a Structural Steel Yield Strength $F_y = 50$ ksi and a Structural Steel tensile Strength $F_u = 65$ ksi. These characteristic values correlate with steel A 572 Grade 50.

Similar to the material input description for reinforcement steel, general, elastic, and plastic material properties have been specified.

General material properties

Table 21: Structural Steel General Material Properties

Material Property	US Units	ABAQUS (SI Units)
Density	490 kcf	7849.05 kg/m ³

Elastic Behavior

Table 22: Structural Steel Elastic Behavior

Material Property	US Units	ABAQUS (SI Units)
Young's Modulus	29000 kcf	1.99948 E+11 Pa
Poisson's Ratio	0.3	0.3

Plastic Behavior

The failure criteria applied in the superstructure elements can be associated with a bending mechanism. Therefore, 'bending' dynamic increase factors (DIF) are used to modify the properties. Steel A 572 Grade 50 assumptions for DIF and SIF are listed in the tables of Section 2.2.3..

Table 23: Structural Steel Material Modification

Property	Steel A572 – Grade 50	
	Yield Strength	Tensile Strength
F_y / F_u [ksi]	50	65
DIF	1.24	1.05
SIF	1.1	1
F_{dy} / F_{du} [ksi]	68.2	71.5.0
F_{dy} / F_{du} [Pa]	4.70 E+08	4.93 E+08

Again, the 'Plastic' input option was chosen (Mechanical-Plasticity-Plastic) in ABAQUS. The yield stress to plastic strain relationship is inputted into the model with values presented in table below.

Table 24: Structural Steel Plasticity Input

Yield Stress [Pa]	Inelastic Strain [-]
4.66E+08	0
4.70E+08	0.018275862
4.93E+08	0.15
4.44E+08	0.198275862

3.4 Bridge Loading and Boundary Conditions

During the blast event, a number of different loading types act on the structure. Besides the impact of the blast event, gravity cannot be neglected and traffic might also be present on the bridge. The *AASHTO LRFD Bridge, Design Specifications* (2012) provide guidelines regarding permanent and transient loads which are established in this section. The blast load has already been discussed in the literature review in Chapter 2.

3.4.1 Extreme Event II

For the analysis, Load Combination Limit State ‘Extreme Event II’ as defined by the *AASHTO LRFD Bridge, Design Specifications* (2012) is employed. It is the applicable load combination related to blast. It allows for a reduced live load and disregard of a number of other typical bridge loads (e.g. wind,

Table 25: Load Combination - Extreme Event II (AASHTO (2012))

Load Combination Limit State	DC, DD, DW, EH, EV, ES, EL, PS, CR, SH	LL, IM, CE, BR, PL, LS	WA	FR	BL
Extreme Event II	γ_p	0.50	1.00	1.00	1.00

3.4.2 Permanent Load

The dead load (DC) of the structure is automatically determined and applied by the software as 3-dimensional elements were and unit weights were input. The load factor γ_p is defined in Table 3.4.1-2 (*AASHTO LRFD Bridge, Design Specifications* (2012)). Depending on the impact the deadweight has on the structural response (magnifying or diminishing e.g. the center span deflection), a maximal and minimal load factor are specified and have to be assumed for the design check ($\gamma_{p,max} = 1.25$ and $\gamma_{p,min} = 0.90$).

In this analysis, $\gamma_p = 1.00$ is assumed. It will not be assessed, what parts of the bridge have what influence on the dynamic response such as resistance and damping during this dynamic simulation. Also, the bridge input can be assumed as fairly accurate to that the uncertainty regarding the structural dimensions is small. The load factor is within the boundary maximum and minimum value specified in the *AASHTO LRFD Bridge, Design Specifications* (2012).

The unit weights can be found in the materials specification, the bridge geometry is presented in the Example Bridge (Section 3.2).

The dead load of the wearing surface (DW) is assumed to be part of the concrete deck and is not considered separately.

Drag forces due to creep (CR), horizontal earth pressure loads (EH), vertical pressure from dead load of earth fill (EV), earth surcharge load (ES), miscellaneous locked-in forces (EL), secondary forces from post-tensioning (PS), and force effects due to shrinkage (SH) are not considered in the analysis.

3.4.3 Transient Loads

Vehicular Live Loads (LL) are assumed in the form of a lane load of 0.064 ksf and is uniformly distributed across the entire bridge deck. The design truck and design tandem loads will not be applied during the analysis. The Dynamic Load Allowance (IM) is assumed in accordance with Table 3.6.2.1-1 of *AASHTO LRFD Bridge, Design Specifications* (2012), IM = 33 %, but will not be multiplied to the static load (LL) as truck or tandem loads not used. The total transient load applied in the analysis model is computed by multiplying the load factor and the vehicular live load:

$$\text{Transient Load} = 0.50 * 0.064 = 0.032 \text{ ksf}$$

Since the ABAQUS model is run with metric input values, the Transient Load is converted into Pascal.

$$\text{Transient Load} = 0.032 \text{ ksf} = 1532.2 \text{ N/m}^2$$

Vehicular centrifugal forces (CE), vehicular braking forces (BR), pedestrian live loads (PL), live load surcharge (LC), water loads and stream pressure (WA), and friction loads (FR) are not considered in the analysis.

3.4.4 Blast Loading (BL)

Blast loading (BL) are not specified in *AASHTO LRFD Bridge, Design Specifications* (2012). The magnitudes of charge weights and possible blast locations have already been presented in the literature review.

The ABAQUS software has the capability to model both air and surface blast events as an ‘Incident Wave’ in the ‘Interaction’-Module. This feature utilizes the CONWEP code as described in the literature review. The blast load in ABAQUS has

to be specified only by the geometrical location of detonation, the surface impacted by the incident wave, and the TNT equivalent weight of the explosive charge. The code automatically converts these descriptions into a time-dependent pressure input to the structure.

Also, options for ‘time of detonation’ and a ‘magnitude scale factor’ are available. In this analysis, detonation occurs at the beginning of the simulation, the time of detonation is therefore set to zero. The magnitude scale factor is set to one.

The following two cases are analyzed:

Table 26: Blast Loading Cases

Name	Charge weight and position
BLAST 1	600 pounds charge, center of directional lane, center of span
BLAST 2	600 pounds charge, center of directional lane, above the central support

3.4.5 Boundary Conditions

Each girder of the AASHTO example bridge has three points of vertical support. In the 2-dimensional model, the two supports located at either end of the girder provide vertical bearing and the central support above the bridge pier, fixes the girder both vertically and horizontally.

In the realistic configuration, the substructure and the actual materials and formats of the bridge bearings, cannot be assumed as completely fixed. For example, the bearing material has a certain modulus of elasticity allowing a certain amount of vertical movement and friction on the outer bearings will influence the horizontal movements. Additionally, the structure has to be held in the third dimension

orthogonal to the bridge span and the multi-dimensional bearing will introduce some torsional resistance.

In the model, the bridge bearings are implemented as line deformation restraints. At the two abutment supports, the girder is held vertically and perpendicular to the span. The central support is modeled as a fixation of the bottom flange in all three spatial directions. Rotations are not restrained at any of the supports.

3.5 Simulation Input

In ABAQUS, the elements described above are merged into one model in an assembly module. Also, the type and duration of the analysis has to be specified.

3.5.1 Analysis Type

The ABAQUS analysis type chosen for the study is ‘Dynamic, Explicit’. The *Abaqus Analysis User’s Guide* (Dassault Systemes, 2014) lists that this analysis type “is computationally efficient for the analysis of large models with relatively short dynamic response times” and “uses a consistent, large-deformation theory” so that “models can undergo large rotations and large deformations”. Therefore it appears suitable and capable to run the problems of this study.

3.5.2 Analysis Duration

In the analysis, the maximum deflection is required to check for critical rotations within the structural elements deck and girder. The maximum deflection occurs as a direct result of the first impact of the pressure wave to the bridge deck. Therefore, even though the highly dynamic impact causes the bridge to oscillate for some time,

the maximum and therefore critical deflection is expected to occur within the first couple hundredth of a second after the detonation. The duration of the simulation period is therefore set to two-tenth of a second (0.2 seconds).

If a clear peak deflection cannot be identified within this analysis duration, a longer period (0.2 seconds after the detonation or longer) has to be run in the simulation software.

To verify this assumption, an analysis with the duration of 10 seconds is run and checked for the maximum deflection.

CHAPTER 4: FINDINGS

In this chapter, the simulation results are presented and analyzed. In the first section, the response of the example bridge to the different blast loadings is introduced. Afterwards, the influence of deterioration in the different scenarios is analyzed.

The response of the bridge is highly dynamic and time dependent. However, since the difference in the model between the scenarios is very small, a similar characteristic behavior is expected from all bridges varying only in amount of displacement. The undeteriorated bridge is set as the benchmark for the evaluation of the effects of bridge section reduction and the output data sets will be compared. Every difference in response and resistance must be caused by and can therefore be linked directly to the section reductions in the different deterioration scenarios.

4.1 Initial Bridge (No Deterioration)

The analysis of the initial bridge is divided into two parts. The girder (or superstructure) is a ductile structural member and the displacements are highly time-dependent. On the other hand, the reinforced concrete slab (road deck) is also considered to be a ductile member; however, the deformations turn out to be of a more permanent nature.

4.1.1 Concrete Deck

The concrete deck has two structural functions in the load resistance concept of the composite bridge. For vertical loads acting on the deck such as the traffic, it serves as the road surface and transfers load from every location of the bridge deck surface in the transverse direction to the closest girder. As the compression zone of the composite beam structure, it then participates in transferring the loads to the bridge bearings, together with the steel.

Therefore, two things are to be looked at during the evaluation of the bridge deck behavior during blast:

- 1) The vertical displacements in the transverse deck section as an indication of the resistance in the direction of the concrete slab span. If the support rotations in this direction exceed the limit criteria, the road deck has structurally failed as a slab between the girders.
- 2) The concrete that is involved in the compression zone of the composite cross section. How is deck deterioration impacting the deflection of the entire superstructure?

For the concrete deck, the absolute values of vertical displacements are time dependent. However, it appears that this is only caused by the girder (entire superstructure) oscillating up and down and not the slab. The vertical deflections of the deck in a transverse section of the bridge remain very constant as they develop instantly after the blast wave impact and do not elastically degenerate.

Figure 23: Girder Naming for Analysis

4.1.1.1 Blast Case 1

Figure 24: Vertical Deformation of Deck (Section 18.5m), Blast Case 1

Figure 25: Deformation Control Locations

The graph above clearly shows the permanent deflections in the transverse deck direction. But, according to the deflection curve, only the deck span located directly under the blast source point is affected by the blast while the adjacent spans do not deform significantly between their respective girder supports.

In order to quantify and rate the structural failure of the span, the displacements at the edge of girder +0 and at the transverse location +1.63 m (roughly the center of the span of the deck between girders 0 and +3) are compared in the following graph. It clearly shows, that for large parts of the girder, the girder and mid-slab-span deflections do not vary disproportionately. Only for a distinct area around the center of the girder span, the discrepancy is very large. Since these relative displacements are not expected to vary over time, time step 0.1 sec. after the explosion is chosen as the point of maximum girder deflection.

Figure 26: Vertical Deformation of Span for two Control Points at time step 0.1 sec., Blast Case 1

The absolute difference between the two control points specified above is presented in the chart below for the sector with significant differences. This chart will be used to determine the spread of the structural failure in the girder span direction.

Figure 27: Length of Deck Deformation

Figure 28: Difference of Vertical Deck Deformation for Case: Steel 0 Deck 0 Blast 1

As outlined above, a support rotation of 2° is considered as critical because (additional) shear reinforcement to allow for greater rotations is not present in the example bridge. Conservatively, this rotation corresponds to a vertical deck deflection of 0.049 m. at the center of the span.

Figure 29: Deck Failure Criteria Sketch

This means, that a patch with the length of roughly 5.75 m of the un-deteriorated bridge deck between the also initial girders 0 and +3 must be considered as destroyed by blast case 1.

This value of the will be the benchmark for the evaluation of the other scenarios. The characteristic distribution of vertical deflections in all other cases is similar to this example, only the size of the destroyed patch will need to be assessed for a rating of the deck response.

4.1.1.2 Blast Case 2

In this scenario, the explosive charge (same charge weight as in Blast Case 1) detonates in the travelling lane above the central support. The graph below shows, that again only the deck span below the source point is significantly impacted by the blast loading. Some differences to Blast Case 1 are identifiable. The total vertical deflection of the entire superstructure is obviously very small (the deflections are measured at the top of the steel/ bottom of the concrete face) because the steel is supported at the bottom of the steel profile at this exact section. The deck deflections however have increased in this section, both with regards to the magnitude and spread. The displacement under the blast source point has increased from 0.24 m to 0.32 m and small sagging of the concrete slab can be observed between the other girders (Located at -5, -3, 0, +3 and +5).

Figure 30: Vertical Deformation of Deck Section 36.8 m at Time Step 0.1 sec. for Blast Case 2

Since only the sag under the blast location exceeds the critical value of 0.05 m mid-concrete-slab-span deflection, only this sag will be considered in the following. The vertical deformations of the edge of the steel profile (girder 0, transverse location 0.18 m) and the slab-mid-span (transverse location 1.63 m) are presented in the chart below.

Figure 31: Difference of Vertical Deck Deformation for two Control Points at Time Step 0.1 sec.

Just as in Blast Case 1, the difference in deformations is a fairly local phenomenon around the blast location. In the chart below, the displacement delta is plotted for the relevant girder section.

Figure 32: Failure Criteria Case: Steel 0 Deck 0 Blast 2

The same method as in blast case1 will be used to assess the magnitude of the impact on the deck span. The center line of the bridge is located at + 36.779 m and acts as an axis of symmetry. Therefore, the length of the structural failure patch obtained from the chart above has to be doubled. In this setting, the destroyed patch has a total length of 8.8 m.

This suggests, that for the same size impact, the patch of structural failure is significantly bigger (Blast Case 1: 5.75 m ; Blast Case 2 : 8.80 m)

4.1.2 Steel Girder Superstructure

In contrast to the concrete deck, the composite girders show a very time dependent deflection curve. Even though the analysis duration is very short, the response has to be broken up into different characteristic periods.

4.1.2.1 Blast Case 1

At first, the deformation below the blast source point is very local as identifiable in the girder deflection chart at 0.005 sec. after the explosion:

Figure 33: Vertical Deformation of Girders at Time Step 0.005 sec.

At this very early time step, the biggest deformation can obviously be observed in the girders (Girder 0 & Girder + 3) located closest to the blast point. Girder -5 actually shows a very small deflection in the upwards.

Figure 34: Girder Location Naming [m]

As time goes on, the blast pressure expands spherically and so the deflection spread. After 0.025 seconds, all five girders show distinct deflections in the span subjected to the blast load while the other span is not yet noticeably displaced, at 0.075 seconds the second girder span shows significant displacements as well (upwards).

The deflection distribution is continuing to shift to a more smoothly spread distribution across the entire length of the spans through the time step of 0.1 seconds after the detonation. A point of contraflexure is still identifiable in the displacement chart but it has almost reached the abutment bearing at girder location 73.56 m.

Figure 35: Vertical Deformation of Girders at Time Step 0.1 sec.

It is now, at a time of circa one-tenth of a second after the explosion, that the maximum span deformation of the girder can be observed. The following chart presents the deflection-time-curve of each girder at one location (18.5m) which is roughly the center of the span subjected to blast. This first deflection after the detonation is expected to be the maximum deflection as the blast is the only analysis impact. All future deflection will therefore be smaller than the initial amplitude. This assumption will be verified in Section 4.1.2.3 (Long Analysis Period).

Figure 36: Vertical Deformation of Girders at Location 18.5 m

The biggest deflections can be observed in girders +3 and +5 after circa 0.1 seconds. Since a number of simplifying assumptions were made around girder +5 (the overhang deck thickness is not increased, the transverse reinforcement is not increased, etc.) that can be expected to increase the flexural resistance of the composite cross section at that location, the composite girder might not be modeled with sufficient accuracy. Therefor the value of 0.262 m at Girder +3 is established as the maximum girder deflection value in the following. The support rotation of Girder+3 is plotted in the chart below.

Figure 37: Absolute Rotation of Girder +3 at Abutment Support

This rotation is smaller than the 2° -rotation failure criterion. Therefore, the model suggests that girder has not structurally failed during this blast loading case.

At the time of 0.15 seconds after the detonation, the inflection point in the deformation distribution has disappeared and the girder is deforming continuously long the length of both spans.

Figure 38: Vertical Deformation of Girders at Time Step 0.15 sec.

Likewise, it can be observed, that the deflection of the first span is starting to decrease. At a time step of 0.2 seconds after the detonation, the second span actually shows greater absolute values of deflection than the impacted span.

Figure 39: Vertical Deformation of Girders at Time Step 0.2 sec.

The time-dependent deflection (as in graph above) at the center of the impacted span will be used as a reference during the comparison and assessment of the different deterioration scenarios. This deflection can be directly correlated to the critical support rotation check so that the failure of the superstructure can be assessed.

The maximum deflection of Blast Case 1 is 0.262 m at the center of the span of Girder +3.

4.1.2.2 Blast Case 2

The explosion above the central pier causes a completely different girder response behavior. After the explosion, the deck sections and thereby the girders around the

charge are pressed downwards locally at first. Both the resulting downwards and upwards deflections are fairly small. Another significant aspect is the apparent vertical deflection of the girder above the support. In the model, the support is assumed linear and pinned at the bottom of the steel so that this displacement means a shortening of the actual girder height of Girders 0 and +3.

Figure 40: Vertical Deformation of Girders at Time Step 0.05 sec. (Blast Case 2)

After 0.075, the pressure wave and the downwards deflection has spread across both spans of the bridge. The support displacements however remain constant.

Figure 41: Vertical Deformation of Girders at Time Step 0.075 sec. (Blast Case 2)

Up until this point of time in the analysis, the distance from the central support to the point of maximum deflection has continuously increased. At 0.1 seconds however, this trend has ended and the point of maximum deck deflection is moving back towards the bearing thus turning around the direction of the bridge deck movement.

Figure 42: Vertical Deformation of Girders at Time Step 0.1 sec. (Blast Case 2)

Now, the bridge deck girders are both moving upwards with the point of maximum positive deflection travelling from both ends of the bridge towards the middle.

Figure 43: Vertical Deformation of Girders at Time Step 0.125 sec. (Blast Case 2)

This tendency continues until at time step 0.2 seconds, the both bridge deck spans have deformations in the upwards direction.

Figure 44: Vertical Deformation of Girders at Time Step 0.2 sec. (Blast Case 2)

This behavior must be classified as very critical. During the process of the bridge deck's oscillation towards the positive deflection direction, the resultant force of the central bearing has probably switched to tension. The model assumes pinned connections disabling the possibility for upwards movement of the structure and

therefore possibly differing from the reality in an unacceptable manner. The actual bridge bearing might not restrict the structure from lifting out of position so that structural failure would have to be assumed. It cannot be guaranteed that the simulation results reflect the reality to a satisfying degree and the results of the simulation results from after 0.1 seconds will not be discussed in the following.

In conclusion, the deflections in this blast case are very small and will not cause girder support rotation close to the critical limit. However, a shortening of the girder section at the support location can be observed and must be studied more closely.

Figure 45: Vertical Deformation of Girder +3 at Location 36.8 m (Blast Case 2)

The maximum deflection is recorded at Girder +3 at 0.027 m which is equal to 1.9 % of the girder height (1.45 m) at this location. The yield point of steel is assumed at circa 0.2 % compression, so that the observed deformation is almost ten times as high. The failure criterion of 2 % shortening is almost reached.

Since the span deflections are very small, the girder shortening will be used as the critical value for the comparison of the deteriorated bridge cases. The support rotations will not be checked as the span displacements are very small.

4.1.2.3 Long Analysis Period

In order to check the assumptions made, a longer analysis period was run for the Blast 1 case. In the chart below, the vertical deformations of the center of the loaded (18 m) as well as the other span (55 m) are presented for 10 seconds after the blast.

Figure 46: Vertical Deformations of Girder +3 Case: Steel 0 Deck 0 Blast 1 - Long Analysis (10 sec)

Expectedly, the first deflection is clearly the biggest deflection, so making the support rotation checks for this deflection can be assumed representative.

Also, it appears that the oscillation of both spans after the detonation occurs around medians that are not zero. The span under the detonation (control point 18 m)

is oscillating around $U2 = - 0.04$ m while a trend of permanent upwards deflection (circa 0.05 m) can be observed in the other span. This would mean that the explosion has caused permanent deformation in the form of a sag in the loaded and a hunch in the other span.

4.1.3 Conclusion

So, even though the girders have not structurally failed in this blast design (Blast Case 1), the serviceability must be assumed severely impacted and the girders will have to be replaced. Heavy bridge rehabilitation work would be required anyway as parts of the concrete deck did structurally fail.

The same can be concluded for Blast Case 2. The girders experience permanent deflections and parts of the concrete deck have failed so that the bridge would have to be replaced. Here, the girders of the initial bridge are close to the failure criterion of compression shortening already and some of the deteriorated bridge cases exceed the criteria.

4.2 Deteriorated Bridges

In this section, the vertical deflections of the deteriorated bridges are presented. The results will be compared to the benchmark values found in the analysis of the initial bridge structure.

4.2.1 Deck Deterioration

The deck deterioration scenarios consist of 1, 2, 4 or 6 patches of deck section loss. In the ‘moderate’ deterioration cases, one inch of the top concrete cover is deteriorated. The ‘severe’ setting, the structural deck height is reduced by 2.5 inches.

4.2.1.1 Moderate Deck Deterioration (Blast Case 1)

The characteristic deformation curve is not changing significantly. However, two things can be observed. The sagging of the deck span between girders 0 and +3 under the blast source increases considerably, but the difference between the different ‘moderate deck deterioration’ scenarios is marginal. Secondly, the vertical deformation of the entire bridge increases by a small amount, these girder deflections will be discussed in section 4.2.1.3.

Figure 47: Vertical Deformations of the Deck, Cases Steel 0 Deck 11, 12, 14 & 16

The increase of sag between the two girders is on the one hand very significant, on the other hand only of minor importance as the sag of the initial structure already caused failure in the deck span. However, the spread of deck failure is expected to increase as well.

Figure 48: Deformation Differences for Cases Steel 0 Deck 11, 12, 14 & 16

The chart above presents the difference between the vertical deformation of the flange edge of the girder (transverse location 0.18 m) and the center of the deck span (transverse location 1.6 m). Again, the differences between the different ‘moderate deterioration’ scenarios are immeasurably small. For all cases of ‘moderate’ deck deterioration, the length of the deck span failure patch increases by circa 60 centimeters from 5.75 m (initial deck) to 6.35 m.

4.2.1.2 Severe Deck Deterioration (Blast Case 1)

The deteriorated patches of the road deck influence the deck response in a similar manner. As visible in the chart below, the transverse distribution of the deflections is still similar to both the initial bridge. The magnitude of deflection is further increased, both the center of the deck span and girder.

Figure 49: Vertical Deck Deformations for Cases Steel 0 Deck 21, 22, 24 & 26

Also the length of the patch of structural failure increases by circa 1.1 meter from 5.75 m (initial bridge) to 6.85 m.

Figure 50: Deformation Differences for Cases Steel 0 Deck 21, 22, 24 & 26

4.2.1.3 Girder Deformation (Blast Case 1)

As outlined above, the concrete deck is participating in transferring loads to the bridge bearing supports of the steel girders. A reduction of surface area of the concrete slab of this composite structural member reduces the flexural stiffness. Thus, deflections will increase. These effects are most prominent in the span as the concrete participation in the positive moment region of the continuous girder, the rigidity of the negative moment region is less affected. Here, the concrete contribution is limited as the top of the section is subjected to tension and the concrete is commonly neglected in the capacity analysis.

Figure 51: Vertical Deformations of Girder +3 at Location 18.5m for Cases Steel 0 Deck 11, 16, 21 & 26

In the chart above, the concrete Deck Cases 11 and 21 have one patch of ‘moderate’ and ‘severe’ deterioration respectively above in the center of the span subjected to blast. The deflection control point plotted in the chart is located in the region of deck deterioration above the girder. Deck Cases 16 and 26 (again, ‘moderate’ and ‘severe’ deterioration respectively) have six patches of section loss. Three of which are located above girder +3 at both centers of the spans and the central support. The other three patches are located above girder -3.

With increasing section loss, the deflections increase, but the dimensions of the increase are small. For bridge decks that with minor to moderate deterioration of the deck (Cases 11 & 16), the deflection increase is of 2 % or less. For heavily deteriorated bridges (Deck Case 26), the deflection scaled up by only 6.7 % considering the severity of this deterioration scenario.

4.2.1.4 Blast Case 2

The deformation curve for the concrete deck slab at the location under the blast (Case Blast 2) changes slightly. As discussed, while the girder deflections at mid-span are very small, the deck deflections increase. Just as in the Blast 1 case, the deteriorated deck shows more deflection. In this case however, the effect of the second patch of deterioration seems at this section seems to be bigger. For the case Deck 14, the span between girders 0 and -3 also shows sagging. However, the deflection is less than 0.05 m so that the rotations at the support are expected to be less than 2 ° and the spans do not fail structurally.

Figure 52: Vertical Deck Deformation at Section 36.8 m for Cases Steel 0 Deck 11, 12 & 14 (Blast Case 2)

The patch of structural failure increases from 8.8 m in the case of no deterioration above the support to 9.3 m for ‘moderate’ deterioration and 9.8 m in the ‘severe’ deterioration case. It appears that only the deterioration at the control point impacts the spread of structural failure. The difference between the cases with additional deteriorations in other patches across the deck surface is very small.

Figure 53: Deformation Differences for Cases Steel 0 Deck 11, 12, 14, 21, 22 & 24 (Blast Case 2)

The steel deformations (profile height shortening) appear to be not effected by the concrete deterioration in Blast Case 2.

4.2.1.5 Combined deterioration

The deterioration of the steel girder appears to not have a noticeable influence on the transverse deformation characteristics of the concrete slab. The only difference in the chart below is the absolute value of bridge deflection as outlined in the comparisons of different models for section 18.5 below. Since the maximum deflection does not increase, the length of the deteriorated patch is expected to not increase either.

Figure 54: Vertical Deformation of Deck for Combined Deterioration (Blast Case 1)

4.2.2 Steel Deterioration

In this section, the results of the different steel deterioration scenarios are assessed and the disparities to the initial bridge structure are identified. For the girder response evaluation, the plots of displacement over time of the node at the center of the loaded span and the two-span girder displacement curve at the time point of maximum deflection are utilized.

4.2.1.1 Single Element Deterioration (Blast 1)

Steel Cases 1 to 4 consist of section loss of the initial girder in 4 distinct locations. Hereby, the contribution of each girder sector towards the overall deformation characteristic can be assessed.

Figure 55: Vertical Deformation of Girder +3 for Cases Steel 0, 1, 2, 3, 4, 5 & 10 Deck 0

Since the differences are very small, the range of maximum values of the deflection curve is to be looked at a bigger scale. It appears that the deterioration of the webs and bearing stiffeners around the supports (Case Steel 2) has little influence on the center deflection.

The deterioration of the flanges around the support (Case Steel 1) and the section loss of the web across the span length (Case Steel 4) seem to result in a similar loss of flexural rigidity of the overall girder structure.

The biggest impact on the overall deflection however has the section loss in the span flanges (Case Steel 3). This was to be expected as the flanges not only have the major role in stiffness of a girder in the span, this scenario also yielded the greatest amount of steel being assumed deteriorated.

Figure 56: Maximum Vertical Deformation of Girder +3 for Cases Steel 0, 1, 2, 3, 4, 5 & 10 Deck 0

Additionally, a combination of all 4 deterioration assumptions into a single model is simulated in the Case Steel 10. Obviously, the deflection is greater than in the cases

of restricted deterioration, but the margin of difference is smaller than e.g. a simple addition of the deflection increases of the cases 1 to 4.

A second tendency can be observed from the chart above. It appears that for the slightly softer structure as indicated by greater deflection, the maximum deformation is measured at a slightly later time step.

Figure 57: Vertical Deformation of Girder +3 at Time Step 0.092 sec for Cases Steel 0, 1, 2, 3, 4, 5 & 10

The global girder deflections curves are very similar. However, some aspects are noticeable. Despite the differences of loaded span deflection, the maximum displacement in the other span is very similar. Only Steel Case 1 has a lower amplitude value than all the other cases. It appears as if the reduction of flange volume

around the support results in such a stiffness reduction, that the continuous girder is developing a slightly more flexible characteristic behavior around the support than in the other cases.

4.2.2.2 Overall Steel Girder Deterioration (Blast 1)

Steel cases 5, 10 and 20 are set up to represent different extents of overall bridge deterioration. As it must be expected, the greater section loss results in greater deflections and the maximum being observed at a later time.

The difference between the initial bridge (Case Steel 0) and Case Steel 20 is a little over 10 % of deflection increase. Again, the softer structure shows the maximum deflection at a later time step.

Figure 58: Vertical Deformation of Girder +3 at Location 18.5m for Cases Steel 0, 5, 10 & 20 Deck 0

4.2.2.3 Blast 2

This scenario shows a very different deformation behavior. During the analysis time, the span deflections and therefore the support rotations are very small (compared to Blast Case 1). The steel deterioration does not seem to increase the maximum deck deflection at center span noticeably. The absolute value is still very small so that support rotations will not become critical.

Figure 59: Vertical Deformations of Girder +3 at Location 18.5m (Blast Case 2)

The permanent deformations of Girder +3 increases with increasing cross section reduction. The biggest increase can obviously be observed in Steel Case 2 (Web around the support and Bearing Stiffener Deterioration), the absolute growth is 1.2 mm.

Figure 60: Vertical Deformation of Girder +3 at Location 36.8m for Cases Steel 0, 1, 2, 3 & 4 Deck 0 (Blast Case 2)

In the combined deterioration scenario, the deformation increase grows to 30 mm (2.1 % shortening of the initial girder) in Steel Case 20. This suggests that for the severely deteriorated bridge, the failure criterion of 2 % (29 mm) shortening is exceeded and the steel profile (idealized column of web and bearing stiffeners) has ‘light damage’.

Figure 61: Vertical Deformation of Girder +3 at Location 36.8 m for Cases Steel 0, 5 & 20 Deck 0 (Blast Case 2)

4.3 Validation of the Model

In this section, accuracy checks of the ABAQUS model are presented. The actual calculations of the section properties can be found in the appendix.

4.3.1 System Deformation

Since the Blast analysis is dynamic and time dependent, the static analysis of the model subjected only to its own dead load and the lane load is checked.

The static analysis with ABAQUS gives a maximum deflection of 0.0386 m.

Figure 62: ABAQUS Result of Static Analysis

This deflection value is checked with simplified manual calculation equations presented in the Appendix. The manual calculations suggest a maximum deflection of 0.0376 m.

The difference between these two deflection values obtained with two totally different methods is very small so that the flexural stiffness of the structure in the ABAQUS model is assumed to be accurate.

4.3.2 Blast Pressure

The blast pressure wave for the analysis is automatically determined by the CONWEP model in the ABAQUS software. However, the software does provide an output for the time-pressure histories assumed at each of the surfaces. In this section, the pressure output values are compared to blast design diagrams by Department of Defense (2014).

At the location directly underneath the blast source (Control Point), the first impact and biggest load can be observed after just 0.0005 seconds.

Figure 63: Blast Loading from CONWEP model for Point located under Blast Source

At a time step of 0.0008 seconds, a circular blast wave impact surface can be identified.

Figure 64: CONWEP model loading at 0.0008 seconds

As the time progresses, the circle of impact radiate outwards transforming into a ring-shaped impact. In the center of the ring, a short moment of negative pressure can be observed. In the following two pictures, the CONWEP loading is presented at the

time steps 0.0013 seconds and 0.0035 seconds after the analysis begin/ the time of detonation.

Figure 65: CONWEP model loading at time steps 0.0013 sec. and 0.0035 sec.

At a second control point located further away from the blast source point (location above Girder -3), the blast wave impact has changed considerably. The pressure is reduced significantly with is not only due to the increase of distance to the blast source point, the surface is also not located directly below the explosion resulting in the blast wave impacting the surface at an angle.

To check this loading input, the values are compared to the impacts obtainable from the figures from *UFC 3-340-02* (Department of Defense, 2014).

The scaled standoff height for Control Point:

$$Scaled\ Distance\ Z = \frac{6.7\ ft}{(600\ lbs)^{1/3}} = 0.79$$

From Figure 2-7 (*UFC 3-340-02*), an Incident Pressure $P_{so} = 1.1\ ksi (=7579\ MPa)$ and a Reflected Pressure $P_r = 10.0\ ksi (=68900\ MPa)$ can be determined for the Control Point. The ABAQUS Analysis User's Guide (2014) states that for an angle of incidence equal to zero, the total pressure is the sum of incident and reflected pressure. At the control point this results in:

$$P_{total} = P_{incident} + P_{reflect} = 7579 + 68900 = 76479\ MPa$$

This value is slightly bigger than the CONWEP model value from the ABAQUS output. The difference could originate from the fact, that while the value obtained from the *UFC 3-340-02*-figure specifies a single point located exactly under the blast, the CONWEP output was obtained for a (small) surface area. The difference between the two values (CONWEP: 70.5, UFC: 76.5) is roughly 8 %.

CHAPTER 5: CONCLUSION

In this final chapter, the results and findings as presented in Chapter 4 are summarized and interpreted.

5.1 Effects of Section Reduction on the Structural Response

The output data suggests that for the small differences in the steel elements, the stiffness of the overall cross section decreases as indicated by the increase of displacement. Of course, this must be expected. However, the margin of increase is rather small. So even though significant amounts of the structure were assumed to have deteriorated, the maximum deflection only increased by single digit percentages.

Likewise, the concrete deck section loss lead to more extensive structural failure patches and greater girder deflections, but the differences are small as well.

This suggests that for bridges with section loss due to deterioration, the blast resistance does not suddenly drop by a great amount or the bridge displays significantly larger or prominently different deflections. After the explosion, both the initial and the deteriorated bridge have significant permanent deformations of the steel structure and part failure of the concrete so that the superstructure is not in a condition to continue to serve as a roadway and would have to be replaced either way.

In the blast case above the central pier support (Blast Case 2), the initial bridge structure shows deformation very close to the 'light damage' critical value. Severe deterioration increases the deformation enough so that the critical value is exceeded and light damage must be assumed.

5.2 Model accuracy

The model cannot be assumed as sufficiently validated as it has not been calibrated with historical experimental data or compared to a different, established method to assess blast loading on bridges.

5.2.1 Simulation

This means, that the absolute values of the simulation output cannot be expected to be exact with sufficient confidence. The actual bridge deflection in the cases studied is not known and no margin of possible error can be established. However, the focus of this report was the examination of a change in bridge behavior with section reductions. It is expected that even though the absolute values may be inaccurate by a certain error, the comparison between the cases remains sufficiently valid. All models are expected to have a similar error since the differences between the models are very small. So in conclusion, the absolute size of concrete deck failure and steel girder rotations has an unknown inherent error and must not be utilized for design purposes, but the differences between the initial bridge and the section loss scenarios are very small. Therefore, the conclusions made in the following are expected to be valid for other model calibrations and varying detonation scenarios.

5.2.2 Explosive Charge

Another source of inaccuracy is the assumed nature of the explosive charge. For the study, a free-air burst was studied. However, for both accidental and intentional explosions on bridges, the detonation will be somewhat confined by the vehicle or

other casings of the explosive material. This may lead to different impacts to the bridge than the one considered in this study. Also, the standoff distance of the explosive charge must be characterized as large for the scenario examined. It is possible that for a vehicular attack, the blast source is located closer to the actual road surface than 6 ft. 7 inches. However, in a more realistic case, the road surface might not be impacted directly as the vehicle framework is in between the deck surface and the burst point.

5.2.3 Deterioration Assumptions

In the model, the only deterioration characteristic implemented is the loss of section. In actual bridges, a variety of aging effects must be expected ranging from changes of material characteristics to cracks and fatigue. Those effects have not been considered in this study.

Pre-damaged and already cracked structures may fail locally as structural resistance is already impaired. If this local damage is not included in the analysis, the bridge response can probably not be predicted accurately. Likewise, embrittlement might change the characteristic behavior as large deformations do not develop. Instead, a more local and sudden failure could be observed.

5.3 Conclusion

In conclusion, bridges that have experienced section loss as assumed in the different scenarios do not behave characteristically differently from the initial structure. But even though the output data suggests, that bridges that have experienced

section loss do not perform disproportionately worse, deterioration cannot be neglected in an analysis of bridge blast resistance. It must be expected, that other aspects of ageing such as embrittlement of steel, corrosion of reinforcement as well as cracks and other fatigue induced mechanisms have great impact and reduce the resistance more significantly.

So, while section loss does lead to the anticipated increase of deflections, the reduction of resistance is small. But section loss should not be assumed to be the only effect of deterioration and further research is strongly encouraged.

5.4 Further Research

In order to gain a better understanding of the bridge response to blast loading and the effects of deterioration, further research should be conducted. This could include:

- Further development of the material parameters (material damage, embrittlement, etc.)
- Further improvement of the simulation model (more accurate representation of the supports, consideration of bolted connections, inclusion of concrete-girder shear connectors, inclusion of the haunch, inclusion of the parapet structure, more accurate model of the concrete deck, etc.)
- Further investigation of the more accurate blast source modeling (e.g. consideration of the confinement in accidental truck explosion, etc.)
- Validation of the model using proven methods or actual testing so that the model output can be verified

APPENDICES

A.1 Static Deformation of the Example Bridge

84 ft. Section (Positive Moment)

Section Properties [mm]

Element	b or h	t
Concrete Slab		
Slab	3000	216
Reinforcement specified in Section 3.2		
Plate Girder		
Top Flange	355.6	15.9
Web	1371.6	12.7
Bottom Flange	355.6	22.2
Others		
Haunch	0	
Modular Ratio	n = 6.06	

Composite Girder EI, distances established from bottom of steel profile

Element i	y_i	A_i	$y_i A_i$	$y_i^2 A_i$	I_{Eigen}
Concrete Slab					
Slab	1517.7	1.07E+05	1.62E+08	2.46E+11	4.16E+08
Top Rebar	1539.9	5.00E+02	7.70E+05	1.19E+09	0.00E+00
Bot Rebar	1459	7.80E+02	1.14E+06	1.66E+09	0.00E+00
Steel Girder					
Top Flange	1401.75	5.65E+03	7.93E+06	1.11E+10	1.19E+05
Web	708	1.74E+04	1.23E+07	8.73E+09	2.73E+09
Bot Flage	11.1	7.89E+03	8.76E+04	9.73E+05	3.24E+05
TOTAL		1.39E+05	1.85E+08	2.69E+11	3.15E+09

Height of Neutral Section (from bottom of Steel): 1325.9 mm

Area of Steel 3.1 E+4 mm²

Resultant I = 2.74 E+10 mm⁴

24 ft. Section (Positive Moment)

Section Properties [mm]

Element	b or h	t
Concrete Slab		
Slab	3000	216
Reinforcement specified in Section 3.2		
Plate Girder		
Top Flange	355.6	31.8
Web	1355.7	12.7
Bottom Flange	355.6	34.9
Others		
Haunch	0	
Modular Ratio	n = 6.06	

Composite Girder EI, distances established from bottom of steel profile

Element i	y_i	A_i	$y_i A_i$	$y_i^2 A_i$	I_{Eigen}
Concrete Slab					
Slab	1530.4	1.07E+05	1.64E+08	2.50E+11	4.16E+08
Top Rebar	1552.6	5.00E+02	7.76E+05	1.21E+09	0.00E+00
Bot Rebar	1471.7	7.80E+02	1.15E+06	1.69E+09	0.00E+00
Steel Girder					
Top Flange	1406.5	1.13E+04	1.59E+07	2.24E+10	9.53E+05
Web	712.75	1.72E+04	1.23E+07	8.75E+09	2.64E+09
Bot Flage	17.45	1.24E+04	2.17E+05	3.78E+06	1.26E+06
TOTAL		1.49E+05	1.94E+08	2.84E+11	3.05E+09

Height of Neutral Section (from bottom of Steel): 1300.5 mm

Area of Steel 4.09 E+4 mm²

Resultant I = 3.53 E+10 mm⁴

12 ft. Section (Negative Moment)

Section Properties [mm]

Element	b or h	t
Concrete Slab		
Slab	3000	216
Reinforcement specified in Section 3.2		
Plate Girder		
Top Flange	355.6	63.5
Web	1324.0	12.7
Bottom Flange	355.6	69.9
Others		
Haunch	0	
Modular Ratio	n = 6.06	

Composite Girder EI, distances established from bottom of steel profile

Element i	y_i	A_i	$y_i A_i$	$y_i^2 A_i$	I_{Eigen}
Concrete Slab					
Slab	1565.4	0.00E+00	0.00E+00	0.00E+00	0.00E+00
Top Rebar	1586	1.56E+03	2.47E+06	3.92E+09	0.00E+00
Bot Rebar	1506.7	1.56E+03	2.35E+06	3.54E+09	0.00E+00
Steel Girder					
Top Flange	1425.65	2.26E+04	3.22E+07	4.59E+10	7.59E+06
Web	731.9	1.68E+04	1.23E+07	9.01E+09	2.46E+09
Bot Flage	34.95	2.49E+04	8.69E+05	3.04E+07	1.01E+07
TOTAL		6.74E+04	5.02E+07	6.24E+10	2.47E+09

Height of Neutral Section (from bottom of Steel): 744.98 mm

Area of Steel 6.43 E+4 mm²

Resultant I = 2.75 E+10 mm⁴

Loading for one girder and its 3 m deck width

Averaged Steel Area:

$$\frac{3.1 \cdot 10^4 \cdot 84 + 4.09 \cdot 10^4 \cdot 24 + 6.43 \cdot 10^4 \cdot 12}{120} = 3.63 \cdot 10^4 \text{ mm}^2$$

Element	Comments	Loading (kN/m)
Steel	$\frac{3.63 \cdot 10^4}{10^6} \cdot 78500 \frac{\text{kN}}{\text{m}^3} =$	2848.7
Miscellaneous Steel	10 % of 'Steel Loading' added	284.9
Concrete	$\frac{3000 \cdot 216}{10^6} \cdot 24000 \frac{\text{kN}}{\text{m}^3} =$	15552
Lane Load	$3 \cdot 1532.2 =$	4596.6
TOTAL:		23282.2

Elastic Modulus Steel: $E = 2.0 \text{ E}+11$

Modular Ratio (Steel/Concrete): $n = 6.06$

Average of Flexural Stiffness:

$$I_{averaged} = \frac{I_{84-ft} + I_{24-ft} + I_{12-ft}}{3} = 3.01 \cdot 10^{10}$$

Maximal Deflection for single span, one side pinned, other side fixed

$$w_{max} = \frac{q \cdot l^4}{184.6 \cdot EI} = \frac{23282.2 \cdot 36576^4}{184.6 \cdot 2.0 \cdot 10^{11} \cdot 3.01 \cdot 10^{10}} = 0.03755 \text{ m}$$

A.2 Deck Reinforcement System Input

Table 27: Deck Reinforcement, Negative Moment, No Deterioration

Layer	Area [m ²]	Spacing [m]	Position [m]
Top Long	0.000198	0.127	0.0206
Top Tran	0.000198	0.1524	0.0365
Bot Long	0.000198	0.127	-0.0587
Bot Tran	0.000198	0.2032	-0.0746

Table 28: Deck Reinforcement, Positive Moment, No Deterioration

Layer	Area [m ²]	Spacing [m]	Position [m]
Top Long	0.000127	0.254	0.0222
Top Tran	0.000198	0.1524	0.0365
Bot Long	0.000198	0.254	-0.0587
Bot Tran	0.000198	0.2032	-0.0746

Table 29: Deck Reinforcement, Negative Moment, Mild Deterioration

Layer	Area [m ²]	Spacing [m]	Position [m]
Top Long	0.000197933	0.127	0.0333
Top Tran	0.000197933	0.1524	0.0492
Bot Long	0.000197933	0.127	-0.0460
Bot Tran	0.000197933	0.2032	-0.0619

Table 30: Deck Reinforcement, Positive Moment, Mild Deterioration

Layer	Area [m ²]	Spacing [m]	Position [m]
Top Long	0.000127	0.254	0.0349
Top Tran	0.000198	0.1524	0.0492
Bot Long	0.000198	0.254	-0.0460
Bot Tran	0.000198	0.2032	-0.0619

Table 31: Deck Reinforcement, Negative Moment, Severe Deterioration

Layer	Area [m ²]	Spacing [m]	Position [m]
Top Long	0.000127	0.254	0.0524
Top Tran	0.000198	0.1524	0.0683
Bot Long	0.000198	0.254	-0.0270
Bot Tran	0.000198	0.2032	-0.0429

Table 32: Deck Reinforcement, Positive Moment, Severe Deterioration

Layer	Area [m ²]	Spacing [m]	Position [m]
Top Long	0.000127	0.254	0.0540
Top Tran	0.000198	0.1524	0.0683
Bot Long	0.000198	0.254	-0.0270
Bot Tran	0.000198	0.2032	-0.0429

BIBLIOGRAPHY

- Agrawal, A.K. and Yi, Z., 2008, “High Precision Analysis of Blast Events on Highway Bridges”, Department of Civil Engineering, The City College of New York
- American Association of State Highway and Transportation Officials, 2010, “AASHTO Bridge Element Inspection Guide Manual”, 1st Edition
- American Association of State Highway and Transportation Officials, 2012, “AASHTO LRFD Bridge, Design Specifications, Customary U.S. Units”, Sixth Edition, AASHTO, Washington, D.C.
- American Association of State Highway and Transportation Officials, 2008, “The Manual for Bridge Evaluation”, 1st Edition, AASHTO, Washington, D.C.
- American Concrete Institute, 1995, “Building Code Requirements for Structural Concrete (ACI 318-95) and Commentary (ACI 318R-95)”, ACI, Farmington Hills, MI
- Biggs, J.M., 1964, “Introduction to Structural Dynamics”, McGraw-Hill Book Company, New York, San Francisco, Toronto, London
- Conrath, E.J., Krauthammer, T., Marchand, K.A. and Mlakar, P., 1999, “Structural Design for Physical Security, State of the Practice”, American Society of Civil Engineers
- Dassault Systemes, 2014, “Abaqus Analysis User’s Guide”, Dassault Systèmes Simulia Corp., Providence, RI

- Department of Defense, 2014, “Unified Facilities Criteria (UFC), Structures to Resist the Effects of Accidental Explosions”, Publication: UFC 3-340-02
- Gebbeken, N. and Döge, T., 2006, “Vom Explosionsszenario zur Bemessungslast”, *Der Prüfenieur*, Oct. 2006, pp: 42 – 52
- Geringer, J.R., Tuan, C.Y. and Lindsey, P.D., 2013, “Assessment of Software for Blast Loading and Structural Response Analysis Using a Lightweight Steel-Joist Roof as a Test Case”, *Journal of Performance of Constructed Facilities*, March/April 2013, pp: 144-154
- Gündel, M., Hoffmeister, B. and Bangert, F., 2010, “Bemessung von Baustrukturen in Stahl- und Verbundbauweise für Anprall- und Explosionslasten”, *bauforumstahl e.V.*, Düsseldorf (Germany)
- Gündel, M., Hauke, B., Hoffmeister, B. and Feldmann, M., 2010, “Bemessung von Stahlbauten für Anprall- Explosionslasten”, *Stahlbau 79, Heft 8*, pp: 590-597
- Krauthammer, T., “AISC Research on Structural Steel to Resist Blast and Progressive Collapse”, Feb. 11, 2016, www.researchgate.net
- Michael Baker Jr., Inc., 2003, “LRFD Design Example for Steel Girder Superstructure Bridge”, Publication No. FHWA NHI-04-041, for FHWA/ National Highway Institute, Washington, D.C.
- National Cooperative Highway Research Program (NCHRP), 2010, “Blast-Resistant Highway Bridges: Design and Detailing Guidelines”, Report No. 645, Transportation Research Board, Washington, D.C.
- Randers-Pehrson, G. and Bannister, K. A., 1997, “Airblast Loading Model for DYNA2D and DYNA3D”, Army Research Laboratory, ARL-TR-1310

- Science Applications International Corporation (SAIC), 2002, “A Guide to Highway Vulnerability Assessment for Critical Asset Identification and Protection”, for the American Association of State Highway and Transportation Officials’ Security Task Force, Vienna, VA
- Son, J. and Astaneh-Asl, A., 2012, “Blast Resistance of Steel Orthotropic Bridge Decks”, *Journal of Bridge Engineering*, July/ August 2012, pp: 589-598
- Starossek, U., 2005, “Progressiver Kollaps von Bauwerken” in *Beton- und Stahlbetonbau 100*, Heft 4, pp. 3-15
- The Blue Ribbon Panel on Bridge and Tunnel Security, 2003, “Recommendations for Bridge and Tunnel Security”, requested by the American Association of State Highway and Transportation Officials (AASHTO) Transportation Security Task Force
- U.S. Department of Transportation, Federal Highway Administration, 2012, “Bridge Inspector’s Reference Manual”, Publication No. FHWA NHI 12-049, Arlington, VA
- U.S. Department of Transportation, Federal Highway Administration, 2005, “Recording and Coding Guide for the Structure Inventory and Appraisal of the Nation’s Bridges”, Report No. FHWA-PD-96-001, Washington, D.C.
- Whittaker, A., Hamburger, R. and Mahoney, M., 2003, “Performance-Based Engineering of Buildings for Extreme Events”, Conference Paper, AISC-SINY Symposium on Resisting Blast and Progressive Collapse, At New York, NY

- Wight, J.K. and MacGregor, J.G., 2009, ‘Reinforced Concrete: Mechanics and Design’, fifth Edition, Pearson Education Inc., Upper Saddle River, New Jersey
- Williamson, E.B. and Winget, D.G., 2005, “Risk Management and Design of Critical Bridges for Terrorist Attacks”, *Journal of Bridge Engineering*, January/February 2005, pp: 96-106
- Winget, D.G., Marchand, K.A. and Williamson, E.B., 2005, “Analysis and Design of Critical Bridges Subjected to Blast Loads”, *Journal of Structural Engineering*, pp: 1243-1255
- Yi, Z., Agrawal, A.K., Ettouney, M. and Alampalli, S., 2014, “Blast Load Effects on Highway Bridges. I: Modeling and Blast Load Effects” in *Journal of Bridge Engineering*, pp: 040 13023 – 1 to 040 13023 – 17
- Yi, Z., Agrawal, A.K., Ettouney, M. and Alampalli, S., 2014, “Blast Load Effects on Highway Bridges. II: Failure Modes and Multihazard Correlations”, *Journal of Bridge Engineering*, pp: 040 13024 – 1 to : 040 13024 – 12