APPLICATION OF MULTI-HAZARD SEISMIC-BLAST DETAILING FOR HIGHWAY BRIDGES

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APPLICATION OF MULTI-HAZARD SEISMIC-BLAST DETAILING FOR HIGHWAY BRIDGES

A THESIS IN
Civil Engineering

Submitted in partial fulfillment of
The requirements for the degree of
MASTERS OF CIVIL ENGINEERING
at
The City College of New York of the
City University of New York

by

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_______________________  ________________________
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ABSTRACT OF THE THESIS

APPLICATION OF MULTI-HAZARD SEISMIC-BLAST DETAILING FOR HIGHWAY BRIDGES

The increase of worldwide terrorist attacks on public transportation has heightened our concerns of protecting the nation’s transportation infrastructure. Highway bridges are an attractive target for terrorist attacks due to ease of accessibility and their overall importance to society.

The primary objective of this research is to investigate multi-hazard seismic-blast correlations of blast-induced bridge components through numerical simulations of a high-precision finite element model of a typical highway bridge in New York.

Seismic-detailing for blast loading on bridges has been investigated to study the correlations between seismic design for blast load effects. High-precision 3D Finite Element models of bridges detailed for blast-resistant applications have been developed by designing the bridges for various seismic zones. In total, 9 cases of simulations for blast-induced bridges have been simulated. From the simulations, four failure mechanisms were observed and have been identified.

Results from the simulation suggest that bridges detailed with higher seismic capacities were able to resist more blasted-induced failure mechanisms. The amount and location of transverse reinforcement in bridge columns played a significant role for better blast resistance. Although, there are several failure mechanisms that arise from blast loadings that do not take place in seismic conditions.
DEDICATION

Mom and Dad
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CHAPTER 1. INTRODUCTION

1.1 BACKGROUND

It has been made evident that the increase of domestic and international terrorist attacks to the nation’s civil structures over several decades has led to tremendous losses. The Oklahoma City bombing on April 19, 1995 was a domestic terrorist bomb attack on the Alfred P. Murrah Federal Building which led to total destruction of the building. International terrorists shocked the world after the simultaneous attacks on The Pentagon and the World Trade Center Towers which led to the collapse of the towers on September 11, 2001. As a result of these events, there has been an increase in awareness and concern of threats against our nation’s bridges, tunnels, and other highway infrastructures. Challenges relating to the physical security of infrastructure protection against terrorist attacks are fairly new and for the most part unexpected. It is deemed necessary to establish design standards to enhance blast resistance of our transportation infrastructure for the prevention of catastrophic failure.

The nation’s transportation infrastructure is considered an attractive target for terrorist attacks due to the ease of accessibility and potential impact on human lives, economic activities, and socio-political damages. In June 2003, a truck driver from Ohio who admitted he was an al-Qaeda agent was convicted of plotting to sabotage a New York bridge by severing the cables of the bridge with specialized equipment. More than 50% of terrorist attacks worldwide are in the form of explosives and approximately 60% of these attacks against highway infrastructure have consisted primarily of explosive attacks [FHWA (2006); Jenkins and Gerston (2001)].

Major efforts have been established specifically on transportation security since the September 11th attacks. A Blue Ribbon Panel (BRP) of bridge and tunnel experts from professional practice, academia, federal and state agencies and toll authorities convened to examine bridge and tunnel security and to develop strategies and practices for deterring, disrupting, and mitigating potential attacks. The panel was organized through a joint effort of the Federal Highway Administration (FHWA) and the
American Association of State Highway and Transportation Officials (AASHTO) and has been among the most significant efforts in the development of recommendations and formulation of short-term and long-term strategies for dealing with terrorist threats to bridges and other transportation assets.

The Blue Ribbon Panel prioritizes all bridges with respect to their vulnerability in terms of their criticality of the ability to deter, deny, detect, delay, and defend against terrorist attacks. The BRP provides recommendations for design criteria based on various mitigating strategies. The following are examples of approaches to mitigating consequences:

- **Create Standoff Distance.** The first level of mitigating terrorist attacks should be to incorporate sufficient standoff distances from primary structural components.

- **Add Design Redundancy.** Structural systems that provide great redundancy among structural components will help limit collapse in the event of severe structural damage from unpredictable terrorist acts.

- **Hardening/Strengthening the Elements of the Structure.** Structural retrofitting and hardening priority should be assigned to critical elements that are essential to mitigating the extent of collapse [BRP (2003)].

Although improvements have been made in recent years, there still lacks available information regarding terrorist preparedness accessible to structural engineers. Consequently, current design guidelines do not adequately consider the issue of bridge security due to lack of awareness. As a result, professional organizations such as AASHTO, Transportation Research Board (TRB), and the American Society of Civil Engineers (ASCE) have established national committees to address the topic of transportation security [NCHRP 645 (2010)].

### 1.2 RESEARCH OBJECTIVES

This investigation analyzes blast load effects on bridge components through developing high-fidelity finite element model of a typical highway bridge in the United States, and identifying typical mechanisms responsible for causing damage/failure of bridge components. It will shed light on multi-
hazard design of bridge components employing available detailing guidelines for earthquake-resistant design of bridges.

1.3 RESEARCH APPROACHES

Typically, design guidelines for structures subjected to hazards require experimental verifications using scaled models. A blast load is considered a unique event and is very difficult to reproduce the same blast wave environment. As a result, experimental studies of blast loads on scaled models may be difficult to conduct due to various parameters affecting structural behavior of components. In addition, experimental blast tests are very expensive and can only be conducted in selected facilities.

Analytical tools are desirable in predicting load and material response when experimental testing cannot be conducted. LS-DYNA is a finite element modeling software which employs computational fluid dynamics and has been used in several investigations for predicting blast loads and material response. This investigation employs LS-DYNA for the simulation of blast load effects on highway bridges. LS-DYNA has the capability of directly applying blast loads on the structure either by simulating the detonation process of high explosive through fluid (blast wave) and structure interaction or by applying the blast pressure load determined from semi-empirical equations directly on structural components. In this investigation, blast loads determined from semi-empirical equations are applied to structural components. The following issues have been considered during the simulation:

- Finite element type for structure members/components
- Influence of time step size during the simulation
- Influence of finite element mesh size
- Application of gravity and blast load
- Simulation numerical stability and reliability issues

The main objective of this research has been to investigate blast load effects on a typical highway bridge in the United States. A three-span reinforced concrete bridge located on a major highway has been chosen from a review of national bridge inventory.
The effects of blast loads on bridges and failure mechanisms of bridge members, which may lead to global collapse of the structure, have been identified through numerical simulation of the finite element model of the bridge. Utilizing AASHTO design guidelines for bridges, the correlations between seismic detailing and blast load effects have been investigated.

1.4 OUTLINE OF THESIS

This thesis is based on knowledge obtained from several experimental and analytical studies of blast load effects. The outline of this thesis is as follows:

The present chapter, Chapter 1, has established the problem statement and objectives of this research.

Chapter 2 provides an in-depth literature review of blast load effects. An overview of the principles of shock propagation originating from the detonation of high explosives, current design guidelines and state-of-practice on blast analysis, and basic analytical procedures for predicting the response of structures to dynamic loads are provided from the literature review.

Chapter 3 describes the characteristics of blast loads and the capabilities of various analysis methods and software for the simulation of blast loads are investigated. A simplified model of the whole bridge for the simulation of blast effects is also introduced.

Chapter 4 defines the finite element model of a hypothetical bridge subjected to blast loads. Material properties and the constitutive model for various materials are presented. Simulation complications such as zero-energy modes, contact reliability, and application of gravity forces are discussed. The determination of time step and mesh size for the simulation are addressed. The importance of the total time necessary to complete one simulation run is also presented.

Chapter 5 presents a hypothetical bridge target subjected to various levels of blast loading to investigate blast-induced failure mechanisms. A description for each failure mechanism during the
simulations are presented and discussed. Selected time history curves are introduced to describe the importance of inertial effects during blast loading.

Chapter 6 discusses the application of seismic detailing for blast-induced highway bridges. The levels of seismic capacity for various charge loads are presented for design purposes and the importance of the scaled standoff distance, Z, is introduced. The simulations of the FEM simplified model of a typical highway bridge is designed with various levels of seismic detailing subjected to different levels of blast loading. Observed failure mechanisms for each blast scenario are identified. The significance of the location and size of an explosive relative to the bridge is discussed as well. Seismic-blast correlations are discussed from examining failure mechanisms during the simulations.

Chapter 7 presents a brief summary of the investigation and final conclusions of the research work. Recommendation for future work is presented as well.
CHAPTER 2. LITERATURE REVIEW ON EFFECTS OF BLAST LOAD

2.1 EXPLOSIONS AND SHOCK PHENOMENA

A detonation is characterized as a stable and very rapid chemical reaction which proceeds at supersonic speeds in the unreacted explosive material. The detonation velocity can range in the order of 22,000 to 28,000 ft/s [Department of Army (1990)]. The explosive material is converted into a very hot, dense, and high-pressured gas that radiates spherically away from an explosive source. The source of strong blast waves in air are provided by the volume of gas which had been the explosive material. Immediately behind the detonation front, pressures can range from 2,700,000 to 4,900,000 psi [Department of Army (1990)]. The duration of the shock wave is measured in microseconds.

As the region of compressed air (shock wave) radiates from the point of burst, pressure-driven effects occur rapidly for explosions closer to the target. This region of compressed air is subdivided into (1) overpressure resulting from the explosion in excess of the ambient pressure and (2) dynamic pressure which is deemed as the resulting air flow. Overpressure is due to the impinging shock front, hydrostatic pressure behind the front, and its reflections. Dynamic pressure is associated with mass transfer of air. Pressure loadings may be characterized in terms of a scaled range, \( Z = \frac{R}{W^{1/3}} \), where \( Z \) is the scaled standoff, \( R \) is the radial distance between center of blast source and target, and \( W \) is the charge weight of the explosive (usually expressed in terms of a TNT-equivalent charge weight). Units for the charge weight and radial distance are pounds and feet.

Depending on their physical state, explosive materials are categorized as: solids, liquids, or gases. Blast pressures, impulses, durations, and other blast effects of high-explosive solid materials are well understood and have been well established. Examples of these high-explosive materials include TNT, RDX, and ANFO [Department of Army (1990)].

When an explosion is confined within the structure, the effects of high temperatures and accumulation of gaseous products will exert additional pressures and increase the load duration within the
structure. The overall effect can be much greater than that of the incident shock pressure due to shock reflections occurring in the confined space. An example of such confinement is that of an explosion located beneath the deck of a bridge. Pressure build up between girders and near the abutments can amplify the applied load as shown in Figure 2.1. If the structure is not designed to sustain the effects of the internal pressures, the combined effects of these pressures may lead to catastrophic failure of the structure [Department of Army (1990)].

![Diagram of blast wave propagation beneath bridge deck](image_url)

Figure 2.1: Blast wave propagation beneath bridge deck [Winget et al. (2005)].

### 2.2 PHYSICAL BEHAVIOR TO BLAST LOADS

Contact blasts (which implies a small scaled range $Z$) on a pier can create high-intensity blast pressures that may lead to disintegration of structural components. This effect often causes breaching of a column, spalling of concrete cover from blast pressures and severe cracking, as shown in Figure 2.2.
A structural element subjected to blast loading may experience rapid reflections and refractions in the material. Depending on material properties, rapid rates of straining and significant disintegration may occur. For example, steel has a limiting deformation velocity that results in material strength increases with increasing strain rates. Consequently, ductile metals cannot deform fast enough to keep up with extreme loading, which may lead to yielding and fracture can be expected, especially if fabrication flaws are present. The Blue Ribbon Panel Report suggests that consequences of attack expressed as damage to bridges and tunnels that are concern as follows:

- Threats to the integrity of the structure (e.g., resulting in replacement of the facility or major repairs)
- Damage that inhibits the structure’s functionality for an extended period of time, such as closure of the facility for 30 days or more
Contamination of a tunnel resulting in extended closure or loss of functionality
Catastrophic failure resulting from an attack based on the threats described above

The Blue Ribbon Panel judged that the ordinary cost of construction to replace a major long-span bridge or tunnel on a busy interstate highway corridor in the United States may be $1.75 billion [BRP (2003)].

2.3 PROGRESSIVE COLLAPSE

American Society of Civil Engineers (ASCE) Standards 7-10 defines progressive collapse as the spread of an initial local failure from element to element, resulting eventually in the collapse of the entire structure or disproportionately large part of it [ASCE (2010)]. For structures that lack structural redundancy to resist the initial loss of key elements, gross collapse of the structure may occur. Although it is usually impractical for a structure to be designed to resist general collapse caused by severe abnormal loads acting directly on a large portion of it, specially designed systems can limit the effects of local collapse and to prevent or minimize progressive collapse.

The prevention of progressive collapse have begun to shape current design guidelines due to attacks such as the Oklahoma City bombing in 1995 where blast waves had sheared the columns that supported the fourth and fifth floors and collapsing on the third floor. Local failures of several structural components lead to global failure of the structure.

2.4 DESIGN GUIDELINES

Although no design codes exist particularly for the design of highway bridges subject to blast loads, there are several design codes that recommend provisions related to the design of mostly building structures to resist explosive loads. This section summarizes applicable design guidelines related to the design of structures (mainly buildings) to resist blast loads.
Structures to Resist the Effects of Accidental Explosions, UFC 3-340-02 (formerly TM 5-1300) [U.S. Departments of the Army, Navy, and Air Force, 2008]

This document was previously approved as a tri-service document; Army TM 5-1300, Navy NAVFAC P-397, and Air Force AFR 88-22, dated in 1990. The conversion and very minor revisions of the 1990 document into UFC 3-340-02 was accomplished in 2008.

Considered to be one of the most widely used publications by both military and civilian organizations, this manual includes comprehensive blast analysis and design features, including information on items such as (1) blast, fragment, and shock-loading; (2) principles of dynamic analysis; (3) reinforced and structural steel design; and (4) a number of special design considerations. Although UFC 3-340-02 does not establish regulatory requirements, it may be used to satisfy any code’s explosive safety requirements.


For planners and civilian designers, this report provides a comprehensive guide to incorporate physical security considerations in to their designs or building retrofit efforts.

Blast-Resistant Highway Bridges: Design and Detailing Guidelines – NCHRP Report 645 [NCHRP (2010)]

The report presents code-ready language containing general design guidance and a simplified design procedure for blast-resistant reinforced concrete bridge columns. Results from experimental blast tests are also presented in the report to investigate the effectiveness of several design techniques.

ISC Security Criteria

To ensure that security becomes an integral part of the planning, design, and construction of new federal office buildings and major modernization projects, the Interagency Security Committee (ISC)
developed the ISC Security Design Criteria. Security in all building systems and elements were considered in the criteria.

By Executive Order in 1995, the ISC was established to develop long-term construction standards for locations requiring blast resistance or other specialized security measures.

2.5 LOAD AND RESPONSE METHODS FOR BLAST ANALYSIS

Computer models have been valuable resources in characterizing blast-load distribution and the resulting column response which are validated by experimental data. Results based on blast testing of structures have been presented by researchers, e.g., small-scale blast tests on square and round non-responding columns [NCHRP 645 (2010)].

2.5.1 Experimental Methods

Although there have been limited tests regarding to detailed knowledge based on blast load effects, blast test data can be used to develop reliable material models of structural components. These material models based on blast test data can be used to improve simulation results in predicting the response of a material due to blast load effects on structural components.

Design and Analysis of Hardened Structures to Conventional Weapons Effects [Department of Army (2010)] carried out blast tests on scaled models of structures subjected to blast loads generated by explosive charges to develop its database. Several researchers have provided response results of FRP-retrofitted reinforced concrete slab structures subjected to blast loads [Kim et al. (2009)]. Nassr et al. [Nassr et al. (2011)] field tested typical wide-flange steel beams under blast loading to study the dynamic response of the material.
2.5.2 Computational Modeling Methods

The finite-element method provides bases for the majority of computational models to predict load and material response. The finite-element method has the ability to solve complex geometries at rather high computational speeds.

Blast prediction techniques are generally subdivided into two methods (1) load determination and (2) response determination. Computer simulations generally employ first-principle or semi-empirical methods to predict load and material response. First-principle methods solve systems of equations starting directly from fundamental laws of physics without making assumptions. Semi-empirical methods utilize extensive data from past experiments along with good engineering judgment to predict load and material response.

Due to lack of experimental data available to the public, it is very difficult to validate first-principle models and any validation applies only to specific scenarios that were experimentally considered [National Research Council (1995)]. When a lack of applicable data exists, response predictions based on first-principle results can be developed with good engineering judgment and experience.

Standard practice for most blast-resistant designs employs a single-degree-of-freedom (SDOF) analysis. SDOF analyses allows inelastic (i.e., permanent) deformations to dissipate energy associated with dynamic blast loads. By applying work and energy principles to the real system an engineer can transform the real system into an idealized system and obtain equivalent system properties which behave closely to the real system in both space and time. The idealized system consists of a concentrated mass-spring-load system, where the distributed masses of the real structure are lumped together into a series of concentrated masses supported by weightless springs where the strain energy is assumed to be stored. Concentrated loads acting on the masses replace the distributed loads. SDOF results compare well with experimental test data when members experience large plastic deformation [Department of Army (1990)].
The response of the blast-loaded columns and slabs in the Alfred P. Murrah federal building that was attacked in Oklahoma City in 1995 were acquired from a SDOF analysis [Mlakar et al. (1998)]. The response of SDOF systems subjected to idealized blast loadings may be presented in form of equations and non-dimensional curves. ConWep [U.S. Army Corps of Engineers (2001)], which is a widely used blast prediction application, employs SDOF models for calculations.

Computer programs which incorporate fluid mechanics computations are the most sophisticated level of load determination. Hydrocodes are computational continuum mechanics tools that use the mechanics and characteristics of fluids and fluid flow under highly dynamic conditions (e.g., air in the case of blast). There consist four hydrocode methodologies: Lagrangian, Eulerian, Coupled Eulerian-Lagrangian, and Arbitrary Lagrangian-Eulerian. Of the four, Eulerian method is not practical for the simulation of structure-medium interaction during blast loads. Eulerian solutions accumulate advection and interface tracking errors and are often limited to relatively short simulation times (on the order of hundreds of microseconds).

Computer modeling involving explosion simulation and fluid-structure interaction in theory can provide the most accurate responses of structures subjected to blast loading. However, such modeling requires very small Finite Element size in the air and explosive domains, which hampers its application to solve real structural problems.

Comparatively, methods that utilize both first-principle and semi-empirical methods have a wider range of applicability compared to semi-empirical methods and require less computational effort and better accuracy than first-principal methods.

2.6 MODELING OF BLAST EFFECTS USING LS-DYNA

LS-DYNA is an advanced general-purpose finite-element code developed by the Livermore Software Technology Corporation (LSTC). Using explicit time integration, the code’s origins lie in highly nonlinear, transient dynamic finite element analysis. Coupled 3D nonlinear general-purpose finite
element procedures provide the highest level of response computation techniques. They account for the interaction of loading and response over time, thus providing the most accurate predictions of both load and response. LS-DYNA is one of the few programs capable of coupling blast pressures with structural response and has been used extensively in blast load simulations. Vasudevan (2011) has compared experimental data of doubly reinforced concrete slabs subjected to blast loads with LS-DYNA. Wang (2001) used LS-DYNA3D to simulate a landmine explosion causing shock wave propagation in soil and air and then interaction with a structure. The simulation was compared with results from a well-defined landmine-explosion experiment.

For this research, a pure Lagrangian approach is presented by directly applying the blast load pressure onto the structure through means of empirical curve-fitting. This approach significantly reduces computational time by excluding fluid-structure interaction.

2.7 DESIGN OF BRIDGES SUBJECTED TO BLAST LOAD

Tailored specifically for bridges, Winget et al. [Winget et al. (2005)] summarizes the results of ongoing research to develop performance-based blast design standards. The potential effects of blast load on bridges and structural design and retrofit solutions to counter blast effects are then discussed. Tokal-Ahmed [Tokal-Ahmed (2009)] utilized 3D analysis program (Extreme Loading for Structures) to simulate a typical bridge structure subjected to blast loads and compared results with a simplified SDOF analysis using a blast load response spectra. Yi [Yi (2008)] addressed high-fidelity simulation of blast load effects on bridge components to identify typical mechanisms responsible for causing damage/failure of typical components and investigated performance of different components during blast events.

Although the field of bridges subjected to blast loads is relatively new, one of the major funded research works is the National Cooperative Highway Research Program’s (NCHRP) Project 12-72. The primary objective of the research was to improve the structural performance and resistance to explosive effects for bridges by developing design guidance. The project report contains effective methods to
mitigate the risk of terrorist attacks against critical bridges. It recommends risk assessment guidelines for bridges, discusses blast effects on bridges, and provides retrofitting and structural design guidelines.

The latest AASHTO specifications [AASHTO (2010)] provides some aspects to be considered in blast design in Sections 3.15 and 4.7.6; although no blast analysis tool or methodology was introduced.

The purpose of the research was to investigate seismic-blast correlation by identifying blast-induced failure mechanisms which may be present during earthquakes.
CHAPTER 3. BLAST LOAD

3.1 FORMATION OF BLAST WAVE

3.1.1 Simplified Model of Unconfined Blast Load

It is of importance to note the differences between static, dynamic, and short-duration dynamic loads. Generally, static loads (loads that are assumed to act on the structure for long periods of time) such as gravity, do not produce inertia effects; therefore, are not time dependent [Fertal et al. (2000)]. Loads induced often times by earthquake or wind gusts are dynamic loads and are time-dependent (normally measured in tenth of seconds). Short-term dynamic loads produced by explosion and debris are non-oscillatory pulse loads which are approximately one thousand times shorter than that of an earthquake [Conrath et al. (1999)]. Figure 3.1 provides an example of different dynamic hazards with their respective amplitude-frequency relationships. These dynamic hazards can be categorized as natural (earthquakes, wind, etc.) and man-made (blast).

An explosion is a sudden release of energy that generates light, heat, pressure and noise. Part of the energy is released as thermal radiation, and the other part is coupled into the air (air blast) and soil (ground-shock) as radially expanding shock waves [FEMA (2006)]. The shock (or blast) wave which

Figure 3.1: Qualitative amplitude-frequency distribution for different hazards [Ettouney (2001)].
accompanies the explosion contributes to the majority of material damage at the surface or at a low or moderate altitude in the air. The overpressure rises nearly instantaneously to its peak and decays as the shock wave expands radially outward from the explosion source. After a very short time, the pressure may drop below the ambient pressure (underpressure) in which a partial vacuum is created from the low-pressure region, causing a wind that initially follows the blast wave which creates a suction effect. This development is depicted in Figure 3.2 at six successive times.

![Figure 3.2: Variation of overpressure in air with distance at successive times [Glasstone (1977)].](image)

The dynamic pressure also increases nearly instantaneously with the arrival of the shock front which consists of a strong wind away from the explosions, and then a very feeble wind toward the explosion. Unlike the overpressure, the dynamic pressure never enters a negative phase because it is a measure of kinetic energy (i.e., energy of motion) and the dynamic pressure is determined from using the square root of the wind velocity. Figure 3.3 shows a typical blast pressure variation with time.
The blast loading is defined by the pressure and impulse (equal to the area under the pressure-time history curve). While the exact values that define the free-field pressure with time variation profile may vary depending on the size of the explosive charge and the location of interest, all pressure-time histories will have the same general assumed form shown in Figure 3.3, except those very close to the detonation. The positive phase duration can vary between a few microseconds and several milliseconds; of course, this depends on the type of explosive and the proximity to the target [Kinney (1985)]. The negative phase is usually neglected in most cases because its contribution on the maximum response has little effect.

Once the overpressure reaches its maximum, it will begin to decay, as depicted in Figure 3.3. Because the overpressure drops to zero in finite time, the decay of blast overpressure does not follow a typical logarithmic decay relation. In terms of a decay parameter, $\alpha$, and of a time, $t$, which is measured from the instant the shock front arrives, the pressure can be obtained from a quasi-exponential form:

$$ P = P_{so} \left[ 1 - \frac{t}{t_o} \right] e^{-\alpha t} $$  \hspace{1cm} (3.1)

where $P$ is the instantaneous overpressure at time $t$, $P_{so}$ is the maximum or peak static overpressure observed when $t$ is zero, and $t_o$ is the positive phase duration.
Because similar shock waves can be created from different charge weights and standoff distances, it is extremely useful to compare all explosive materials on an equal footing. Experiments have been carried out to determine the characteristics of the blast wave generated by an explosion with a given reference set of explosion data (usually TNT). Generally, scaling equations relate the parameters needed to define the profile in Figure 3.3. The most commonly used form of blast scaling is the cube-root scaling law [Conrath et al. (1999)] shown in Equation 3.2.

\[
Z = R / W^{1/3}
\]  

3.2

Figure 3.4 illustrates the equivalent charge weight, \( W \), and the standoff distance, \( R \), between the blast source and the target with blast loading applied to the structure.

![Illustration of blast loads on a building.](image)

Once the scaled standoff distance has been established the parameters required to define an idealized blast wave can be predicted from commonly used “standard” air blast curves, often referred as “spaghetti charts”. Figure 3.5 shows one these charts.
The basis for these charts and curves are obtained from a vast collection of theoretical predictions and empirical information and exist for both hemispherical and spherical “free-field” bursts.

3.1.2 Prediction of Blast Pressure

There have been a number of studies during the 1950’s and 1960’s focused on blast wave parameters for conventional high explosive materials. Brode (1955) estimated peak overpressure due to spherical blast based on scaled standoff distance. Newmark and Hansen (1961) introduced a relationship to calculate the maximum overpressure for a high explosive charge detonation at the ground surface.
All blast parameters are primarily dependent on the distance from the explosive source and the amount of energy released by a detonation. Blast wave parameters such as blast wavefront velocity \( U_s \), air density behind the wavefront, \( \rho_s \), and the maximum dynamic pressure, \( q_s \), are given as:

\[
U_s = \sqrt{\frac{6 p_{so} + 7 p_0}{7 p_0}} \cdot a_0
\]

\[
\rho_s = \frac{6 p_{so} + 7 p_0}{p_{so} + 7 p_0} \cdot \rho_0
\]

\[
q_s = \frac{5 p_{so}^2}{2 (p_{so} + 7 p_0)}
\]

where \( p_0 \) is ambient air pressure ahead of the blast wave, \( \rho_0 \) is the density of air at ambient pressure ahead of the blast wave, and \( a_0 \) is the speed of sound in air at ambient pressure.

The ratio of the weight of an explosive to an equivalent weight of TNT is defined as TNT equivalency. It is of common practice to use TNT equivalencies to relate the energy output of common explosives to that of TNT. Bashera [Bashera (1994)] states that most of the data related to explosions used TNT and thus data related to any other explosive should be benchmarked against its TNT equivalent. Table 3.1 summarizes conversion factors for different explosives based on peak pressure and impulse [Department of Army (1990); Tedesco (1999)].

<table>
<thead>
<tr>
<th>Explosive</th>
<th>Equivalent Weight, Pressure (lb-m)</th>
<th>Equivalent Weight, Impulse (lb-m)</th>
<th>Pressure Range (psi)</th>
</tr>
</thead>
<tbody>
<tr>
<td>ANFO</td>
<td>0.82</td>
<td>--</td>
<td>1-100</td>
</tr>
<tr>
<td>Composition C-4</td>
<td>1.37</td>
<td>1.19</td>
<td>10-100</td>
</tr>
<tr>
<td>HBX-1</td>
<td>1.17</td>
<td>1.16</td>
<td>5-20</td>
</tr>
<tr>
<td>Minol II</td>
<td>1.20</td>
<td>1.11</td>
<td>3-20</td>
</tr>
<tr>
<td>PETN</td>
<td>1.27</td>
<td>--</td>
<td>5-100</td>
</tr>
<tr>
<td>TNT</td>
<td>1.00</td>
<td>1.00</td>
<td>Standard</td>
</tr>
<tr>
<td>TRITONAL</td>
<td>1.07</td>
<td>0.96</td>
<td>5-100</td>
</tr>
</tbody>
</table>
Existing literature and methods for predicting loads are all based on TNT; therefore, to ensure comparability of results with previous data and research projects, TNT equivalency should be used.

### 3.1.3 Air Burst

Air burst explosion is an explosion which is located at a distance from and above the structure so that reflections from the ground of the initial wave occur before the arrival of the blast wave at the structure. The air burst environment is produced by detonations which occur above the ground surface and at a distance away from the protective structure so that the initial shock wave, propagating away from the explosion, impinges on the ground surface prior to arrival at the structure. As the shock wave continues to propagate outward along the ground surface, a front known as the Mach front (Figure 3.6) is formed by the interaction of the initial wave (incident wave) and the reflected wave. This reflected wave is the result of the reinforcement of the incident wave by the ground surface [Department of Army (1990)]. Therefore, shock can be considered as a plane wave (uniform pressure) over the full height of the front for design purposes.

![Air burst blast environment](Figure 3.6)

Figure 3.6: Air burst blast environment [Department of Army (1990)].

Various semi-empirical analytical tools (most notably ConWep and BlastX) are being used to model blast-effects on structures. Winget et al. (2004) conducted parameter studies with ConWep and
BlastX to “evaluate the effectiveness of structural retrofits, refine the performance-based standards, and develop general blast-resistant guidelines specifically for bridges”.

Developed by USAE Engineer Research and Development Center, ConWep is a collection of conventional weapons effects calculations from the equations and curves of TM 5-855-1 [Department of Army (1990)]. ConWep code can only consider free-air blast; therefore ground interaction is not considered. Many designers have requested and referenced ConWep for government projects. The algorithm for blast loads in LS-DYNA is based on an implementation by Randers-Pehrson and Bannister (1997) of the empirical blast loading functions implemented in the ConWep code [Kingery and Bulmash (1984)].

BlastX code performs calculations of the shock wave and confined detonation products pressure and venting for explosions either internal or external to a structure [Science Applications International Corporation (2006)]. Through fundamental first-order principles of wave reflection, BlastX can track pressure values as they radiate from an explosion source and as they reflect off surfaces. Based on experimental and analytical research, BlastX significantly overestimate loads on slender square and circular members (i.e., bridge columns) subjected to blast loads; consequently, BlastX is not capable of modeling round geometries such as columns of bridge piers [NCHRP (2010)].

3.2 LS-DYNA SIMULATIONS

3.2.1 Bridge Components Subjected to Blast Loads

Applying blast wave load accurately on various bridge components is a difficult task. For example, assume that a 2000-lb TNT charge is detonated under a 60-ft span hypothetical highway bridge at point C, as shown in Figure 3.7. The TNT charge is located 10 ft away from column A and 50 ft away from column B. Size of the pier is 3ft x 3ft.
The following criteria for simulation of bridge components subjected to blast load should at least satisfy:

- The pressure and impulse of blast wave near points A and B should be similar to that generated by experimental or semi-empirical data using ConWep program [USAE Engineer Research & Development Center (2005)].
- The time of arrival of blast waves reaching points A and B should be similar to those by ConWep program so that the time sequences of blast wave load and structural response have correct dynamic effects.
- The blast load should have the ability to be assigned to bridge components.

Many methods exist for analyzing the responses of structures subjected to blast loads, ranging from simple SDOF analyses to highly complex, 3D nonlinear finite element analyses. Low accuracy methods in the prediction of either load or structural performance commonly use simplified methods such as single or multi-degree-of-freedom, pressure-impulse (P-I) diagrams, and response surfaces developed from finite element analysis [Sunshine et al. (2004)].

A blast load is assumed as a single load distribution on a SDOF model that does not vary with position along the member. In reality, blast loads are spherical or hemispherical waves that will propagate along the length of a member, applying the load at different positions along the member at different points in time. Converting the blast load to a single concentrated force is not an accurate representation of the actual blast scenario. Another major disadvantage is that SDOF analysis does not simulate nor predict the failure mechanism of the structure when subjected to an extreme blast scenario [Williamson and Winget (2005)]. Simplified methods must assume structural response modes, component
interactions and blast loads; therefore, blast scenarios with a small scaled standoff distance are not expected to be adequately covered by SDOF or MDOF idealizations. Our goal is to investigate failure mechanisms of a bridge under blast load and a simple SDOF analysis is not an accurate and acceptable approach.

A more sophisticated and accurate load and response determination analysis can be obtained using Hydrocodes (computational fluid dynamics) such as LS-DYNA [LSTC (2008)]. Finite element software employing hydrocodes can include phenomena not captured by simplified analysis techniques, such as localized member failure, multiple reflections off complicated geometries, and blast loads coupled with structural response. Traditionally, there are two approaches of blast load simulation in LS-DYNA: *LOAD_BLAST blast load is applied directly on a structure or simulating the detonation process by ALE method. In this research, *LOAD_BLAST function is applied directly to Lagrangian mesh.

3.2.2 Blast Simulation by *LOAD_BLAST

ConWep is a collection of conventional weapon effects and calculations based on equations and curves in the TM 5-855-1 army handbook [Department of Army (1998); USAE Engineer Research & Development Center (2005)]. The empirical blast loading functions implemented in the ConWep code provides bases for the empirical blast loading functions implemented in LS-DYNA keyword (*LOAD_BLAST). *LOAD_BLAST blast load replaces the computation of wave propagation on the structure. The use of *LOAD_BLAST function allows the use of a much smaller model since only the structure is modeled.

3.2.3 Proposed Blast Load Simulation Approach

Although an explosion on a bridge deck may cause local damage, the columns of the bridge are the crucial members that support bridge loads and this research is focused on global failure mechanisms of reinforced concrete columns subjected to blast loading; therefore, the weight of the deck is converted into a distributed pressure across the bent of the bridge. A simplified model of the whole bridge for the
simulation of blast effects has been proposed. Hence, a pier-bent approach is modeled with Lagrangian structure element with *LOAD_BLAST blast function applied to the elements.

This simplified approach neglected the reflection and superposition of blast wave near the structure components, but it provides a tool to qualitatively understand the failure mechanisms of bridges subjected to blast loading, and to reveal the effectiveness of the multi-hazard detailing on the blast resistance of ordinary highway bridges.
CHAPTER 4. MODELING OF BRIDGE

4.1 HYPOTHETICAL TARGETED BRIDGE

The National Bridge Inventory database contains detailed technical and engineering information about hundreds of thousands of bridges in the United States including year built, bridge design, condition and many other fields. Federal Highway Administration (FHWA) compiles bridge inventory data provided by all states, although individual states maintain their own inventory of bridges in the country. Currently, the NBI database includes over 600,000 bridges located in 50 states including Puerto Rico.

Through a detailed search of bridges in the NBI database and based on a similar type of study for other geometrical features of a bridge, a three-span bridge has been selected as a hypothetical bridge for the development of finite element simulation of the bridge. As-built drawings of the bridge with similar features were obtained for the hypothetical bridge selected through the search of NBI database. The plan of the bridge on drawing is displayed in Figure 4.1.
Figure 4.1: Typical bridge plan.
A 3-span non-continuous bridge is considered as the hypothetical bridge. Plan and elevation of the bridge are shown in Figure 4.2. Key parameters of the bridge geometry and design load are listed in Table 4.1.

![Figure 4.2: Typical bridge plan and elevation.](image)

Table 4.1: Typical hypothetical bridge parameters.

<table>
<thead>
<tr>
<th>ITEM</th>
<th>VALUE</th>
</tr>
</thead>
<tbody>
<tr>
<td>Redundancy</td>
<td>Non-continuous</td>
</tr>
<tr>
<td>Length of Maximum Span</td>
<td>62 ft</td>
</tr>
<tr>
<td>Number of Spans in Main Unit</td>
<td>3</td>
</tr>
<tr>
<td>Design Load</td>
<td>MS 18 or HS 20</td>
</tr>
<tr>
<td>Deck Width</td>
<td>40 ft</td>
</tr>
<tr>
<td>Deck Thickness</td>
<td>13 in.</td>
</tr>
<tr>
<td>Lanes on Structure</td>
<td>2</td>
</tr>
<tr>
<td>Height of Pier</td>
<td>16 ft</td>
</tr>
<tr>
<td>Number of Piers</td>
<td>3 x (2 group) = 6</td>
</tr>
<tr>
<td>Pier Section</td>
<td>Rectangular 3.0 ft x 3.0 ft</td>
</tr>
<tr>
<td>Material / Design Type</td>
<td>RC concrete pier, bent &amp; deck, steel stringer</td>
</tr>
</tbody>
</table>
Rebar detailing in bent and piers are shown in Figure 4.3. Longitudinal rebars from piers extend into bents as well as footings. Sections used for stringers are shown in Table 4.2. A bearing, which defines the boundary conditions of piers and stringers, is another key member of the bridge. Since elastomeric bearings are extensively used to replace old bearings, they are used in this research as well.

Figure 4.3: Details of (a) Pier Section (b) Bent Section.

Table 4.2: Steel sections used for bridge stringers.

<table>
<thead>
<tr>
<th>Diaphragm</th>
<th>Intermediate</th>
<th>16WF36</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>End</td>
<td></td>
<td>18C4.7</td>
</tr>
<tr>
<td>Span</td>
<td></td>
<td></td>
</tr>
<tr>
<td>1</td>
<td>Stringer No. 1, 6</td>
<td>Stringer No. 2, 3, 4, 5</td>
</tr>
<tr>
<td>2</td>
<td>36WF150</td>
<td>36WF150</td>
</tr>
<tr>
<td>3</td>
<td>36WF150</td>
<td>30WF108</td>
</tr>
</tbody>
</table>

4.2 MODELING OF BRIDGE COMPONENTS

A detailed pier-bent model of the bridge described previously has been built in LS-DYNA. The following steps have been taken for the numerical simulation of blast loads on a highway bridge:

- Determine computational solving technique
- Determine mesh scheme
- Determine mesh size, topology and plan the FEM model
- Assign material properties and boundary conditions
- Construct a detailed model for each structural component and verify
• Control hourglassing, eroding, dynamic relaxation, and contacting parts
• Verify that requirements are satisfied, if not modify according
• Simulate bridge and interpret results

An explicit solver in LS-DYNA has been chosen since it can handle a large number of elements. There are two main descriptions for material movement in LS-DYNA (i.e., Lagrangian and Eulerian). In the Lagrangian description, numerical mesh distorts with material movement while numerical mesh is fixed in space in Eulerian description. Figure 4.4 displays movement of Lagrangian and Eulerian mesh. The elements of the pier-bent model are constructed entirely of Lagrangian mesh. Although numerical instabilities arise due to distortion and grid tangling of the mesh when elements experience large displacements using the Lagrangian description, an element rezoning or erosion technique may be used to avoid severe element distortion during the simulation of blast load effects.

![Figure 4.4: The Lagrangian mesh (left) and Eulerian mesh (right).](image)

Commonly used finite element analysis software packages, such as SAP2000 and STAAD Pro, model typical bridge members as frames. Also, since these kinds of software packages use an implicit solver technique to solve the governing equations of motion for the system under consideration, the drawback is that they require the factorization of the stiffness matrix for each time step; even more, this requirement greatly increases computation time for problems in which the stiffness of elements in the structural model change due to nonlinear response. A detailed bridge member shape using FEM modeling is desired when geometry details may change the load characteristics significantly. A detailed finite element model should include as much information on bridge geometry and behavior when bridge
components are subjected to blast load so that all failure modes could be identified. Solid elements are used for the majority of bridge members including footing, pier, bent, and bearing.

Since blast test experiments of a pier-bent model is unlikely to be available, behavior of each member should be investigated separately using available blast test data on bridge components. A detailed description of different bridge components for modeling is described in the following.

**Concrete Columns, Bent, and Footing**

Pier columns consist of concrete cover, core and steel rebar, as depicted in Figure 4.5.

![Figure 4.5: Modeling of pier components.](image)

A detailed modeling of rebars is important for the simulation of blast load effects on concrete structures [Krauthammer and Otani (1997)]. Reinforced concrete members are usually modeled by an
equivalent monolithic element that behaves both as steel and concrete during dynamic loading. Though an equivalent monolithic element maybe suitable for hazard loading such as earthquakes, wind, etc., it is not appropriate for reinforced concrete members subjected to blast loads. Yi (2009) demonstrated this fact by modeling a concrete column subjected to blast loads (i) as consisting of pure concrete, (ii) by equivalent monolithic element and (iii) by modeling rebars and concrete separately. From experimental data on reinforced concrete columns subjected to blast loads [Magnusson and Hallgren (2004)] it was determined that Yi’s third case, i.e., column with concrete and rebars modeled separately, is more reasonable.

In order to investigate failure mechanisms of the pier-bent system during a blast load event, a detailed modeling of the bridge pier, pier bent and footing has been modeled. Bottom of pier footing has a fixed boundary condition as per construction drawing. Confinement effects of rebar on core concrete have been considered by modeling bridge piers with cover concrete and core concrete as separate layers. Longitudinal rebars have been extended into the footing and bent.

**Equivalent Deck and Bearing**

To represent the actual weight of the deck, and equivalent deck system is created, as shown in Figure 4.6. The weight of the deck is uniformly distributed across the bent by block supports which rest upon each bridge bearing. Steel sections are connected along the block supports so that the movement of a block support influences the movement of other block supports. During the blast event, the elastomeric bearings will encounter large deformations. To prevent the equivalent deck from falling off the bent during the blast simulation, a lateral restraint parallel to the blast load direction is applied to the block supports which rest upon the elastomeric bearings. Due to the possible impact force produced by the equivalent deck and the pier bent touching, contact functions are defined.
Figure 4.6: Modeling of equivalent deck and support bearings.

A step-by-step finite element modeling of the pier-bent system is depicted in Figure 4.7(a) to (f).

(a) Pier Footing.
(b) Rebar cage added to footing.

(c) Concrete cover and core added to rebar cage.
(d) Reinforced bent applied to pier columns.

(e) Elastomeric bearings placed atop bent.
4.3 MATERIAL AND CONSTITUTIVE MODELING

The characteristics of material properties used in reinforced concrete construction are dependent on the rate of loading. The strain rate for static loading is approximately $10^{-5} \text{s}^{-1}$. For impact and blast loadings, strain rates range between 1 and $1000 \text{s}^{-1}$ [Bischoff and Perry (1991)]. During an explosion, structural materials will experience very high rates of loading for a very short period of time. Under dynamic loading conditions such as blast loading, the mechanical properties of structural materials can be quite different from that under static loading.

4.3.1 Concrete Material Properties and Constitutive Model

Under dynamic loading, concrete may gain values that are higher than static conditions. It has been shown that the design compressive strength of concrete can increase about 25 to 30 percent during
dynamic loading of concrete [Bischoff and Perry (1991)]. Figure 4.8 displays stress-strain curves of concrete at various strain rates.

![Stress-strain curves of concrete at different strain-rates](image)

Figure 4.8: Stress-strain curves of concrete at different strain-rates [Ngo et al. (2004)].

LS-DYNA contains several constitutive models that can be used to represent concrete during high rates of loading. Among these models, Material Type 159 (Continuous Surface Cap Model, CSCM) has been found to be appropriate for concrete subjected to dynamic loads with high rates of loading. Material type 159 was developed to predict the dynamic performance – both elastic deformation and failure – of concrete used in safety structures when involved in a collision with a motor vehicle [Murray et al. (2007)]. The main features of the model are:

- Isotropic constitutive equations
- Three stress invariant yield surface with translation for pre-peak hardening
- A hardening cap that expands and contracts
- Damage-based softening with erosion and modulus reduction

To study the effects of high strain rates on the behavior of concrete, Zadeh (2011) simulated concrete cylinders subject to blast loading using the CSCM material model and found that the calculated DIF values were very close to experimental concrete specimens subject to high strain rates performed by Ross et al. [Zadeh (2011); Ross et al. (1995)].
Under extreme loading conditions, a failure criterion is needed to “fail” the material. Concrete is a hydrostatic pressure-dependent material; therefore, the failure criterion is based on the unconfined compressive strength for concrete. Figure 4.9 shows a typical stress-strain curve for concrete, including the effects of high strain rate.

Figure 4.9: Stress-strain curve for concrete [Department of Army (1990)].

Transverse reinforcement may enhance the ductility and member strength in reinforced concrete members with axial compression forces. At locations where stringent seismic detailing is not required for transverse reinforcement in reinforced concrete columns has also been considered. Based on the model proposed by Légeron and Paultre (2003) the increase of strength and ductility of concrete is related by the effective confinement index, $I'_e$, and is used to predict the uniaxial behavior of confined concrete under compression as follows:

$$I'_e = \frac{f'_{le}}{f'_c}$$

where $f'_c$ is the unconfined concrete strength and $f'_{le}$ is the effective confinement pressure at peak stress, which is a measure of the restraint applied by the hoops to the lateral expansion of the confined concrete core under axial compression [Légeron and Paultre (2003)]. The model has been shown to predict very well the moment curvature envelope and the force-displacement response of a wide range of columns with concrete strength ranging from 4.35 ksi (30 MPa) to 17.4 ksi (120 MPa) confined with steel of yield
strength ranging from 36.3 ksi (250 MPa) to 203.1 ksi (1400 MPa). Figure 4.10 displays the stress-strain curve of confined and unconfined concrete.

![Stress-strain curve of confined and unconfined concrete](image.png)

Figure 4.10: Stress-strain curve of confined and unconfined concrete [Légeron and Paultre (2003)].

The mechanical properties of unconfined concrete and confined concrete from the transverse reinforcement have also been considered for blast load simulation by merging nodes shared between concrete solid elements and steel reinforcement beam elements; thus, LS-DYNA automatically simulates the confinement of concrete based on theoretical equations. Steel and concrete elements are assumed perfectly bonded from merging of the nodes, thus no slippage occurs between rebar and concrete. The dynamic material property for the unconfined strength of concrete is presented in Tables A.1 of Appendix A.

### 4.3.2 Steel Material Properties and Constitutive Model

Steel is a critical component of reinforced concrete structures subjected to blast loads. The inelastic response of metallic materials to dynamic loading can be easily monitored and assessed due to the isotropic properties. From past experimental data, it has been found that the yield strength can almost be doubled for mild steel under high strain rates; the ultimate tensile strength can increase by about 50% and the upper yield strength even higher. On the other hand, with increasing strain rate, the ultimate
tensile strain decreases. Malvar [Malvar (1998)] provides a more detailed understanding of steel reinforcing bars under the effect of high strain rates.

It has been observed that the failure strain for steel ranges between 13 to 20 percent. Hence, the failure criterion is based on the maximum principal strain criterion. Stress-strain curve for reinforcing steel can be seen in Figure 4.11.

![Stress-strain curve for reinforcing steel](image)

Figure 4.11: Stress-strain curve for reinforcing steel [Department of Army (1990)].

LS-DYNA’s Material Type 003 (Plastic Kinematic) is used to apply both initial elastic data as well as the secondary plastic (post-yield) portion of the stress-strain curve for steel reinforcing. Beam elements are formulated using Hughes-Liu beam formulation with one-integration point which is located at the center of the element; thus it can be modeled with solid and shell elements, which also have one-integration point at the midsection of the element. Dynamic properties of steel rebar are presented in Table A.3 of Appendix A.

4.3.3 Elastomeric Bearing Material Properties

Elastomeric bearings, as shown in Figure 4.12, are designed to accommodate longitudinal movements and rotations of the bridge superstructure while transmitting vertical loads through to the structure’s foundations and have been extensively used throughout the United States. Elastomeric bearings may be used in all bridge types with expansion movements limited to 2 to 3 inches. Reinforcing steel shims are implemented into the elastomers to increase the compressive stiffness.
Elastomers are highly viscoelastic and are strongly rate dependent; hence, the mechanical properties of elastomers are very sensitive to the rate of loading. To investigate the performance of elastomeric bearings under high strain rate loads and its influence to the performance of bridge girders, Yi (2009) simulated the effects of high rates of loading on elastomeric bearings using LS-DYNA and his model performed fairly well by capturing mechanical characteristics of bearings observed during experiments, as shown in Table 4.3.

Table 4.3: Validation of FEM bearing model [Yi (2009)].

<table>
<thead>
<tr>
<th></th>
<th>Calculation</th>
<th>Experiment</th>
</tr>
</thead>
<tbody>
<tr>
<td>Compressive Modulus</td>
<td>16463.4 psi</td>
<td>16462 psi</td>
</tr>
<tr>
<td>Compressive Strength</td>
<td>14564 psi</td>
<td>12300 – 20300 psi</td>
</tr>
<tr>
<td>Tensile Strength</td>
<td>3679.5 psi</td>
<td>3625 psi</td>
</tr>
<tr>
<td>Tensile Failure Strain</td>
<td>35.6%</td>
<td>200 – 600%</td>
</tr>
<tr>
<td>Shear Failure Strain</td>
<td>61.9%</td>
<td>-</td>
</tr>
</tbody>
</table>

Displayed in Table 4.3, elastomeric bearings during numerical simulations fail at 35.6% tensile strain whereas the tensile strain from experimental data observed values in the range of 200 – 600 %, though tensile failure strength remained the same. Due to the lack of detailed information on tensile behavior of elastomeric bearings, the failure criterion was based on the tensile strength and not the tensile strain of elastomeric bearings. Table 4.4 displays material properties for the finite element modeling of elastomer and reinforcing steel shims.
Table 4.4: Selected parameters for elastomeric bearing material in FEM model [Yi (2009)].

<table>
<thead>
<tr>
<th>Material</th>
<th>Property</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>ELASTOMER</td>
<td>Bulk Modulus</td>
<td>12050 psi</td>
</tr>
<tr>
<td></td>
<td>Shear Modulus</td>
<td>86 psi</td>
</tr>
<tr>
<td></td>
<td>Tensile Failure Strain</td>
<td>0.305</td>
</tr>
<tr>
<td></td>
<td>Compressive Failure Stress</td>
<td>20300 psi</td>
</tr>
<tr>
<td>STEEL SHIMS</td>
<td>Density</td>
<td>7.29E-04 lbs/ft^2</td>
</tr>
<tr>
<td></td>
<td>Yield Stress</td>
<td>40000 psi</td>
</tr>
<tr>
<td></td>
<td>Elastic Modulus</td>
<td>3.05E + 7 psi</td>
</tr>
<tr>
<td></td>
<td>Failure Strain</td>
<td>0.23</td>
</tr>
</tbody>
</table>

LS-DYNA (*MAT_006) linear viscoelastic material model was used to simulate the mechanical properties of elastomers. The tensile strength rather than the tensile strain controlled the failure of bearings based on the assumption that the tensile stiffness was to be the same as the compressive stiffness [Yi (2008)]. The dimension of an elastomeric bearing is 22 inches in width, 9 inches in length, and 3 inches thick.

4.4 BODY FORCE, DYNAMIC RELAXATION, HOURGLASSING, AND CONTACT

**Body Force and Dynamic Relaxation**

Inertial effects are of importance in the simulation of blast loading and material response. Body forces due to inertia effects and all other forces are applied to the structure as dynamic forces due to LS-DYNA explicit solver. To overcome this dilemma, a dynamic relaxation is applied during the duration of unwanted dynamic effects by creating a critically damped dynamic system to rapidly reduce the dynamic effects. At time zero of the simulation, dynamic relaxation is applied for unwanted dynamic effects and until the structure has maintained its natural frequency. Once the structure obtains its natural period, which is approximately 25 – 45 milliseconds, the dynamic relaxation condition is removed as the blast load is applied.

**Hourglassing**

Although one-point integration solid and shell elements used in LS-PREPOST LS-DYNA save extensive amounts of simulation time, they are prone to zero-energy modes. For example, if a linear
quadrilateral element is estimated using only one integration point at the center of the element for in-plane deformation, then there will be no stiffness present to resist the shear mode which will cause no strain at the center; thus, the strain energy found at the center misses this mode of deformation and the energy of this mode tends to be over-estimated. These spurious modes of deformation, also known as “hourglass effects” pose the problem of lacking stiffness to resist certain “zero-energy” modes of deformation. These modes are oscillatory in nature and tend to have periods that are much shorter than those of the overall structural response. Hourglass modes must be effectively controlled or the deformations may grow large and produce an unrealistic geometry. Small damping is usually added into the system to avoid numerical problems. The effects of hourglassing can be seen in Figure 4.13.

![Figure 4.13: Undeformed mesh (left) and deformed mesh due to hourglassing (right).](image)

**Contacts**

When parts of common shared nodes are eroded from the simulation due to material failure, contacts between structural components need to be defined; otherwise, contacting parts that are not defined may intrude into the adjoining structural components without any counterforce. For example, high accelerations of the equivalent deck mass which rest upon the elastomeric bearings may intrude into the bent due to large deformations or erosion of the elastomeric bearings.

**4.5 GEOMETRICAL AND MATERIAL NONLINEARITY**

Typically, bridge columns produce larger axial capacity than axial demand and the axial load is usually ignored because the inclusion of axial loads usually will increase both shear and flexural resistance and improve performance. However, when the axial load is in excess of the balance point load
and/or second-order effects (i.e., P-Δ effects) are significant, the inclusion of axial load is crucial and necessary during blast load events. For finite element modeling of blast loads on bridges, geometric nonlinearity must be accounted for.

Material nonlinearity is associated with the inelastic behavior of components, such as spalling of concrete and large lateral deformation of reinforcement due to plastic strain accumulating.

From the combined effects of geometric and material nonlinearity, local and global failure modes are present in bridges subject to blast loads. Localized flexural or shear failure may result from close-in effects of an explosion. Shear failure may take place in the form of spalling and localized breaching of concrete. Even under minimum levels of blast load, local damage may lead the concrete core material into nonlinear region. The formation of a plastic hinge in the column leads to a larger energy absorption ability compared to compressive failure of concrete situated directly towards the blast wave. Thus, global modes of failure rather than local modes of failure of the structure may be considered for better designs of bridge components under blast loads.

4.6 DETERMINATION OF TIME STEP

Selection of the optimum time step is an important parameter for the simulation of blast load and material response. Smaller time steps typically lead to more accurate results than models with larger time steps, albeit while compromising computational efficiency [Knight et al., (2004)].

To ensure numerical stability during the simulation, LS-DYNA determines a critical time step based on the smallest node-to-node element length, \( L_e \), and is determined as follows:

\[
\Delta t_{min} = \frac{L_e}{c}
\]

where \( c \) is the instantaneous wave speed (speed of sound).
Blast loads will generally drive structural materials into the nonlinear range; thus, the determination of the critical time step size may vary using LS-DYNA’s default code. Numerical instability may arise if the smallest element used for the determination of the critical time step size is eroded due to failure of the material or if large deformations are present. It is desirable to produce a stable simulation without the influence of time step size. A controlled time step smaller than the critical time step is used in order to study the influence of material properties, geometry of structural components, etc. A controlled time step size of 1.00E-06 seconds has been used for the simulation of blast loading on bridge components to satisfy the critical time step for numerical stability and to capture more data points for better accuracy for the determination of peak blast load effects.

4.7 SIMULATION RUN-TIME

The total time required for simulation run-time can be quite extensive and is of importance. For this investigation, the simulation run-time of 0.3 seconds with over one-million-degrees-of-freedom takes approximately four days to complete on a computer with a 3.10 GHz processor and 4GB RAM. The main response of the structure is acquired during a simulation run-time of 0.3 seconds. Figure 4.14 provides an acceleration-time history of a point close to the explosion.

Figure 4.14: Nodal acceleration-time history curve.
Figure 4.14 indicates that the maximum response is obtained before 0.3 seconds. The structure begins to obtain its natural period around 0.25 seconds.

The increase in accuracy of the simulation is provided by dividing the structure into small, discrete elements. The downfall of the approach is that large amounts of time is spent on constructing the geometric model and inputting necessary parameters to describe the characteristics of the blast event. Even more time is needed to run the simulation to ensure that the results converge to a stable and accurate solution. Consequently, the calculation time to complete one simulation run may take nearly a three weeks.

Generally, total calculation time may be significantly reduced by taking advantage of geometric and load symmetry; however, since blast load is a highly nonlinear load and is not applied symmetrically about the structure, this advantage is not valid for the simulation. Another advantage that may reduce the total calculation time is by using a coarser mesh at locations where blast loads are less significant to the structural system being investigated, such as the footing of the bridge. Element eroding technique may be applied to elements which have reached their respective failure criterion.

**4.8 DETERMINATION OF MESH SIZE**

The influence of the mesh size on the simulation of elastic and inelastic response of bridge columns subjected to blast loads has been investigated through analytical and experimental literature. Yi (2009) simulated the response of a 3ft x 3ft reinforced concrete column with a height of 16 ft fixed against translation and rotation at the bottom of the column. A finite element model of the reinforced concrete column subjected to blast loading was simulated through LS-DYNA with various mesh sizes ranging from 1-10 inches. The results from Yi’s simulation were compared with experimental blast tests data on two reinforced concrete beams by Magnusson and Hallgren (2004). It was concluded that a mesh size ≤1.5 inches may be appropriate for the simulation.
CHAPTER 5. BLAST EFFECTS AND FAILURE MECHANISMS

5.1 BLAST-INDUCED FAILURE MECHANISMS

Four blast-induced failure mechanisms (denoted as F1 to F4) in the simulation of a hypothetical highway bridge designed to meet AASHTO (2010) seismic detailing for New York State, have been identified. We shall refer to this bridge as NY1. Seismic detailing for NY1 is presented in Figure 5.1.

![Figure 5.1: Seismic detailing for NY1.](image)

The required length of transverse reinforcement for NY1 shall satisfy Article 5.10.11.4.1c, as shown in Figure 5.1. Areas in zone (A.) are required zones for transverse reinforcement.

The following will present a discussion for each failure mechanism that may be present during the simulation of blast loading on bridge components.

**Failure Mechanism F1. Spalling of Concrete Cover:** Under blast loads, concrete cover may exhibit significant spalling.

As a shock wave propagates throughout the concrete column and the wave has reached the back-face of the column, a tensile wave is produced from wave reflections on the surface which leads to
spalling of the concrete column at and near the back-face of the pier column. Figure 5.2 shows spalling of the concrete cover located on the back-face of the column due to blast loading.

![Figure 5.2: Spalling of back-face concrete cover.](image)

**Failure Mechanism F2. Crushing of top bent concrete:** Concrete under support bearings may experience crushing of concrete due to the deck slamming atop the support bearings and the transfer of loads from the bearings to bent concrete. To minimize this kind of failure mechanism, the use of higher compressive strength concrete or improved detailing under bearings may be helpful. Concrete crushing of the bent below support bearings can be seen in Figure 5.3. Because the blast pressure was not applied to the deck there exist uncertainties of the severity for this failure mechanism.
As the deck load smashes against the bent at high accelerations due to the center column bending under high pressures, bent concrete beneath the support bearings is crushed. Exposed rebar can be seen in Figure 5.3.

**Failure Mechanism F3. Formation of Plastic Hinge:** The formation of a plastic hinge in the middle pier at the location of high blast loads is developed from plastic strain accumulating in the steel rebar. Consequently, total breach of the concrete core takes place from the formation of the plastic hinge, as shown in Figure 5.4.
Blasts testing on scaled models of reinforced concrete columns have been conducted by NCHRP (2010). From observation, their results show breaching of the concrete core and permanent deformation of steel rebar in one of the test columns, as shown in Figure 5.5.

Figure 5.4: Formation of plastic hinge in pier column.

Figure 5.5: Plastic hinge formation of test column subjected to blast loading.
**Failure Mechanism F4. Shearing of Bent:** Under high levels of blast loads, the bent may experience intense levels of shearing stress due to high inertial forces acting upon the bent from the weight of the deck which is initiated from column bending under the blast pressure. Shearing of the bent near column connections can be seen in Figure 5.6.

![Figure 5.6: Shearing of reinforced concrete bent.](image)

The equivalent deck system which rest atop the elastomeric bearing supports is free to move in the vertical direction. Consequently, under high blast loading crushing of concrete and shearing of the bent may occur. To prevent or mitigate these types of failure mechanisms is to restrain bridge girders, which rest upon the bearings, from vertical movement with typical fixed bearing connections.

**5.2 TOP PIER AND BENT ACCELERATION-TIME HISTORIES**

Time histories of the top center column and mid-section of the bent accelerations may be used to investigate the causes of certain failures, such as crushing of concrete under support bearings. Acceleration-time history curves from the simulation of NY1 subjected to various levels of blast loading, are shown in Figure 5.7 to 5.12.
Figure 5.7: Top pier acceleration-time history curve due to 500 lb-TNT.

Figure 5.8: Bent mid-section acceleration-time history curve due to 500 lb-TNT.
Figure 5.9: Top pier acceleration-time history curve due to 1000 lb-TNT.

Figure 5.10: Bent mid-section acceleration-time history curve due to 1000 lb-TNT.
Figure 5.11: Top pier acceleration-time history curve due to 2000 lb-TNT.

Figure 5.12: Bent mid-section acceleration-time history curve due to 2000 lb-TNT.

Examining the acceleration-time history for each level of blast loading for NY1, as shown in the above figures, indicates a longer duration of loading is applied for lower levels of loading. Columns subjected to larger amounts of blast loadings had reached their natural period sooner than lower levels of loading. Hence, shorter durations of loading produced higher shock impulses.
Figure 5.13 shows a time history curve of bent concrete underneath support bearings being crushed for NY1 subjected to 1000 lb-TNT.

![Vertical stress-time history curve of concrete under support bearing.](image)

From examining Figure 5.13 and Figure 5.10, crushing of the concrete underneath support bearings occurs almost instantaneously as the bent accelerates in the negative vertical direction.

The slamming of the deck onto the bent is initiated by the blast load producing a bending moment in the pier column. Bending at the top of pier column at a high acceleration initiated bending of the bent at nearly the same time. As a result, the bent deflected in the vertical direction at a high acceleration causing the deck to slam against the bent at high accelerations. This led to failure of concrete underneath support bearings. To mitigate this kind of damage, concrete with higher compressive strengths may be helpful or better detailing under support bearings.
CHAPTER 6. ENHANCED SEISMIC DESIGN FOR BRIDGES SUBJECT TO BLAST LOADS

6.1 APPLICATION OF SEISMIC-BLAST DETAILING

Enhanced seismic resistance may provide sufficient blast protection and may be applicable to mitigate progressive collapse of the structure due to blast loading. However, it would be misleading to say seismic design and detailing should provide adequate protection for reinforced concrete columns subjected to blast loads. An informal workshop supported by the General Services Administration (GSA) and the National Institute of Standards and Technology (NIST) discussed the applicability of seismic rehabilitation technologies to enhance the resistance of buildings to progressive collapse, recognized similarities and differences between seismic and blast loading, and provided examples of seismic strengthening technology applied to blast resistance.

This research investigates the effects of blast loads on a 3-span simply supported highway bridge that has been designed to meet seismic loads in New York using AASHTO specifications (2010). The scaled standoff distance, $Z$, is a function of the actual distance of the charge relative to the point of interest and the equivalent TNT weight. Therefore, the scaled standoff distance is an indication of blast load intensity. A larger scaled standoff, $Z$, denotes a smaller intensity of the blast loading. A range of the scaled standoff distance provides bases for the design of multi-hazard, seismic-blast detailing. For this investigation, seismic detailing for blast-induced columns depended on the value $Z$. From experimental data of scaled reinforced columns subjected to blast loads, the following conditions shall apply for the simulation of blast-induced bridge columns designed to satisfy NY seismic requirements referring to AASHTO specifications (2010):

1. For $Z > 1.25$
   - Design for Seismic Zones 1 and 2

2. For $1.25 \geq Z \geq 1$
Design for Seismic Zones 3 and 4; in addition, transverse reinforcement shall be provided throughout the whole length of the column.

3. For Z < 1

- Design for Seismic Zones 3 and 4; in addition, transverse reinforcement shall be provided throughout the whole length of the column and the total gross sectional area of transverse reinforcement required in Article 5.10.11.4.1d shall be increased by 50% for rectangular columns.

Highway bridge columns designed for Seismic Zones 1 and 2 are considered low intensity blast loading and should conform to the design and detailing provisions required by AASHTO LRFD Bridge Design Specifications (2010). Bridge columns in these zones shall satisfy Article 4.7.6 and detailing requirements of Article 5.10.12. Detailing requirements are less stringent and should achieve an acceptable performance under low levels of blast loading.

Bridge columns designed for Seismic Zones 3 and 4 where the scaled standoff distances are smaller will experience higher intensities of blast loadings. Transverse reinforcement should satisfy all seismic detailing for Seismic Zones 3 and 4 as specified in Articles 5.10.11.4.1c, 5.10.11.4.1d, and 5.10.11.4.1e for reinforced concrete columns. To account for potential plastic hinges formed during higher blast loadings, transverse reinforcement should be applied throughout the whole length of the column in these zones. These techniques allow for more energy dissipation and achieve a flexure failure mode. In addition, where the scaled standoff distance is less than 1 the total gross sectional area of transverse reinforcement required to satisfy Seismic Zones 3 and 4 shall be increased by 50%.

A high-fidelity, finite element 3-D model of the simplified bridge pier-bent system has been developed utilizing LS-DYNA in order to identify damage/failure mechanisms of bridge components.

Intended for the 3-span, simply supported highway bridge, a vehicle bomb is simulated under the mid span, 10 feet away from the middle column.
### 6.2 DESIGN OF BLAST-RESISTANT BRIDGES AND BLAST LOAD CASES

The hypothetical bridge, previously shown in Figure 4.2, is the intended 3-span, non-continuous bridge target. Parameters of the hypothetical bridge are displayed in Table 6.1.

<table>
<thead>
<tr>
<th>ITEM</th>
<th>VALUE</th>
</tr>
</thead>
<tbody>
<tr>
<td>Redundancy</td>
<td>Non-continuous</td>
</tr>
<tr>
<td>Length of Maximum Span</td>
<td>62 ft</td>
</tr>
<tr>
<td>Number of Spans in Main Unit</td>
<td>3</td>
</tr>
<tr>
<td>Design Load</td>
<td>MS 18 or HS 20</td>
</tr>
<tr>
<td>Deck Width</td>
<td>40 ft</td>
</tr>
<tr>
<td>Deck Thickness</td>
<td>13 in.</td>
</tr>
<tr>
<td>Lanes on Structure</td>
<td>2</td>
</tr>
<tr>
<td>Height of Pier</td>
<td>16 ft</td>
</tr>
<tr>
<td>Number of Piers</td>
<td>3 x (2 group) = 6</td>
</tr>
<tr>
<td>Pier Section</td>
<td>Rectangular 3.0 ft x 3.0 ft</td>
</tr>
<tr>
<td>Material / Design Type</td>
<td>RC concrete pier, bent &amp; deck, steel stringer</td>
</tr>
</tbody>
</table>

The hypothetical bridge is centered on an existing bridge located in New York State. It has been designed to satisfy all AASHTO bridge service loads and seismic detailing requirements in New York in accordance with AASHTO Article 5.10.11 [AASHTO (2010)]. Seismic capacity data of the bridge is listed in Table 6.2. The area of longitudinal reinforcement in reinforced concrete columns are the same for all seismic-blast design categories in order to study the relationship of transverse reinforcement and bridge performance. For ease of identification, we shall denote bridge designed for $Z > 1.25$ as NY1, $1.25 \geq Z > 1$ as NY2, and $Z < 1$ as NY3.

<table>
<thead>
<tr>
<th>Pier Parameter</th>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Design Load</td>
<td>Moment (kip-ft)</td>
<td>817</td>
</tr>
<tr>
<td></td>
<td>Shear (kips)</td>
<td>35.52</td>
</tr>
<tr>
<td></td>
<td>Axial Force (kips)</td>
<td>147.3</td>
</tr>
<tr>
<td>Elastic Capacity</td>
<td>Moment (kips-ft)</td>
<td>929.2</td>
</tr>
<tr>
<td></td>
<td>Shear (kips)</td>
<td>1345.7</td>
</tr>
<tr>
<td></td>
<td>Axial Force (kips)</td>
<td>6551.65</td>
</tr>
</tbody>
</table>
Seismic detailing for NY2 and NY3 are shown in Figure 6.1. The volumetric transverse reinforcements provided in Figure 6.1 should extend throughout the whole length of the column to satisfy stringent requirements to produce a flexure failure mode, which is most desirable.

![Concrete Strength 5000 psi](image)

Figure 6.1: NY2 detailing (left) and NY3 detailing (right).

Three levels of blast loads have been applied for each blast design category to examine seismic-blast detailing correlations. The amount of TNT-equivalent for the simulations of blast loads are 500, 1000, and 2000 lb-TNT. Location of the center of blast is 10 ft away from the middle pier column and measured 5 ft from the bottom of pier footing. A total of 9 cases of blast load simulations have been considered, as displayed in Table 6.3.

<table>
<thead>
<tr>
<th>Load Case</th>
<th>Bridge Identification</th>
<th>Level of Blast Load (lb-TNT)</th>
<th>$Z$ (ft/lb$^{1/3}$)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>NY1</td>
<td>500</td>
<td>1.26</td>
</tr>
<tr>
<td>2</td>
<td>NY1</td>
<td>1000</td>
<td>1.00</td>
</tr>
<tr>
<td>3</td>
<td>NY1</td>
<td>2000</td>
<td>0.79</td>
</tr>
<tr>
<td>4</td>
<td>NY2</td>
<td>500</td>
<td>1.26</td>
</tr>
<tr>
<td>5</td>
<td>NY2</td>
<td>1000</td>
<td>1.00</td>
</tr>
<tr>
<td>6</td>
<td>NY2</td>
<td>2000</td>
<td>0.79</td>
</tr>
<tr>
<td>7</td>
<td>NY3</td>
<td>500</td>
<td>1.26</td>
</tr>
<tr>
<td>8</td>
<td>NY3</td>
<td>1000</td>
<td>1.00</td>
</tr>
<tr>
<td>9</td>
<td>NY3</td>
<td>2000</td>
<td>0.79</td>
</tr>
</tbody>
</table>
6.3 OBSERVED FAILURE MECHANISMS SIMULATIONS

Blast load cases from Table 6.2 have been simulated and four failure mechanisms of structural members designed to satisfy AASHTO (2010) seismic requirements were encountered during blast loadings. The load cases are defined as unique scenarios, thus, all failure mechanisms, which are described in the proceeding chapter, may not be observed for each case. Different failure mechanisms are identified for each load case from the simulation, as shown in Figures 6.2 to 6.10.

Figure 6.2: Failure mechanisms of NY1 subjected to 500 lb-TNT.
Figure 6.3: Failure mechanisms of NY1 subjected to 1000 lb-TNT.

Figure 6.4: Failure mechanisms of NY1 subjected to 2000 lb-TNT.
Figure 6.5: Failure mechanisms of NY2 subjected to 500 lb-TNT.

Figure 6.6: Failure mechanisms of NY2 subjected to 1000 lb-TNT.
Figure 6.7: Failure mechanisms of NY2 subjected to 2000 lb-TNT.

Figure 6.8: Failure mechanisms of NY3 subjected to 500 lb-TNT.
Figure 6.9: Failure mechanisms of NY3 subjected to 1000 lb-TNT.

Figure 6.10: Failure mechanisms of NY3 subjected to 2000 lb-TNT.
6.4 SEISMIC-BLAST CORRELATION

From the following examination of bridge response for each load case, we can clearly view that decreasing the scaled standoff distance by applying a larger amount of TNT-equivalent had negatively increased failure mechanisms for each bridge design category. However, the number of failure mechanisms decreased with an increase of transverse reinforcement. Evidently, the increase of seismic capacity of reinforced columns provided better resistance against blast loads.

The degree of damage for each for each load case is a function of seismic detailing and the scaled standoff distance. Although, all levels of seismic detailing for blast resistance experienced spalling of concrete cover, the extent of spalling was less severe with lower levels of blast loading. Spalling of concrete cover is of minor issue and should not pose a serious hazard to the overall performance of the bridge.

Under low levels of blast loading, all bridges suffered eroding of concrete surface and crushing of concrete beneath support bearings, but were able to withstand the blast impact. Bridge columns designed for Seismic Zones 3 and 4 were able to withstand medium levels of blast loading. Under high levels of blast loading, NY1 and NY2 bridges suffered significant damage with crushing of core concrete and formation of plastic hinges in the column; in addition, propagation of cracks along the bent near column connections may lead to shear failure of the bent. Ultimately, bridges may collapse under these circumstances. Table 6.4 provides an outlook of failure mechanisms present for each load case under various levels of blast loading.

<table>
<thead>
<tr>
<th>Failure Mechanisms</th>
<th>Blast Load Levels (lb-TNT)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>NY1</td>
</tr>
<tr>
<td>Item</td>
<td></td>
</tr>
<tr>
<td>F1 Spalling of Pier</td>
<td>●</td>
</tr>
<tr>
<td>F2 Crushing of Bent</td>
<td>●</td>
</tr>
<tr>
<td>F3 Plastic Hinge</td>
<td></td>
</tr>
<tr>
<td>F4 Shear of Bent</td>
<td></td>
</tr>
</tbody>
</table>

● = Observed failure mechanisms during simulations.

Table 6.4: Seismic-Blast correlation for blast-resistant highway bridges.
CHAPTER 7 SUMMARY AND CONCLUSIONS

7.1 SUMMARY

The September 11 attacks on the Pentagon and World Trade Center has clearly illustrated the catastrophic damage to our civilian structures. The increase of terrorist attacks worldwide and the number of threats against our transportation infrastructure has heighten our concerns towards the infrastructural security. Transportation infrastructures are an attractive target for terrorist due to their accessibility and the overall impact it has on society. The Blue Ribbon Panel (2003) believes that a critical structure in the transportation system, such as a bridge or tunnel, could produce an economic loss exceeding $10 billion dollars. The NIST/GSA workshop on application of seismic rehabilitation to mitigate blast induced progressive collapse suggested the urgent national need to develop design standards for blast-resistant facilities. The focus of this research has been to investigate blast load effects on a highway bridge and extreme-hazard blast correlations. The objective of this investigation has been accomplished by simulating the blast load and structural response of a high-fidelity finite element model of a highway bridge, pier-bent system in LS-DYNA.

7.2 CONCLUSIONS

Important conclusions of the study are as follows:

- Increasing the scaled standoff distance significantly reduced the amount of damage to the structural system. Installing standoff barriers is a cost-effective approach for mitigating blast induced progressive collapse of the structure.
- In most cases, spalling of concrete is of minor issue and only led to local damage of the structure. Higher compressive strength concrete may be used to mitigate local damage. Service performance of the bridge may produce little to no effect.
• Increasing the area of transverse reinforcement improved the confined compressive strength of the concrete. In return, this improved the ductility of the system by allowing more energy dissipation through means of plastic strain accumulating in the steel.

• Enhancing the seismic capacity of bridge columns produced less failure mechanisms. Although, several failure mechanisms that are present in blast loads are not present in seismic conditions. Blast loading produces a higher impulsive load due to their short durations. Shearing at the footing are excessively larger than seismic conditions. Seismic activity may be predicted, whereas blast loading is unexpected. Breaching of core concrete and shearing of the bent are not experienced in seismic loading.

7.3 RECOMMENDATIONS

A comprehensive investigation has been carried to study the effects of blast loading on a highway bridge, pier-bent system. Some future needs in this area are considered in the following.

• The focus of this research was to investigate seismic-blast correlations of rectangular reinforced concrete columns. The geometry of the column can be of significance with higher levels of loading. In fact, experimental blast testing on rectangular and circular reinforced columns showed that circular columns experienced a 34% decrease in impulse [NCHRP (2010)]. There is also a need to study the performance of seismic-blast correlations for circular columns.

• Calibration of material properties may be improve to produce a more accurate simulation. Experimental blast testing on bridge components are scarce and the bases of material modeling for simulations are based on limited literature. There is a need for empirical data of blast effects on bridge components.
APPENDIX A

A.1 CONCRETE CONSTITUTIVE MODEL

MAT_159 (CSCM) is an elasto-plastic damage model with rate effects that was developed for concrete. It was originally developed for roadside safety applications and is applicable for other dynamic applications as well.

The required strengths, stiffness, hardening, softening, and rate effects parameters are functions of the concrete compressive strength and maximum aggregate size. The input parameter for normal strength concrete are valid for compressive strengths between 4061 psi (28 MPa) and 8412 psi (58 MPa).

Concrete is considered a hydrostatic-dependent material, therefore, the failure criterion was based on the compressive strength of concrete which is an indirect measurement of the unconfined tension strength. Typically, the unconfined tension strength is about 8 to 15 percent of the unconfined compressive strength.

The model has the ability to treat a compressive region between the failure surface and cap without numerical difficulties due to the continuous intersection between the failure surface and hardening cap. The yield surface of the model in Principal Stress Space is depicted in Figure A.1. Formulation of the yield surface shown in Figure A.1 mainly depends on the input value of the compressive strength and maximum aggregate size of concrete in LS-DYNA [Murray (2007)].
The formulation of the yield surface may follow the internal friction theory (Mohr-Coulomb Failure Criterion). The critical shearing stress is linear-related to the internal friction. In terms of principal stresses $\sigma_1$, $\sigma_2$, and $\sigma_3$ ($\sigma_1 > \sigma_2 > \sigma_3$) the linear relationship is as follows:

$$
\frac{\sigma_1 - \sigma_3}{2} = \frac{\sigma_1 + \sigma_3}{2} \sin \varphi + c_m \cos \varphi
$$

where $c_m$ and $\varphi$ are the cohesion and friction angle of the material, respectively.

Input parameters for well-confined concrete core for MAT_159 are listed in Table A.1.

Table A.1: User input parameters for well-confined concrete for MAT_159.

<table>
<thead>
<tr>
<th>Parameters</th>
<th>Description</th>
<th>Values</th>
<th>Units</th>
</tr>
</thead>
<tbody>
<tr>
<td>RO</td>
<td>Mass Density</td>
<td>2.280E-04</td>
<td>lbf-s$^2$/in$^4$</td>
</tr>
<tr>
<td>IRATE</td>
<td>Rate Effects</td>
<td>1</td>
<td>-</td>
</tr>
<tr>
<td>ERODE</td>
<td>Eroding Option</td>
<td>1.10</td>
<td>-</td>
</tr>
<tr>
<td>RECOV</td>
<td>Modulus Recovery</td>
<td>1</td>
<td>-</td>
</tr>
<tr>
<td>FPC</td>
<td>Unconfined Compressive Strength</td>
<td>6671.7</td>
<td>psi</td>
</tr>
<tr>
<td>DAGG</td>
<td>Maximum Aggregate Size</td>
<td>0.75</td>
<td>in.</td>
</tr>
</tbody>
</table>
A.2 STEEL CONSTITUTIVE MODEL

MAT_003 (Plastic-Kinematic) model has the ability to represent isotropic and kinematic hardening plasticity. This material model was used for steel rebars and ties. The initial yield strength of steel rebar is affected more than the ultimate yield strength under strain rates. Strains are more sensitive than the yield stress of steel; hence, strains are often easier to measure than stresses. Thus, the failure criterion was based on the maximum principal strain criterion (Saint-Venant Failure Criterion). Figure A.2 shows the Saint-Venant failure envelope.

![Figure A.2: Saint-Venant failure envelope on the meridian plane.](image)

The formulation of the yield surface for Figure A.2 can be defined in terms of the principal stresses $\sigma_1$, $\sigma_2$, and $\sigma_3$:

$$\sigma_1 - \nu(\sigma_2 + \sigma_3) \leq \sigma_{yp}$$  \hspace{1cm} A.2

where $\nu$ is Poisson’s ratio, and $\sigma_{yp}$ is the yield stress of the material.

Input parameters for MAT_003 are listed in Table A.2.
Table A.2: User input parameters for MAT_003.

<table>
<thead>
<tr>
<th>Parameters</th>
<th>Description</th>
<th>Values</th>
<th>Units</th>
</tr>
</thead>
<tbody>
<tr>
<td>RO</td>
<td>Mass Density</td>
<td>7.330E-004</td>
<td>lbf·s²/in⁴</td>
</tr>
<tr>
<td>E</td>
<td>Young’s Modulus</td>
<td>3.046E+007</td>
<td>psi</td>
</tr>
<tr>
<td>PR</td>
<td>Poisson’s Ratio</td>
<td>0.30</td>
<td>-</td>
</tr>
<tr>
<td>SIGY</td>
<td>Yield Stress</td>
<td>8.050E+004</td>
<td>psi</td>
</tr>
<tr>
<td>ETAN</td>
<td>Tangent Modulus</td>
<td>1.878E+005</td>
<td>psi</td>
</tr>
<tr>
<td>FS</td>
<td>Failure Strain</td>
<td>0.20</td>
<td>-</td>
</tr>
</tbody>
</table>

A.3 ELASTOMERIC BEARING CONSTITUTIVE MODEL

Elastomeric support bearings for the simulations were characterized by two separate parts consisting of purely elastomer properties and purely steel properties. Elastomers were modeled with MAT_006 (Visco-Elastic) and steel shims were modeled with MAT_003 (Plastic-Kinematic) constitutive properties. Input parameters for elastomeric bearing support are presented in Table A.3.

Table A.3: User input parameters for elastomeric bearing.

<table>
<thead>
<tr>
<th>Parameters</th>
<th>Description</th>
<th>Values</th>
<th>Units</th>
</tr>
</thead>
<tbody>
<tr>
<td>Elastomer</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>RO</td>
<td>Mass Density</td>
<td>1.100E-004</td>
<td>lbf·s²/in⁴</td>
</tr>
<tr>
<td>BULK</td>
<td>Elastic Bulk Modulus</td>
<td>1.205E+004</td>
<td>psi</td>
</tr>
<tr>
<td>GO</td>
<td>Short-time Shear Modulus</td>
<td>86.0</td>
<td>psi</td>
</tr>
<tr>
<td>GI</td>
<td>Long-time Shear Modulus</td>
<td>78.0</td>
<td>psi</td>
</tr>
<tr>
<td>BETA</td>
<td>Decay Constant</td>
<td>0.070</td>
<td>-</td>
</tr>
<tr>
<td>Steel Shims</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>RO</td>
<td>Mass Density</td>
<td>0.20</td>
<td>lbf·s²/in⁴</td>
</tr>
<tr>
<td>SIGY</td>
<td>Yield Stress</td>
<td>4.00E+004</td>
<td>psi</td>
</tr>
<tr>
<td>E</td>
<td>Young’s Modulus</td>
<td>3.05E+007</td>
<td>psi</td>
</tr>
<tr>
<td>FS</td>
<td>Failure Strain</td>
<td>0.20</td>
<td>-</td>
</tr>
</tbody>
</table>
The long-time shear modulus can be considered a Dirac delta function with an impulse occurring at time $t = \tau$. The shear relaxation behavior can be described as:

$$G(t) = G_\infty + (G_0 - G_\infty)e^{-\beta t} \quad \text{A.3}$$

A Jaumann rate formulation is used from a Dirac delta integral as:

$$\nabla \sigma'_{ij} = 2 \int_0^t G(t - \tau) D'_{ij}(\tau) d\tau \quad \text{A.4}$$

where $\nabla \sigma'_{ij}$ denotes the deviatoric part of the stress rate, $\nabla \sigma_{ij}$, and the strain rate, $D_{ij}$. 
REFERENCE


Jenkins, B. M., and Gersten, L. N. (2001). *Protecting Public Surface Transportation against Terrorism and Serious Crime*, Mineta Transportation Institute, San Jose, California.


SAIC. (2006). “BlastX Version 6.4.2.2” Science Applications International Corporation, US Army Engineer Research and Development Center, Geotechnical and Structures Laboratory and Air Force Research Laboratory, MNAL.


