Design of Bridges for Security Against Terrorist Attacks

by

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Design of Bridges for Security Against Terrorist Attacks

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Dedication

Dedicated to my loving parents Marcia and Duane Gannon.

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The University of Texas at Austin, 2004
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Since the unfortunate events of September 11th, the United States must focus additional attention on determining vulnerabilities to terrorist attack. One such vulnerability that has received consideration is the threat of terrorist action against a transportation target. This report summarizes results of research to investigate cost effective measures to improve bridge security against a terrorist threat. It discusses previous research performed on risk management and threat assessment, and discusses the dynamics of extreme loadings on structures. It also discusses the analysis methods and results of parameter studies used to determine cost effective bridge retrofit or design change options for improved security. This research provides a guideline for a bridge engineer to create a bridge design to protect against terrorist blast loads.

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CHAPTER 1

Introduction

1.1 BACKGROUND AND SIGNIFICANCE OF WORK

The fundamental landscape of world culture has gone through a tremendous change within the course of a very short time. Since the unfortunate events of September 11, 2001, countries around the world have become more aware of potential vulnerabilities within their borders. Things once believed to be safe may now be considered to be at risk. Acts of terrorism have been carried out against the United States on targets both within our borders and abroad, and it is crucial to investigate and work towards solutions to the problems that terrorism brings with it. According to the Center for Defense Information "Terrorism Project" website (Center for Defense Information, 2002), "Terrorists seek to weaken a hated political authority that is responsible (in their eyes) for illegitimate policies." The website describes the goals of terrorism in more detail with the statement, "By their attacks, terrorists seek to prove that the political authority they target:

- Cannot protect its own population
- Cannot protect the symbols of its authority
- Cannot protect society's institutions
- Cannot protect society's infrastructure
- Cannot protect its own officials
- Cannot end the threat of more terrorism
- Cannot maintain normal, peaceful conditions in society"

When viewing these goals and taking into account past instances of terrorism against the infrastructure of the United States and other countries, it becomes clear that there is a need to investigate the potential risk of attack to transportation systems and to devise methods of protection against these risks. The Center for Defense Information "Terrorism Project" website also contains a discussion of the ease, and consequences, of committing terrorist acts against transportation infrastructure. According to the website,

> Transportation networks are prime targets. For example, pipe bombs and other explosive devices placed in culverts long have been used by many groups to blow up passing vehicles. But the roads themselves could be targets. For example, on any given day, cars, presumably with mechanical or electrical failures, are parked along interstates and ring roads that encompass large metropolitan areas. Such vehicles are often left empty as the owners seek help or, in extreme weather, temporarily abandon them. The "normalcy" of these occurrences could easily mask an explosive-laden vehicle. Damage would be increased if a bridge or abutment were involved. Indeed, some highway interchanges are so complex (e.g., the series of "fly-overs" in south Houston) that even the threat of such terrorist action would temporarily immobilize major transportation. Moreover, the effect of such threats would be compounded in cities with tunnels. Also, highway bridges over rail lines provide an opportunity for a well-timed "accident" to drop a vehicle onto the rails.

The main points of this discussion demonstrate the abilities of terrorist attackers to achieve their goals of disrupting the function of society through damaging transportation infrastructure, and also make note of the seemingly simple manner in which these goals could be achieved. The main focus of this research is to formulate guidelines that can be used by engineers designing components of the transportation infrastructure to mitigate the risk of catastrophic failure under the extreme circumstances presented by terrorist attacks. Specifically, this research focuses on measures that can be taken to protect bridges and their supporting substructures against varying degrees of attack. In addition, it proposes acceptable levels of damage which correlate to structural importance and severity of attack.

The significance of this particular focus on bridge security is appropriate given several recent events such as the threats made against four of the state of California's suspension bridges, and the validation of these threats by the videotape showing detailed shots of the Golden Gate Bridge (CNN, 2002) captured from Al Qaeda members in Spain. CNN has also published reports about a man arrested for his role in conspiring to attack and destroy New York's Brooklyn Bridge (CNN, 2003). The report discusses Al Qaeda plans to use cutting torches and other tools to sever several of the suspension bridge's cables. Clearly, such an attack and subsequent damage to these bridges is consistent with typical terrorist goals of disrupting society and its infrastructure, as well as demonstrating that government cannot protect its population and national symbols. Also, in a 1997 report, Brian Jenkins (Jenkins, 1997) describes attacks to over 550 transportation targets worldwide and makes a statement regarding an increase in attacks against public transportation. As is seen in Figure 1.1 below, 6% of these 550 attacks were directed towards bridges.

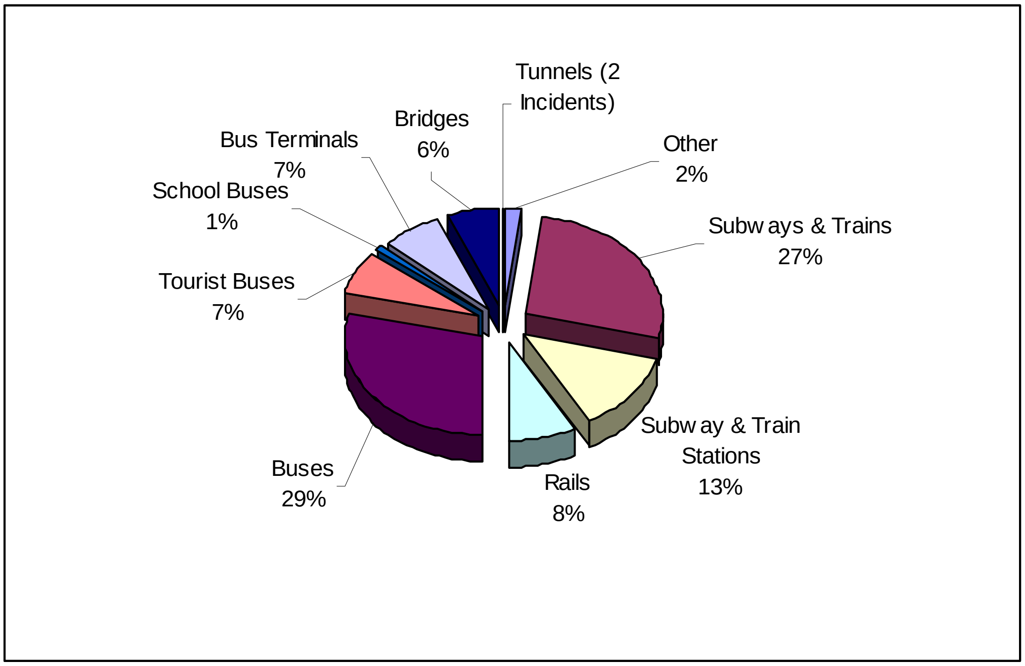


Figure 1.1 Terrorist Attacks Against Transportation Systems (Jenkins 1997)

With the ever increasing risk of terrorist attack and the clear potential for transportation systems to be targets, it is essential to evaluate measures to reduce vulnerabilities. This research will provide guidelines useful to engineers and risk managers to improve security and performance of critical bridges against terrorist attack.

1.2 PROJECT FOCUS

This research, including previous work, will help government officials and transportation engineers work to identify threats and assess vulnerabilities, and to take actions to reduce vulnerabilities in a cost-effective manner. Phase I of this research has already been completed, and this report will focus on work carried out under Phase II. The Phase I research report provides a literature review with information about transportation security, terrorist threat potential, and mitigation techniques (Winget, 2003). Also included in the Phase I report is information about criticality, risk assessment, risk management, and bridge modeling. Phase II of the research work will focus on identification of cost-effective measures of improving bridge performance under blast loading either through retrofit of existing structures or design changes in new structures as a means of reducing vulnerability. Certainly, it is not practical, or perhaps even possible, to protect a structure against any attack of unknown type or magnitude; however it is possible to limit risk and to manage an attack of a reasonable degree. The overall goal of this research is to provide solutions to limit the risk of terrorist attacks against critical bridges to an acceptable level in an efficient and cost-effective manner.

To fully understand the range of threat scenarios, bridge importance, and expected performance levels, it is important to be familiar with previous project research performed by Captain David Winget (Winget, 2003). Chapter 2 of this document provides a summary of some important concepts presented in “Design of Critical Bridges for Security against Terrorist Attacks” (Winget, 2003). The first portion of the discussion focuses on risk assessment, including asset criticality, threat scenarios, and attack consequences. The next portion discusses risk management and the concept that countermeasures can be implemented that mitigate risk with or without structural retrofits.

In addition to a required understanding of the risk of possible threats and their consequences, it is important to understand how blast loads affect structural behavior so that measures can be taken to improve performance as necessary. Accordingly, Chapter 3 of this report describes the dynamics of blast loads and provides an explanation of various alternative modeling approaches that can be used to predict structural response.

After describing the nature of blast loadings and modeling alternatives, Chapter 4 describes the parameter studies and modeling concepts for bridge girder systems. This chapter includes discussion of performance-based standards applied to bridge superstructures as well as a discussion of modeling changes made during the research project. The next chapter, Chapter 7, presents the findings of the girder parameter studies. Evaluation of the results in this chapter includes explanation of the cost-benefit analysis of girder retrofit options. Recommendations to designers for mitigation of blast loads to girder bridges are also provided.

Chapter 6 contains a discussion of structural analysis, performance, and risk assessment of truss bridges subjected to terrorist attack. This chapter demonstrates an analysis approach used to determine the ability of trusses to redistribute internal loading following the failure of one or more critical members. It includes a comparison of different truss geometries and identifies characteristics important to withstanding member loss. Also included is a discussion of possible causes of initial member failures, progressive collapse significance and causes, and recommendations to mitigate risk of terrorist attack to truss bridges. Chapter 7 is the final chapter of this report; it provides a summary of conclusions reached for all bridge types, recommendations of further research, and recommendations to designers. Additional consideration of bridge substructure systems is included within the appendixes of this report.

CHAPTER 2

Risk Management Procedures and Techniques

2.1 Overview

Once the importance of the design of bridges for security is established, it becomes important to define clearly the problem parameters that need to be addressed. To effectively design a bridge to resist an attack, it is necessary to define the possibilities for type of attack, the importance of a structure which may be attacked, and the risk that such an attack will occur. In "Design of Critical Bridges for Security against Terrorist Attack" (Winget, 2003), a procedure is outlined for threat, risk, and criticality assessment. This chapter provides a summary of the information presented in that report which serves as a starting point for research of terrorist threats to bridges.

2.1.1 Threat Definition

In order to provide a set of guidelines for designing or retrofitting bridges against terrorist attack, it is essential to define the nature of the potential attack scenarios that must be resisted. These numerous scenarios make it unfeasible to design a structure to withstand all possible combinations. Understanding terrorists' goals and tactics is essential for determining the most likely modes of attack, and viewing the problem from this perspective forms the basis of a threat point-of-view analysis. This method selects the most likely terrorist courses of action for the basis of design.

As mentioned above, terrorist goals will play a critical role in determining appropriate scenarios for design. Goals most often encountered include making a high visibility statement, destroying a landmark or critical asset, exerting political pressure, creating public fear and panic, maximizing casualties, disrupting the economy, and interrupting main or emergency transportation routes (National Academy of Sciences, 1995). When considering bridge security and terrorist action against transportation systems, goals would likely be destruction of high profile bridges or bridges critical to emergency and general transportation. Ideally for terrorists, an attack plan will be realistic, coordinated, cohesive, simple, creative, flexible and secretive (Department of Justice, 2002). Each of these elements increases the likelihood of a successful attack which will achieve the overall goals of the terrorist action. Typically, it would be expected that terrorists would use crude explosives in vehicle-delivered scenarios, or small amounts of tactically located hand-placed explosives. These hand-placed explosives could be very effective in a variety cases; however, many critical locations are difficult to access, thereby reducing the speed, simplicity, and flexibility of using this form of attack. As such, when evaluated in light of the criteria likely to be used by terrorists to plan an attack, the hand-placed explosive scenario, though perhaps more effective in destroying a key bridge component, could rank lower in the overall attack plan due to other constraints.

2.1.2 Risk Assessment

To develop a bridge security plan, there must be a definition of unacceptable risk to provide information on what a structure must be designed to resist. In "Design of Critical Bridges for Security against Terrorist Attack" (Winget, 2003), risk assessment procedures from several sources were combined and tailored specifically for bridges. The purpose of these risk assessment procedures is to answer the questions: (1) What can go wrong? (2) What is the likelihood that it would go wrong? (3) What are the consequences? (Haimes, 2001). A modified version of the U.S. DOT's vulnerability assessment provides an effective framework for assessing threats to bridges (Abramson, 1999). A four-step process was constructed based on that framework. These steps, explained in some detail below, are as follows:

- Identify Critical Assets
- Identify Threats to Critical Bridges
- Formulate Threat Scenarios
- Assess the Consequences of an Attack

2.1.2.1 Identify Critical Assets

The first step, involving identification of critical assets, requires investigation into many factors related to bridge importance. Examples of these factors would be average daily traffic, access to populated areas, access to important facilities for emergency or military purposes, symbolic significance, and detour availability. The criticality assessment procedure requires creation of categories for each of these factors, and assigning a score based on the importance of a bridge relating to that factor. Each criterion must also be assigned a weighting factor to account for the relative importance to the others. This system allows for a score to be computed for each bridge and provides a method to rank their relative criticality.

An example of a system like the one described above comes from TxDOT's database and is called the *Texas Bridge Criticality Formula*. This database accounts for the categories listed earlier as well as site-specific information including lack of capacity of available detours, access to schools and hospitals, utilities across a bridge, location near hazardous facilities, and importance to hurricane evacuation routes. One missing element of this database is symbolic importance. It has been mentioned many times previously in this report and in other literature that this issue is an important factor in terrorist goals.

Nevertheless, development of some system ranking bridge criticality is necessary to carry out a risk assessment. The basic procedure for critical asset identification is demonstrated below in Table 2.1.

Table 2.1 Bridge Criticality Determination

Bridge No.	Emergency Importance	Symbolic Importance	Average Daily Traffic	Criticality Score (Weighted Average)
	Criticality Weighting Factor			
	.25	.25	.5	
1	3/5	1/5	3/5	2.5/5
2	2/5	2/5	4/5	3/5
3	4/5	2/5	3/5	3/5

2.1.2.2 Identify Threats to Critical Bridges

Identification of threats specific to each critical asset is essential in investigating plausible terrorist actions against a bridge. To narrow the large number of unpredictable terrorist actions that must be considered, a technique was developed based on a simplified version of the Military Decision Making Process (Department of the Army, 1997). This method uses a threat point-of-view analysis considering terrorist potential objectives and resources to determine most-likely courses of action. This procedure uses a ranking system based on brainstorming of feasible terrorist courses of action and assigning weighted criteria to the terrorists' decision making process. For example, a threat point-of-view analysis of a bridge traversing a waterway would consider motor-vehicle-delivered explosives on the bridge deck, including one or more small scale explosions, ramming the bridge support structure with a maritime vessel, hand-placed or vehicle-delivered explosives at an approach structure, or a combination of explosives and vessel collision (Abramson, 1999). This analysis would consider the likelihood of success of each attack scenario, and also the ability of such an attack to achieve the terrorist goals of disruption and destruction of the targeted infrastructure.

2.1.2.3 Formulate Threat Scenarios

This step is the combination of information developed in asset and threat identification. By using particular knowledge, including vulnerabilities, of each critical bridge and all likely threat scenarios, formulation of specific courses of action can take place. This step can be generalized by bridge type, with additional consideration given to the most critical bridges, to develop a plan to mitigate risks to a larger number of bridges by using standardized countermeasures for different categories of structures (e.g., plate girder bridges with moderate criticality can all use a standardized set of countermeasures).

2.1.2.4 Assess the Consequences of an Attack

In order to fully asses risk to critical assets, the consequences of a terrorist attack must be considered. Potential consequences include loss of life, injuries, loss of bridge service due to structural damage, financial costs of repairs or replacement, effect on the transportation system, and financial impact to the surrounding area. The high cost associated with disruption of a transportation system can be seen when considering the recent collision of a truck with a bridge in Connecticut (CBS, 2004) on I-95. A fire caused by the fuel oil carried by the truck created enormous deflections of a bridge span requiring replacement of the supporting girders. Reconstruction of the damaged bridge portion took place at a fast pace but still required several days to complete. Direct costs needed for cleanup, traffic control, erection of a temporary bridge, and construction of a new bridge were estimated at over $11 million (CBS, 2004). Accounting for the indirect costs of delay, impacts on nearby businesses, etc., the total costs associated with such an event can be enormous. Though this event was an accident, it gives an indication of the extent of potential costs and consequences associated with an intentional act of violence carried out by terrorists.

As a means to simplify the process of assessing the consequences of an attack, and making note of the relationship to many of the potential consequences with criteria considered in step 1, this step in the risk assessment process can be performed in conjunction with criticality assessment. This assumption is a reasonable one because of the previously stated terrorist goals of attracting attention and maximizing damage. This observation allows that the same parameters used to determine bridge criticality can be used in relation to consequences of attack. The completion of these steps provides the necessary information for formulating potential attack scenarios which can be used to develop design criteria. A summary of different threats and impact of the attack on a bridge of a defined criticality is shown in Table 2.2. This table can be used to organize the consequences of each different threat.

Table 2.2 Threat Scenario Categories

Threat Scenario Categories		Severity of Impact			
		Catastrophic (Criticality > 75)	Very Serious (Criticality 51 -75)	Moderately Serious (Criticality 26 -50)	Not Serious (Criticality < 25)
Probability of Successful Occurrence	Highly Probable	Severe	Severe	High	Moderate
	Moderately Probable	Severe	High	Moderate	Low
	Slightly Probable	High	Moderate	Low	Low
	Improbable	Moderate	Low	Low	Low

2.1.3 Risk Management

After assessing potential risks, vulnerabilities, and consequences, risk management can be performed. Risk management involves using information provided about critical assets and risk assessment to take action to mitigate the possibility of these risks to a structure. The risk management process should answer the following questions: (1) What can be done and what options are available? (2) What are the associated trade-offs (costs, benefits, risks)? (3) What are the impacts of current management decisions on future options (Haimes, 2001)? Applying these questions specifically to terrorist risk management of bridges involves investigation of retrofit options, non-structural mitigation methods, evaluation of current design practices, and comparisons to associated costs. A five-step process is outlined to manage risks to specific bridges or bridge types using cost-effective countermeasures (Winget, 2003). A list of these steps is shown below, and a description of each step is provided in the next five subsections.

- Identify Countermeasures
- Determine Countermeasure Cost
- Cost-Benefit Analysis
- Implement Countermeasures and Reassess Risks
- Monitor Effectiveness

2.1.3.1 Identify Countermeasures

Identification of available countermeasures is a critical first step in risk management. It involves consideration of measures to provide deterrence, detection, and defense. In the case of bridge security, examples of countermeasures would be increased security by personnel or use of closed-circuit television for monitoring activities on a bridge, increased standoffs to bridge components, or structural hardening of the bridge itself. Also included in this step of the risk management process is the screening of countermeasures to ensure feasibility of use in regard to issues such as resources, convenience, and ease of implementation.

2.1.3.2 Determine Countermeasure Cost

This step is used to provide information for a cost-benefit analysis of countermeasure alternatives. Costs associated with purchase, installation, maintenance, and replacement of each countermeasure should be considered.

2.1.3.3 Cost-Benefit Analysis

A cost-benefit analysis is important as a means of assessing the relative worth of each potential countermeasure. Results of an analysis of this type provide a method of selecting the most effective countermeasures. It is recommended that this analysis be performed based on the amount of risk mitigation achieved by each countermeasure. This procedure makes a connection between each countermeasure and the cost savings provided by deterrence or reduction in the severity of impact of an event. It is important to evaluate all potential benefits of a particular countermeasure, including other threats that would be part of a complete risk analysis. For example, improved lighting on a bridge may increase the effectiveness of remote bridge monitoring, but it also is effective in improving driving conditions and overall safety. Strengthening of piers will lead to better behavior under potential blast load scenarios, and it will also improve performance in the case of a vehicle impact. The overall goal of a cost-benefit analysis is to provide information to ensure resource allocation in the most efficient and effective ways possible. The information collected by a cost-benefit analysis can be assembled into a countermeasure summary sheet. This sheet is an effective way to organize information for the purpose of selecting countermeasures to implement. It is important to note that a cost-benefit analysis should consider not only initial costs, but also long term expenses such as operating and maintenance expenditures. Importance of the consideration of all associated costs can be demonstrated in the case of the use of closed-circuit television monitoring. This threat mitigation option has a relatively low initial cost, however the long-term expense of monitoring and maintenance may make it a less efficient use of resources than structural hardening. An example of a Countermeasure Summary Sheet (SAIC, 2002) can be seen in Table 2.3.

Table 2.3 Countermeasure Summary Sheet

Countermeasure	Function / Effectiveness				Costs per year		
	Deterrence	Detect	Defend	Reduce Impact	Capital	Operating	Maintenance
Countermeasure 1	M	L	L		$	$	$
Countermeasure 2	M	H			$	$	$
Countermeasure 3				H	$	$	$
Countermeasure 4	L		H		$	$	$
L = Low Effectiveness M = Medium Effectiveness H = High Effectiveness							

Source: Modified from SAIC "A Guide to Highway Vulnerability Assessment for Critical Asset Identification and Protection."

2.1.3.4 Implement Countermeasures and Reassess Risks

Implementation of the countermeasures deemed appropriate through cost-benefit analysis is performed to attempt to mitigate risk to a structure. It is important to reevaluate risk after a countermeasure is in place. This reevaluation will allow for determination of countermeasure effectiveness, and need for additional action if necessary. It should be noted that no countermeasure will completely eliminate risks to bridges; however, it must be reduced to a level accepted by the risk manager.

2.1.3.5 Monitor Effectiveness

The selected countermeasures must be monitored for effectiveness. This monitoring includes investigation into appropriateness of use of an effective countermeasure in similar situations. The purpose of this step in the risk management process is to ensure that resources are allocated appropriately at the present time, and to provide information for future countermeasure use.

2.1.4 Process Significance

In order to improve performance of a bridge against a terrorist attack, a significant amount of initial research must be performed to provide information about threat definition, criticality assessment, risk assessment, and risk management. The entire risk assessment and management process is diagramed conveniently in Figure 2.1 below (Winget, 2003).

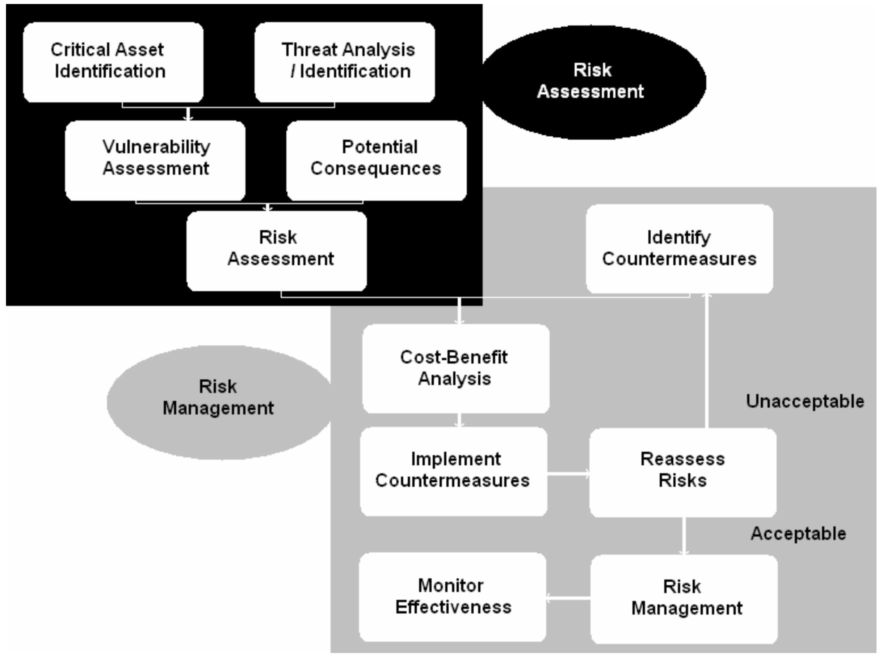


Figure 2.1 Risk Assessment and Management Processes **(Winget 2003)**

These concepts are fundamental to all portions of this research. They provide the foundation for threat and load definition, countermeasure evaluation, and performance-based standard development and implementation. Concepts from this chapter will be revisited in later portions of this report. For further information regarding these topics, refer to "Design of Critical Bridges for Security against Terrorist Attack" (Winget, 2003).

CHAPTER 3

Modeling, Dynamics, & Blast Loads

3.1 SIGNIFICANCE

Investigation into the consequences of terrorist threats to bridges requires an understanding of the properties of both the bridges under attack and the attack itself. It is necessary to consider alternative methods of modeling a bridge system, and to understand the characteristics of the loading to which that system is to be subjected. The subsequent sections of this chapter provide an explanation of common modeling approaches, their applicability to bridges under blast loading, general dynamics principles, and properties of blast loads.

3.2 BLAST LOAD CHARACTERISTICS & DYNAMICS

Prior to discussion of the analysis method selected, it is necessary to understand the characteristics of blast loads a structural system may face. Blast load properties and their dynamic nature must be considered in order to determine the most appropriate modeling approach because of their effect on the structural system response.

3.2.1 Blast Description

Utilizing information gathered on explosions through research that took place shortly after World War II, the characteristics of blast loads are readily described (Biggs, 1964). There are several valuable resources on this topic such as the Department of the Army TM 5-1300 (Department of the Army, 1990), Explosive Loading on Engineering Structures (Bulson, 1997). The textbook

“Structural Dynamics” (Biggs, 1964) collects this information and provides a description of the nature of a blast from a surface burst that takes place at or near the ground surface. Figure 3.1 illustrates the definition of a surface burst as it applies to this research.

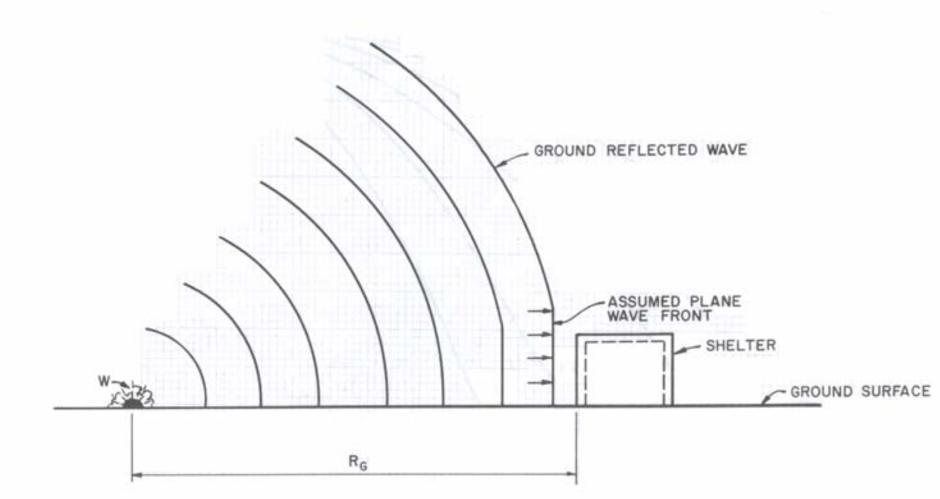


Figure 3.1 Illustration of an Unconfined Surface Burst (Department of the Army, 1990)

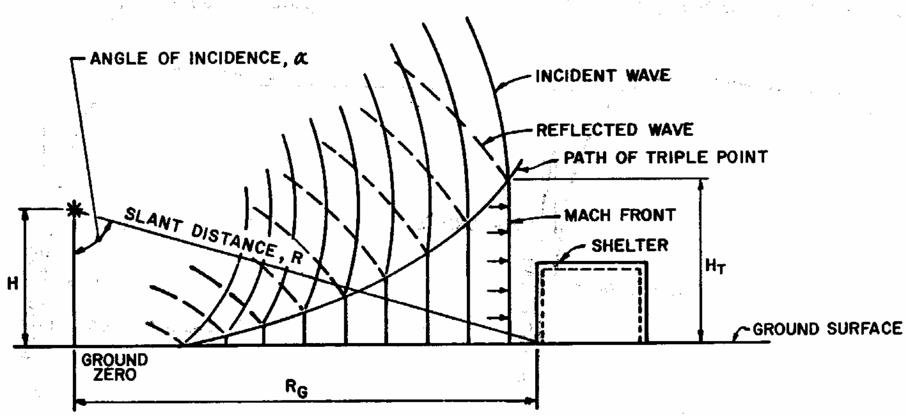


Figure 3.2 Illustration of an Unconfined Air Burst

The textbook makes note of the complexity of bursts away from the ground surface. The concepts of wave reflections and pressure front merging are more complicated with blasts acting at a distance from a reflecting surface as can be seen in Figure 3.2. Several resources are available to aid in the understanding of blast loads and the associated loadings on structures. Some of the most useful resources are the Department of the Army TM 5-1300 (Department of the Army, 1990), Explosive Loading on Engineering Structures (Bulson, 1997) and Structural Dynamics: Theory and Applications (Tedesco, 1999).

For this research, it is necessary to consider both surface bursts and airbursts depending upon the threat scenario and location of an explosive relative to components of a bridge. Biggs (1964) states that an explosion will cause a circular shock front to be propagated away from the point of burst. This shock front will travel away from the blast location with a certain velocity and peak pressure. The pressure will then decay behind this pressure front. The relationship between the overpressure and the radial distance from the point of burst can be seen in Figure 3.3.

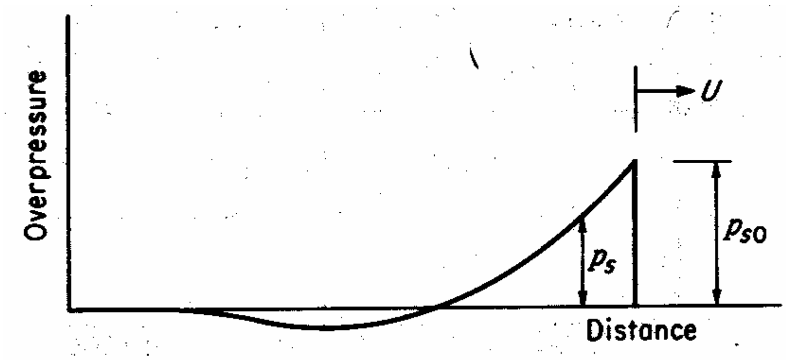


Figure 3.3 Blast Overpressure-Distance Relationship (Biggs, 1964)

A blast is considered to have three components: (1) the initial diffraction loading, (2) the general overpressure effect, and (3) the drag loading (Biggs, 1964). The shock front striking a surface, such as a building or bridge, causes a diffraction effect resulting in higher pressures due to the reflection of the wave on the front face of the object and the time lag before the overpressure acts on the object's rear face. Next, the object is subjected to the general overpressure, and finally, the "wind" created by the high velocity shock front produces a drag force on the object. The relationships between both overpressure p_s and dynamic pressure p_d (the pressure created by the velocity of the moving air particles) with time at some location for a blast are shown in Figure 3.4 (Biggs, 1964).

The dynamic pressure p_d is calculated by ½ ρv^2, where ρ is the air density and v is the velocity of the air particles. The dynamic pressure creates drag forces on an object, and these drag forces can be computed using an appropriate drag coefficient C_d and the dynamic pressure. The total loads acting on a surface include contributions from both the dynamic pressure and the drag forces. A diagram showing the load history for a blast load acting against a rectangular object is shown in Figure 3.5 (Biggs, 1964). This diagram shows the pressure-time history of a blast, and includes the effects of overpressure and dynamic pressure. The quantity p_r shown in Figure 3.5 is the total reflected pressure. It includes the amplification effect of the overpressure caused in part by the formation of a reflected wave acting on the object.

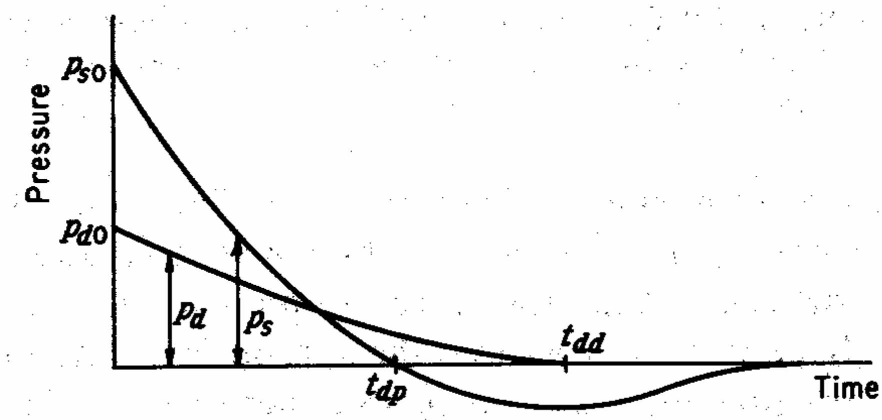


Figure 3.4 Blast Pressure-Time Relationship (Biggs 1964)

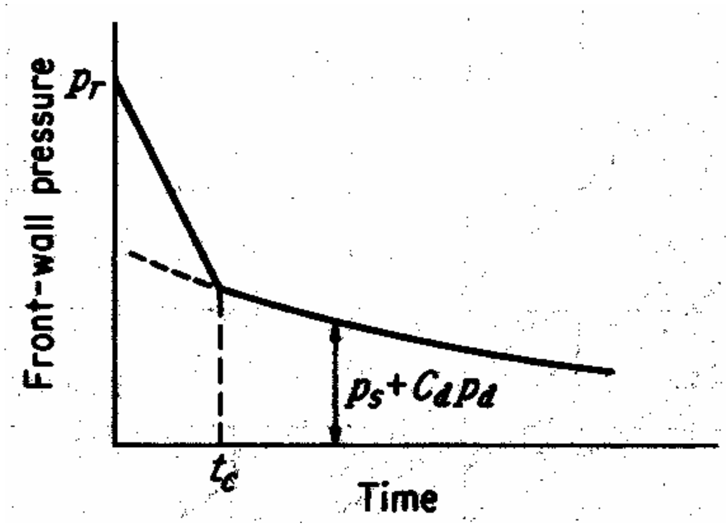


Figure 3.5 Pressure Pulse for a Rectangular Object (Biggs, 1964)

3.2.2 Blast Property Scaling

Useful relationships for both the range and pressure loading duration as a function of explosive size have been developed (Biggs, 1964). Range is defined as the distance between an explosion and a target, and the term yield is often used to identify explosive weight. These relationships, shown in Equations 3.1 and 3.2, illustrate the importance of standoff in reducing effective yield of a blast, and the variation in impulse created by explosives of different yield. Impulse is a critical parameter in dynamic analyses. It is defined as the area beneath a load-time curve, and is important to the response of a dynamically loaded system. For loads with an extremely short duration in relation to the natural period of the object under load, the actual shape of the load-time curve may not be as important as the total impulse (Paz, 1997).

$$R_1 / R_2 = \sqrt[3]{Y_1 / Y_2} \tag{3.1}$$

$$t_{d1} / t_{d2} = \sqrt[3]{Y_1 / Y_2} \tag{3.2}$$

Because Equations 3.1 and 3.2 are expressed as ratios, the units of each entity must only be consistent within each fraction. It is typical, however, to define the yield of an explosive in terms of an equivalent weight of TNT. Based on pressure and impulse, amounts of different types of explosive material can be scaled to an equivalent weight of TNT, which is the standard to which all other explosive materials are compared (Department of the Army, 1990). The available conversion allows for relative comparisons of different explosive types. This issue is important because of the unknown nature of a terrorist threat. Observations of recent terrorist attacks demonstrate it has been common to use ammonium nitrate and fuel oil mixtures (ANFO) (Ettouney, 2002). This observation, combined with knowledge of the payload capacity of various trucks and other vehicles, allows for a definition of a likely threat as described in Chapter 2 of this report (Conrath, 1999). It should be noted that a threat with a magnitude of 4000 pounds of TNT-equivalent explosives is approximately the same amount of ANFO used in the Oklahoma City bombing of the Murrah Building. Shown below in Figure 3.6 (Hinman, 1997) is an illustration of important parameters that define an explosive threat for the purpose of design or analysis.

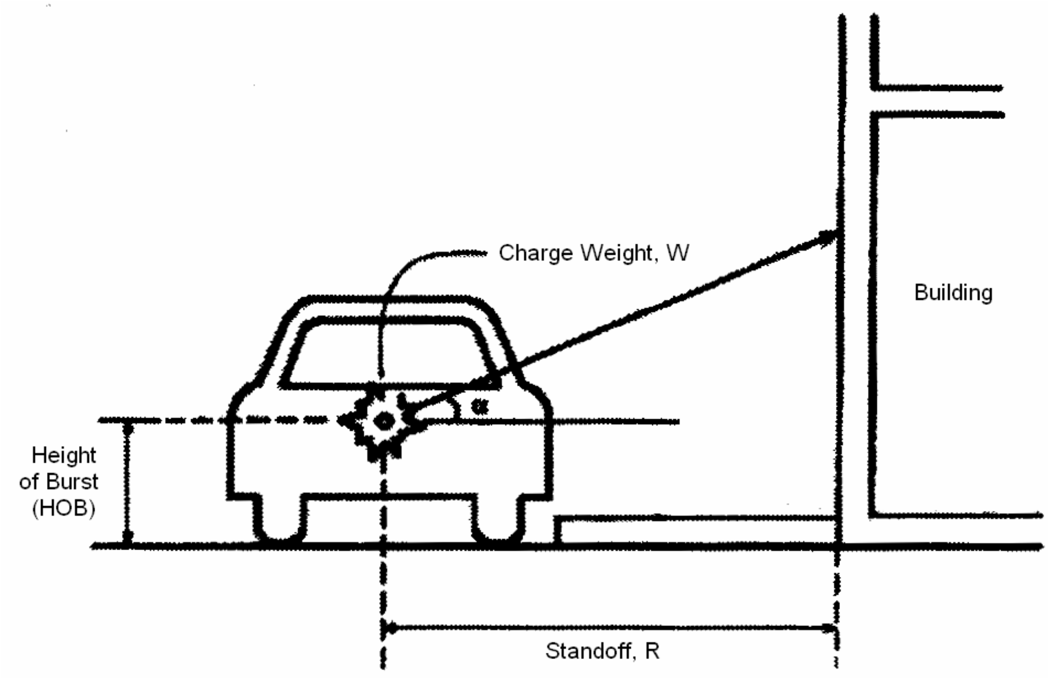


Figure 3.6 Threat Definition Parameters (Hinman, 1997)

3.2.3 Strain Rate Effects

An important topic to consider when investigating the effects of blast loadings on a structure is the effect of strain rate. Because structural materials cannot deform as quickly as the applied blast loads act, both concrete and metals achieve strength increases (although not necessarily the same increases) based on loading rate. Accounting for this factor is essential when choosing a method of structural analysis for applications involving blast loads and impact. A variety of methods can be used to deal with strain rate effects on material properties. It is possible to use simplified dynamic increase factors or material models that actually account for the strain rate influences to compute allowable material stresses. A specific discussion of the techniques employed in treating these strength increases in the structural models used in this research is included in later chapters when describing the details of each model. General effects of strain rate on concrete (Figure 3.7) and various metals (Figure 3.8) (Tedesco, 1999) are shown below.

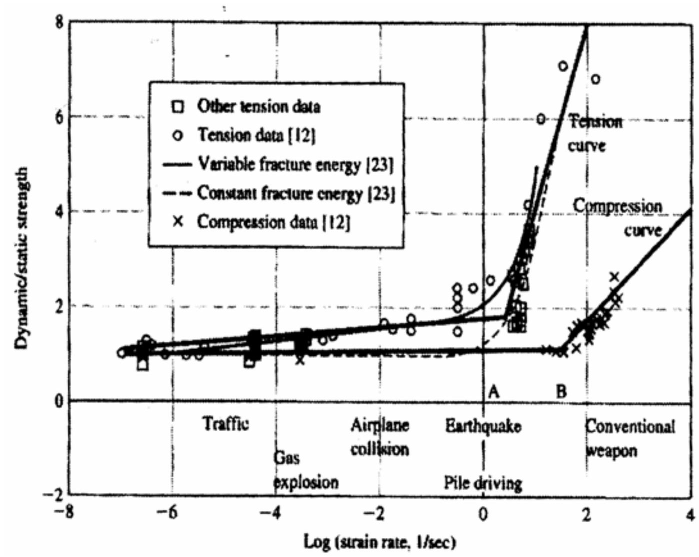


Figure 3.7 Concrete Strain Rate Influence on Strength (Tedesco, 1999)

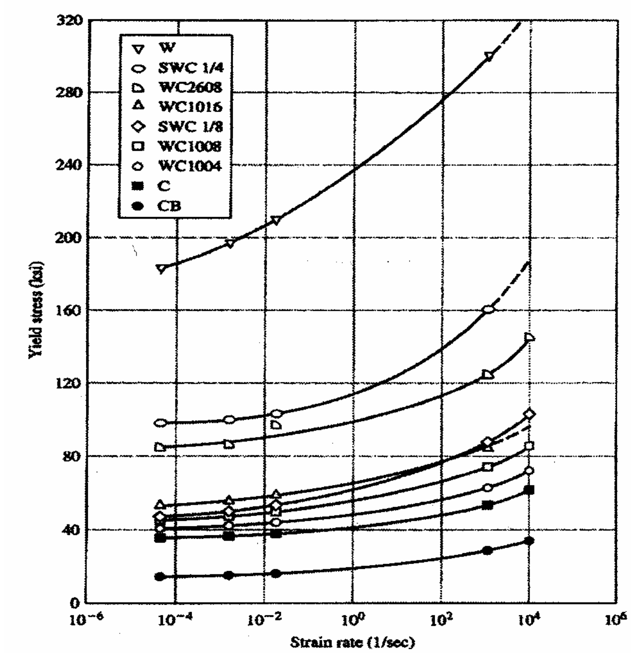


Figure 3.8 Strain Rate Influence on Yield Stress of Various Metals (Tedesco, 1999)

3.3 MODELING ALTERNATIVES

There are a wide variety of alternatives for modeling a structural system, each with advantages and limitations specific to the definition of the system and its corresponding loading. The accuracy and simplicity of analysis will vary with each modeling approach. A modeling technique may consider coupling of structural response and loading, may be static or dynamic in nature, and may include one or more degrees-of-freedom. These alternative methods and their typical relative accuracies are shown below in Figure 3.9 (Winget, 2003).

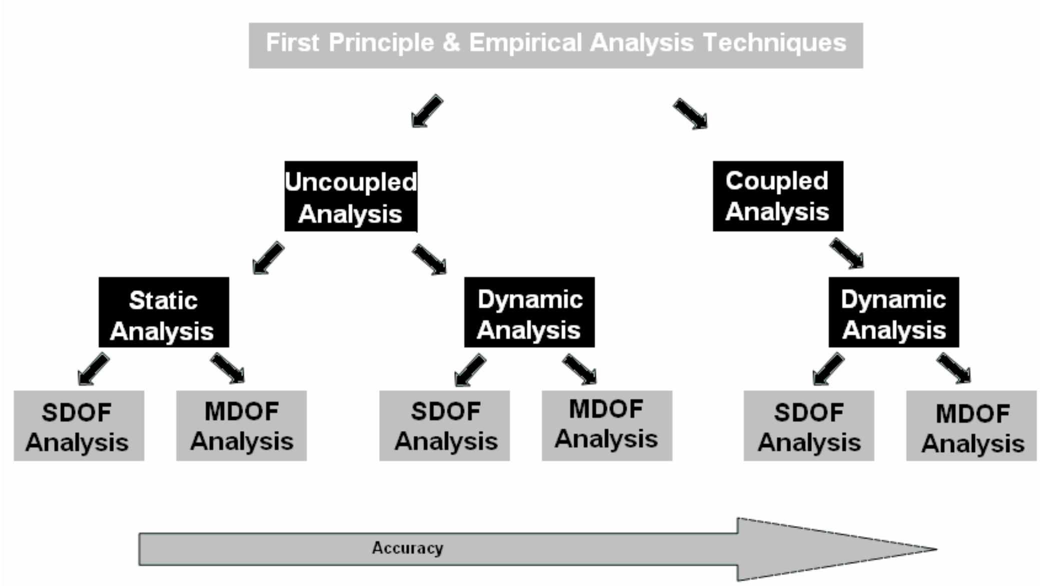


Figure 3.9 Summary of Analysis Methods (Winget, 2003)

Selection of the most appropriate analysis method for a specific application should, at a minimum, consider computational resources, accuracy provided by each technique specific to the situation in question, and required accuracy. In the case of a bridge subjected to a dynamic blast loading, a coupled multiple degree-of-freedom dynamic analysis would provide the greatest accuracy. However, a complicated model may require significant computational resources, and therefore may not be the most efficient method to investigate blast effects on a wide range of bridge types and parameters. Alternatively, an uncoupled static single degree-of-freedom analysis allows for a large number of analyses needed to carry out a study of the effectiveness of varying certain bridge properties, though this analysis method cannot accurately account for the dynamic nature of blast loadings and bridge system response.

As discussed previously in Chapter 1, "Introduction," the purpose of this research is to provide guidelines to engineers for the design or retrofit of bridges to resist terrorist attacks. The strengthening of these bridges will limit damage from blast loadings or vehicle impacts caused by terrorists, and will also improve bridge response to incidents such as accidental explosions or unintentional vehicle impacts. To provide these recommendations, information must be gathered on the most effective methods of mitigating such events. This information can be collected by evaluating the relative structural response of bridges with specific retrofits or combinations of retrofits under blast loadings. In order to enable a large number of variations in bridge design or retrofit to be investigated, it is necessary to utilize an analysis approach that is simple enough to be easily repeated numerous times while still providing sufficient accuracy to represent valid predictions of response. An important point to note is that although reasonable accuracy is required in computing the response of each structure to blast loadings, the most important feature of the analyses is providing a relative comparison of the benefit of each individual or combination of retrofits and design changes. In addition, due to the large degree of potential variability in magnitude and position of the applied blast loads, detailed analyses are not warranted.

3.4 SIMPLIFIED DYNAMIC MODELING APPROACH

Selection of an analysis method appropriate for the purposes of this research requires consideration of the complex dynamic nature of blast loads, the coupling of local and global response, required accuracy, and available computational resources. As discussed previously, a simple static approach is likely too simple to capture the behavior of a bridge system under complex loading, and a sophisticated coupled nonlinear dynamic analysis which accounts for material and geometric nonlinearity and the interaction of the load with the dynamic system could provide the greatest accuracy but with a large amount of required resources. When considering the unknown nature of a terrorist threat, and the associated blast loading for any specific threat, as well as the need for only a relative comparison of retrofit effectiveness, it is reasonable to consider a simplified approach that captures as accurately as possible the response of a large number of retrofit options. To this end, it is appropriate that models developed for the examination of bridge components should include sets of single degree-of-freedom (SDOF) systems analyzed to compute dynamic response. In addition to the appropriate compromise between accuracy and simplicity, this approach represents the state of practice for blast design. Similar blast analysis approaches can be found in the army manual TM 5-1300 (Department of the Army, 1990), or other blast design references. Discussions of the detailed models for each specific component examined are provided in later sections of this report. A general description, however, of the chosen approach is discussed in the following subsections.

3.4.1 Simplified Dynamic Approach & Model Parameter Explanation

The simplified dynamic models utilized in this research are intended to calculate the dynamic response of individual structural components subjected to blast loads. Using available software developed by the Army Corps of Engineers such as CONWEP (USACE, 2003) or BlastX (USACE, 2003) (this software is only for authorized users and is not widely available), a blast load for a specific element can be determined as a function of time. CONWEP software utilizes well-known formulas to describe variation in blast pressure and impulse as a function of time, and BlastX develops a complex environment in which wave reflection within a vented room is used to calculate pressure and impulse histories. More detailed descriptions of the capabilities of these software packages are provided later in this report. The blast loading is then applied to a structural component modeled as a single degree-of-freedom system. This system consists of a mass and a mass-less spring. The stiffnesses and internal resistance limits for this spring correspond to information obtained from the component being modeled. There are several simplifying assumptions that can be made with regard to determining these system properties. For this research, the calculation of these parameters is performed by assuming that the element being considered displaces in the static displaced shape $\Phi(x)$ along its length (L) corresponding to a static load (F) of the same type and shape as the dynamic load to which the component is subjected. The maximum deflection, $\Phi(L/2)$, is defined as Δ. The stiffness (k) is provided by Equation 3.3. The relationship between stiffness of a component and the applied force is shown graphically in Figure 3.10. The model used in this research considers the formation of plastic hinges in beam section which reach their plastic moment capacity. The formation of plastic hinges within a beam is analogous to a change in boundary or release conditions, and therefore will necessitate the use of a different displaced shape. For this reason a beam, and accordingly its equivalent single degree-of-freedom model, will have multiple stiffnesses corresponding to each portion of its deformation history. Each stiffness is calculated using the procedure outlined above, incorporating the displaced shape consistent with the conditions created by plastic hinge formation. The use of these incremental stiffnesses is an important feature of this research as it allows for a more accurate consideration of the actual deformation history. Because of the large displacements which occur due to blast loads this feature of the analysis is essential.

$$k = F/\Delta \tag{3.3}$$

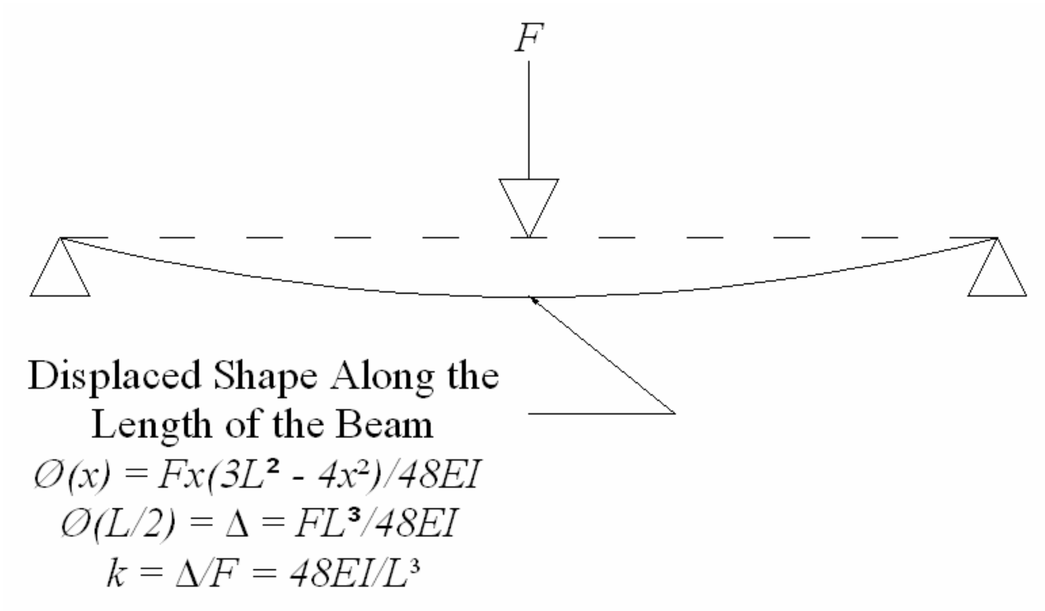


Figure 3.10 Force-Displacement Relationship for a Beam

The internal resistance limit for each stiffness is determined by calculating the ratio of applied load which causes a change in stiffness. This change in stiffness occurs due to the formation of one or more plastic hinges, which causes a change in the assumed static displaced shape. Internal resistance and stiffness are related by Equation 3.3. It is possible for a system to allow for the formation of multiple plastic hinges at different quantities of load, and therefore a structural component will have multiple stiffnesses. Once a component has formed a sufficient number of plastic hinges to create a mechanism (a system that will continue to deform without an increase in applied load) the stiffness is zero. The stages of deformation for a beam are shown below in Figure 3.11 to demonstrate the calculation of each stiffness and resistance limit.

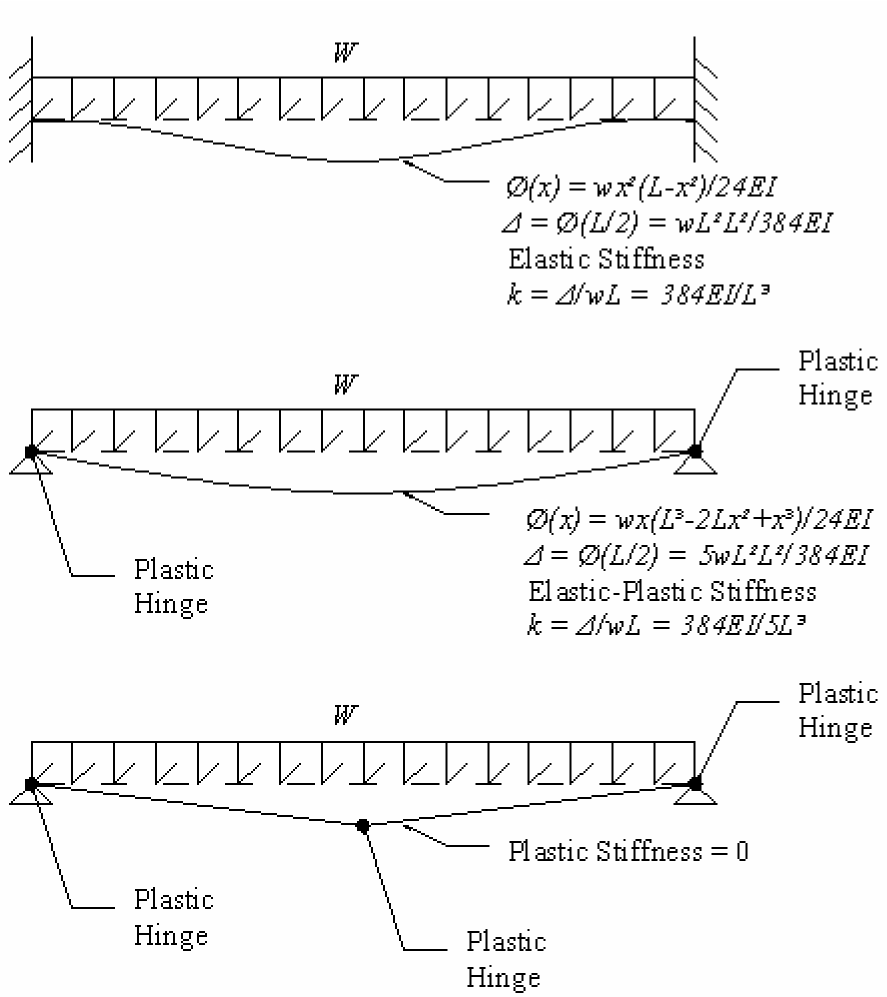


Figure 3.11 Force-Deformation History of a Beam

An example of the calculation procedure for these system properties is included in Appendix C, and an illustration of their typical relationship with each other is shown in Figure 3.12.

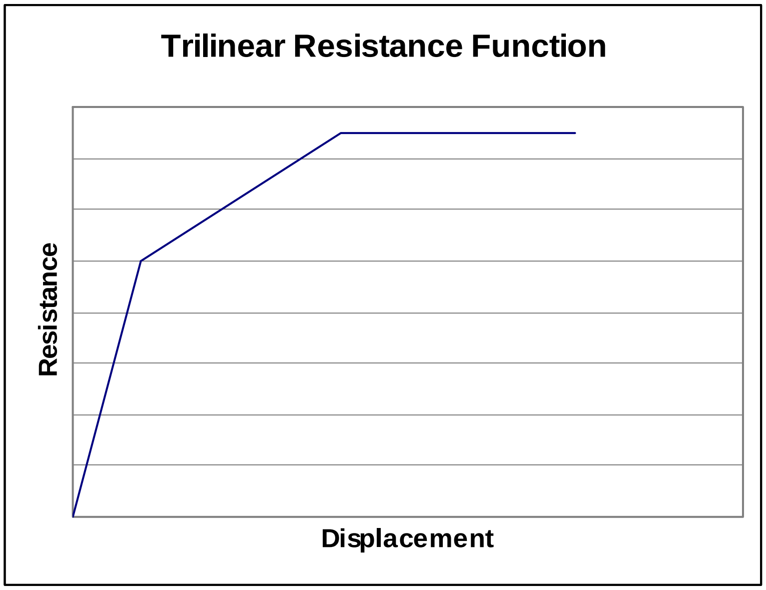


Figure 3.12 Trilinear Resistance Function

3.4.2 Transformation Factors

Calculation of the system parameters as described in the previous section represents an idealized approach to analysis that is based on the assumption that the deformation of any component subjected to a dynamic load can be accurately described using the displaced shape that would result under a statically applied load of the same form. The actual displaced shape of a component subjected to dynamic loads, in general, will be a combination of the different modes of vibration based on the natural frequencies of the component. Utilizing an SDOF approximation is reasonable when the dynamic response of a component is dominated by a single mode, and such is the case for the systems being considered in this research. In order to complete the conversion between the actual structural element's characteristics and those of the idealized system, transformation factors must be computed for, and applied to, the actual mass and dynamic load so that equivalent properties for the idealized SDOF system can be determined. These transformation factors are based on equating the work performed by each system. Calculation of the mass transformation factor, M_f, for a system with evenly distributed mass can be accomplished using Equation 3.4 shown below, where m is the system mass per unit length, L is the length of the element being considered, and $\Phi(x)$ is the element's assumed displaced shape normalized such that the peak deflection is one. As discussed above, the displaced shape can be assumed to be any number of different functions. For this research, however, it is assumed to be the static displaced shape of the component under investigation. Use of this displaced shape is consistent with the recommendation in the textbook "Structural Dynamics" (Biggs, 1964). Shown in Figure 3.13 is an example of a beam element under a uniform load, and the corresponding static displaced shape used to characterize the deformation for dynamic analysis. Thus, the dynamic response involves determining the amplitude of displacement in this shape.

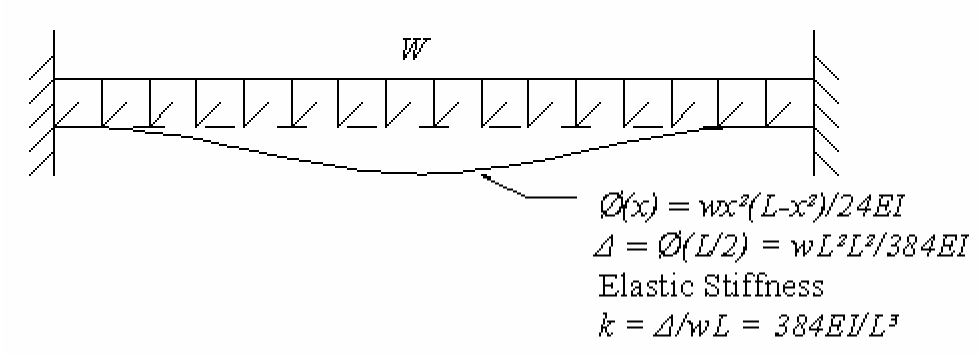


Figure 3.13 Determination of Assumed Displaced Shape

$$M_e = \int m \cdot \phi(x)^2 \tag{3.4}$$

Calculation of the load factor, L_f, is highly dependant on the type and shape of the loading. In the case of a point loads, the load factor would be calculated by Equation 3.5, where F_r is the magnitude of each point load and Φ_r is the magnitude of the displaced shape at the location of each load, and r is the total number of applied point loads. Again the displaced shape must be normalized such that the peak displacement value is one.

$$L_f = \frac{\sum_r F_r \cdot \phi_r}{\sum_r F_r} \tag{3.5}$$

In the case of a uniformly distributed load, the load factor would be calculated by Equation 3.6 where p is the magnitude of the distributed load, L is the length over which the load acts, and $\Phi(x)$ is the displaced shape normalized in the same manner as previously discussed.

$$L_f = \frac{\int_L p \cdot \phi(x)dx}{pL} \tag{3.6}$$

3.4.3 Analysis Procedure

The previous sections of this report demonstrate the methods used to determine the required system properties and transformation factors for a beam modeled as a single degree-of-freedom system. This information is then used to calculate the dynamic response of the system subjected to a dynamic blast load.

Calculation of dynamic response of a structural system is possible through the solution of the differential equation that describes dynamic equilibrium. The typical form of this differential equation is shown below in Equation 3.7. In this equation, M_f is the mass factor, m is the system mass, u is the system displacement, t is time, k is the system stiffness, L_f is the load factor, and F is the applied force.

$$M_f \cdot m \cdot \ddot{u}(t) + k \cdot u(t) = L_f \cdot F(t) \qquad (3.7)$$

Due to the complex nature of blast loads and the difficulty in describing such loads conveniently as a mathematical function, a closed-form solution to Equation 3.6 would be difficult or impossible to obtain. Therefore, a time-stepping approach based on a numerical solution to Equation 3.7 is used to determine the dynamic response of the model SDOF system as a function of time. For this research, Newmark's method was selected. Newmark's method is a commonly used numerical method which uses an assumption about system acceleration to project dynamic response further in time. Acceleration can be assumed to be constant or linear through the use of different coefficients within the method. For the purpose of this research, acceleration was assumed to vary linearly over a given time step. Additional information about the Newmark Beta method or other numerical approximation methods can be found in a variety of dynamics textbooks (see, for example, Paz, 1997).

3.4.4 Summary

The previous sections discuss the method of analysis, supporting reasons for method selection, calculation of required parameters, and simplifying assumptions made for bridge modeling. A detailed description of the specific component models and assumptions is provided in subsequent chapters as well as in the appendices referenced previously.

CHAPTER 4

Superstructure Modeling & Analysis

4.1 SIGNIFICANCE

As discussed in previous chapters, bridge substructures are critical structural components that must be protected against terrorist attacks. Of course, superstructure response to such events must also be considered. This chapter focuses on the modeling and analysis of steel girder/concrete deck systems. The information presented is strongly related to material presented in another report accompanying this research dealing with prestressed girder system analysis. As discussed in “Design of Critical Bridges for Security against Terrorist Attack” (Winget, 2003), the most likely terrorist courses of action on girder bridges are above- or below-deck explosions near the supports or near midspan. Blasts in these locations will likely be caused by vehicle-delivered explosives, and analysis of girders under this type of loading is therefore the focus of this chapter. The intent of the research on this subject is to determine methods of mitigating risk to girder bridges, and to prevent significant damage to, or loss of, one or more girder spans.

4.2 PROBLEM DEFINITION

Damage to a girder subjected to an above- or below-deck explosion will be caused by overloading from either flexure or shear, or may arise due to flange local buckling. These modes of response occur because of the blast pressures acting both along the length and across the width of a deck and girder system. The orientation of these forces relative to a girder system can be seen in Figures 4.1 and 4.2.

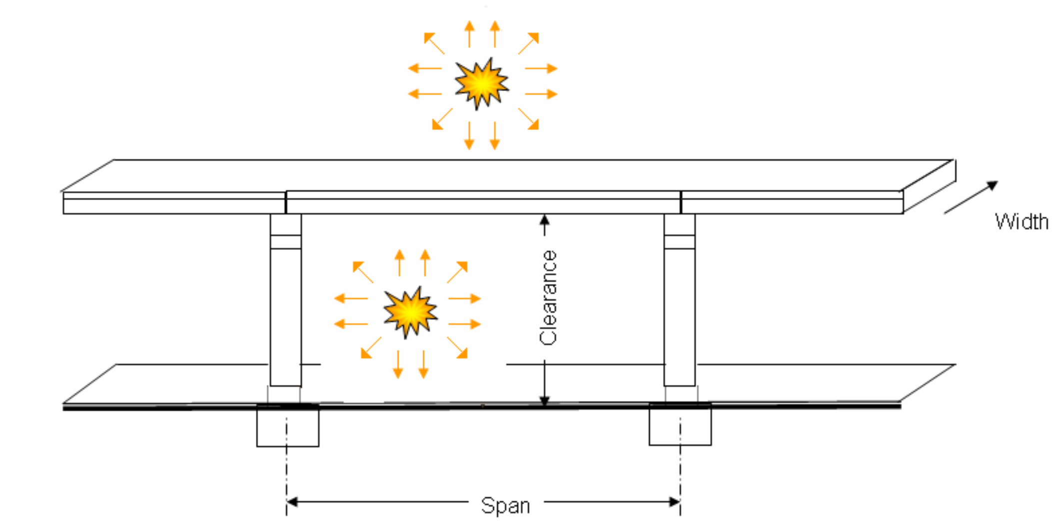


Figure 4.1 Typical Blast Location Relative to a Girder System

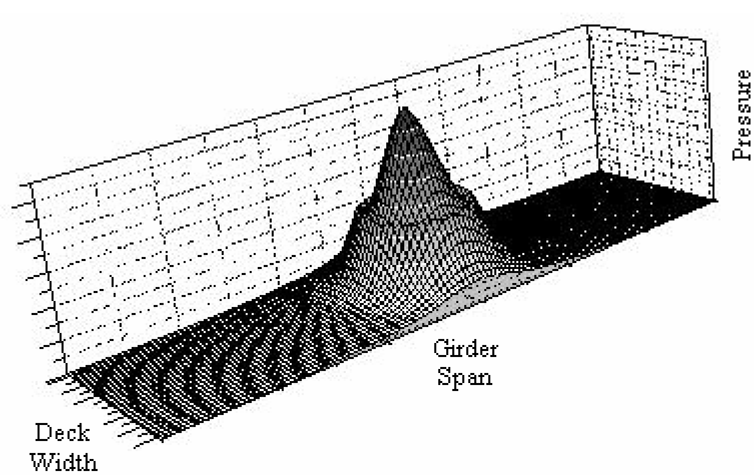


Figure 4.2 Typical Pressure Distribution Over a Girder System

The analysis of steel girder systems subjected to blast loadings of an unknown magnitude at an unknown location along the length of a girder span is highly sensitive to the assumptions made to simplify the analyses. For reasons described in previous chapters relating to performance-based standards, and in an effort to maintain consistency throughout different analyses, blast magnitudes studied are the same as those used for substructure analysis. Those blast magnitudes are derived based on the typical explosive type used by terrorists, and the available capacity of vehicles used to deliver these explosives. In addition to assumptions about load magnitude, there are many other aspects of the modeling procedure that have a significant impact on analysis results. The type of analysis used is a major factor in the type of result obtained. Just as with pier systems, a wide variety of analysis options exist, ranging from simple single-degree-of-freedom static analyses to complicated coupled multiple-degree-of-freedom dynamic analyses. As discussed in previous chapters, it is important to consider issues such as problem definition, required accuracy, available computational resources, and the intended use of the results when selecting an analysis approach. In addition to these considerations, consistency with analysis methods employed on other bridge components must be taken into account. The fact that a large number of analyses are required to provide a relative comparison of design and retrofit effectiveness in improving structural performance, the unknown nature of the load, and the desired consistency with pier analyses again suggest use of a simplified single-degree-of-freedom dynamic analysis approach. The degree of damage to a girder system in flexure will be based on ductility limits obtained from previous research characterizing the response of structural components subjected to blast loads (USACE, 2004). Provided values were modified through coordination with the Texas Department of Transportation, and through discussion with Project Supervisor Dr. Eric Williamson, Project Advisor Kirk Marchand and researcher David Winget. Additional discussion of damage estimation and other similar topics is provided later in this chapter in the section discussing performance-based standards for girder systems.

4.3 ANALYSIS VARIATIONS CONSIDERED

Throughout the duration of this research, several variations of a single-degree-of-freedom dynamic analysis were investigated. These variations include changes to load type used, composite deck action considerations, and use of an approach analyzing a single girder or an entire deck system. The following subsections describe the final method of analysis used, a discussion of other methods considered, and reasoning for the use of the selected modeling approach.

4.3.1 Development of the Load Path Approach

A critical assumption to be made with regard to girder analysis relates to the treatment of the concrete deck. For both above- and below-deck blast scenarios, it is possible to consider the deck acting either entirely composite with the supporting girders, entirely non-composite, or with some portion of the deck acting to increase the moment capacity and stiffness of the supporting girders. This issue is an important aspect to consider because, as demonstrated in previous chapters, the strength and stiffness of a component are critical to determining properties of an equivalent single-degree-of-freedom system used for analysis. To investigate the effects of these assumptions, the dynamic response examination of a single girder with a varying amount of strength increase from deck composite action can be considered. Shown below in Table 4.1 are the midspan displacements calculated for a large girder subjected to the same blast load with different thicknesses of deck assumed to be contributing to the single-degree-of-freedom strength and stiffness. The table contains results for two different blast magnitudes, each applied to the same girder configurations.

Table 4.1 Amount of Composite Deck Action Effects on Girder Response

Blast Type	"Mid-Size" Explosive at 12 ft Standoff		
Composite Deck Amount	0 %	50 %	100 %
Girder Midspan Displacement as a Percentage of the Span Divided by 2	14.1%	9.8%	7.6%
Blast Type	"Large" Explosive at 12 ft Standoff		
Composite Deck Amount	0 %	50 %	100 %
Girder Midspan Displacement as a Percentage of the Span Divided by 2	44.6%	33.2%	25.5%

It is clear from the large variation in calculated centerline girder midspan displacements that the assumed amount of composite action of the deck is critical in dynamic response calculations. In order to eliminate the large variations seen in Table 4.1, consideration of a different analysis method is necessary. An alternative to considering the deck and girders to act as one single-degree-of-freedom system is to consider the deck as one single-degree-of-freedom system, and the girders below as another. In effect, this modeling approach requires determining the dynamic response of the deck system, and using information obtained from those analyses to calculate the dynamic structural response of the girder system. Because the analysis approach follows the load along a path from acting on the deck system to causing reactions on the girder system, in this report this technique is referred to as the ‘load path approach’. This load path approach does not use one multiple-degree-of-freedom or one single-degree-of-freedom system, but rather a series of single-degree-of-freedom models. A diagram showing the concept of the load path approach is shown in Figure 4.3.

The most significant advantage to the use of this load path approach is the ability to account for changes in load applied to a girder based on failure of portions of the deck. More details about the application of this method are provided later in this chapter after discussion of the significance of the variation of the load over the deck.

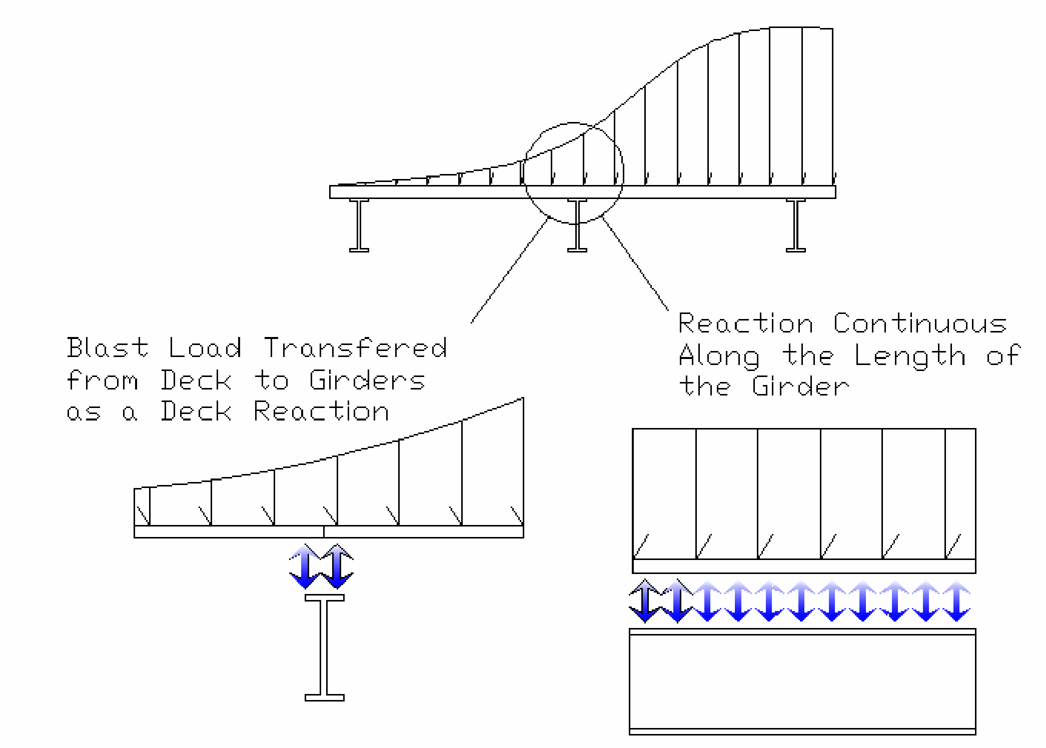


Figure 4.3 Load Path Diagram

4.3.2 CONWEP Uniform Load Applicability

Significant uncertainty exists when considering the nature of blast loads acting on a bridge system. As discussed earlier, some of this uncertainty has been accounted for through use of information about likely terrorist capabilities and courses of action, and through the use of performance-based standards. However, further investigation of applied blast loads is important. Many similarities exist between the analyses of the substructure and girder systems. The type of applied blast load used for pier analysis, however, is not suitable for use on girder systems. For the pier systems previously discussed, the CONWEP (USAE, 2003) software generated a uniform equivalent load that was used to define the loads used in the dynamic analyses. This load characterization, with modifications for wave reflections, was appropriate for pier systems because of their size and because of the way blast wave reflections occur beneath a bridge deck. A uniform equivalent load generated by CONWEP is not appropriate for girder systems because of the large span lengths of these members and the fact that CONWEP calculates the uniform equivalent load independent of the size of the reflecting surface, which in the case of a girder system is the entire surface of the bridge deck. The actual blast load acting on a bridge deck is similar in shape to that of a bell. Figure 4.4 shows a typical pressure distribution along the length of a girder.

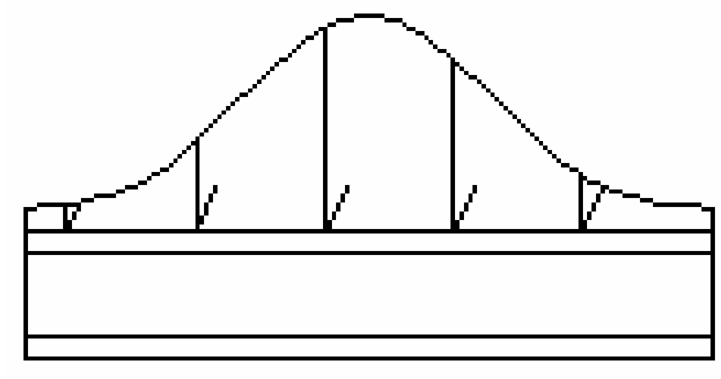


Figure 4.4 Typical Pressure Distribution Along a Girder Length

Because of this nonlinear distribution and the large span length, it is unreasonable to expect a system with a uniform equivalent load to perform in the same manner as one in which the loads are defined more precisely. Also available through CONWEP is a spatial distribution of the peak pressures and impulses acting on a deck surface. It can be seen in Figure 4.4 that a more appropriate method for approximating the actual pressure distribution over a blast-loaded deck would be to use a number of distributed loads as opposed to the single equivalent uniform load shown below in Figure 4.5.

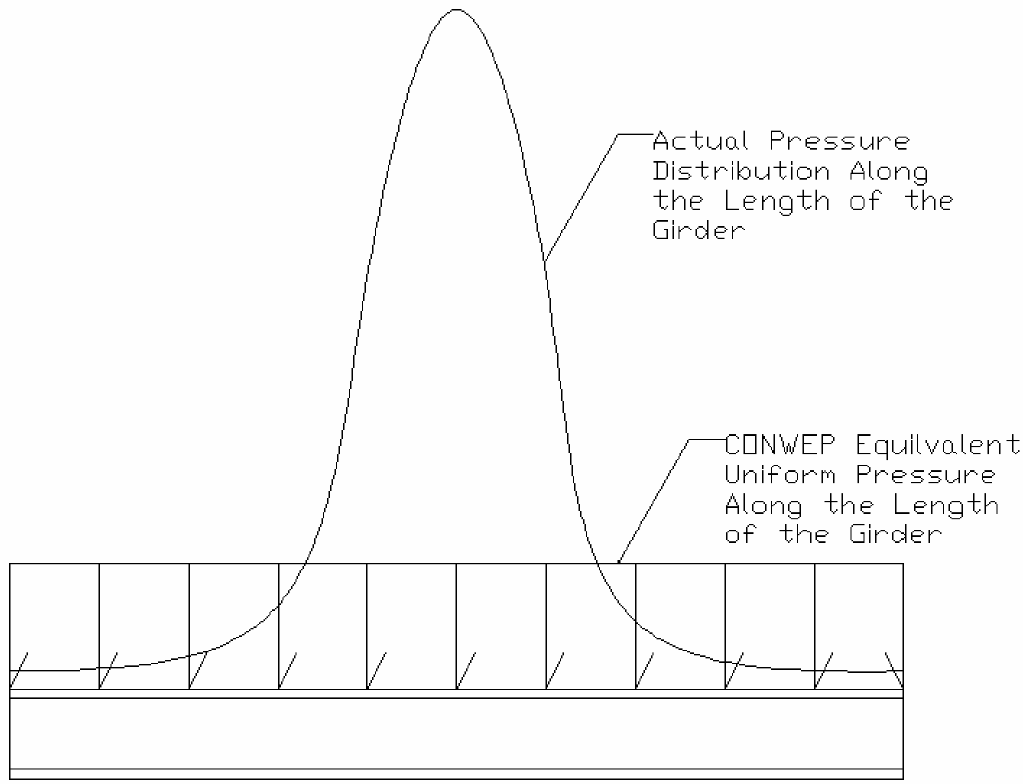


Figure 4.5 Deck Loading Options Provided by CONWEP

4.3.3 Use of Multiple Distributed Loads

The use of multiple distributed loads to approximate the actual blast loading calculated by CONWEP presents some difficulties which must be considered. The use of these distributed loads along a girder length creates the need for calculation of the single-degree-of-freedom system properties for that load type. As discussed in Chapter 3, "Modeling, Dynamics & Blast Loads," because of assumptions made regarding a component's displaced shape, system properties used for an equivalent SDOF analysis will vary depending on the type of loading considered. Because of the complicated actual shape of the blast load, this research approximates the blast using three distributed loads. A diagram of the general shape of a blast load and its approximation using distributed loads is shown in Figure 4.6 below.

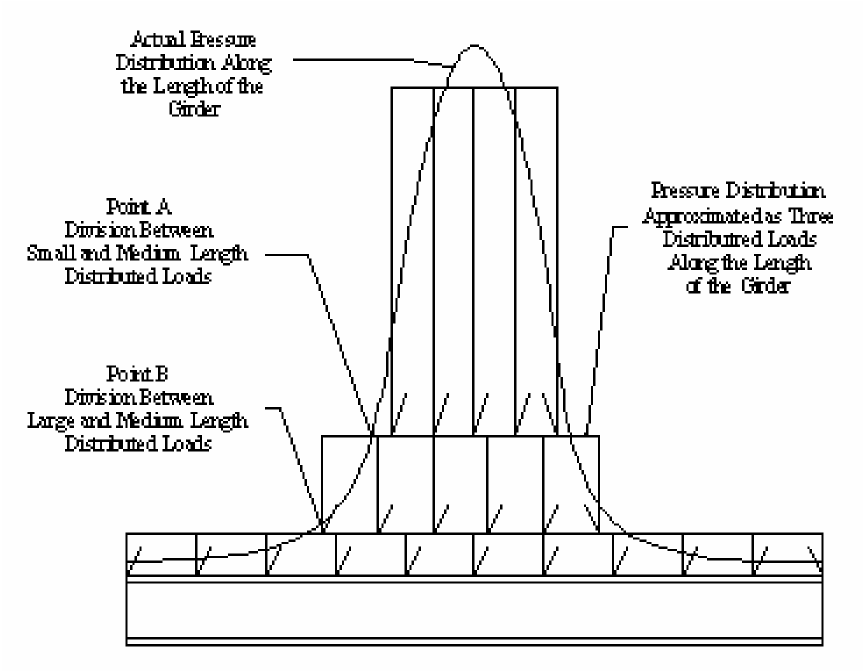


Figure 4.6 Blast Load Approximation as Three Uniformly Distributed Loads

Derivation of a girder's displaced shape, stiffness values, resistance limits, and load-mass factors are given in Appendix D. These properties were developed in a manner consistent with information presented in Chapter 3, but are somewhat mathematically intense because of the complicated functions required to define the displaced shape of a beam subjected to three different distributed loads of varying lengths.

Another difficulty presented by the use of multiple distributed loads is variation in structural response obtained as a result of the division of the actual load into these distributed loads. It is possible, using the same blast load, to divide the actual load into different magnitudes and lengths of uniform load acting along the girder length to approximate the actual load distribution, causing differences in structural response. Because of the relatively low magnitude and small variation of the blast load near the edges of a girder for a midspan blast, the location of the division between the longest length load and the medium length load (defined as point B in Figure 4.6) is not a critical parameter in system response. This observation does not hold for the division between the medium and short length distributed loads (defined as point A in Figure 4.6). In the region where this division is made, a large pressure gradient exists, and therefore structural response is influenced by this division location. Shown below in Figure 4.7 is the variation of several different girder systems' structural response (peak midspan displacement) with respect to the division location given as a percentage of the peak blast pressure.

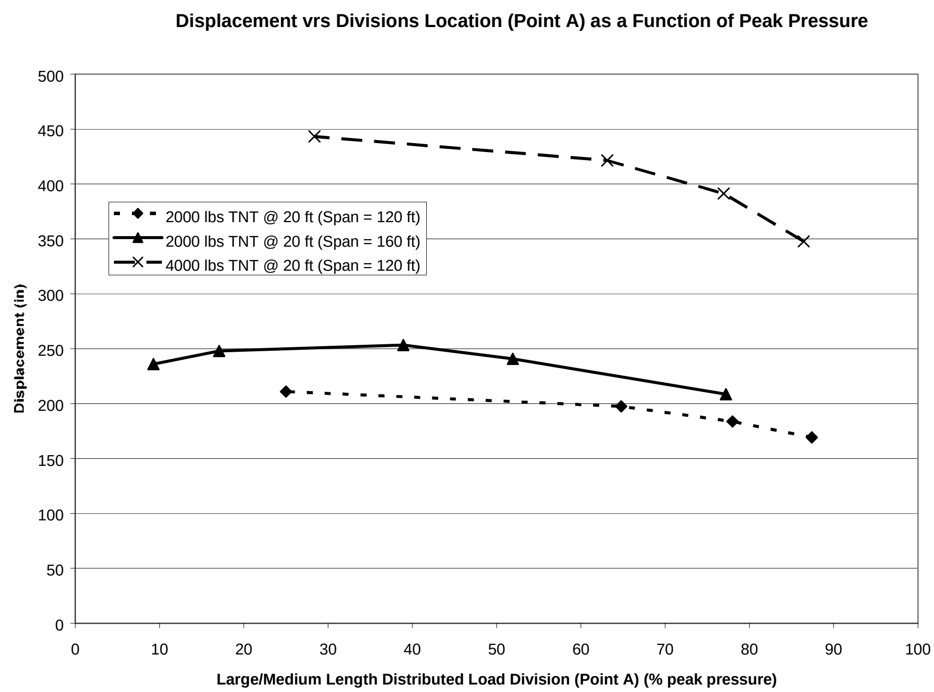


Figure 4.7 Displacement vs. Division Location for Girder Systems

In order to maintain a conservative approach to response estimation, the division between the medium and short length distributed loads should be made at the location of 45% of the peak pressure. This choice allows for the largest displacement to be calculated for most systems. It would be very difficult to verify that this value should be used for all systems because that would require an extremely large number of analyses for each structural configuration considered. Because it is the goal of this research to provide retrofit options through comparative study, it is most important that the largest number of system responses will be calculated conservatively. As such, the location corresponding to the location of 45% of the peak pressure is used for all girder system response calculations. Determination of a critical value for load division was attempted through the use of location along the length of a girder span, as well as relation to peak pressure. Through investigation of single degree-of-freedom analysis results it was determined that the most appropriate method was to use a percentage of the peak applied pressure.

4.3.4 Girder Analysis Procedure Outline

As discussed above, many assumptions are necessary for the analysis of bridge deck and girder systems subjected to blast loads. These assumptions led to the development of the load path procedure, and the definition of the blast load being represented by three different magnitudes of uniform load acting along a span. This section outlines the overall analysis procedure used in this research to perform parameter studies to determine the most effective retrofits to be used to improve dynamic response of girder systems subjected to blast loads.

To determine the response of a girder system, each girder is investigated individually. The CONWEP software is used to generate blast loads for above- and below-deck loadings. The above-deck loadings generated are used directly;

however the below-deck loads require magnification to include ground reflections not accounted for in CONWEP. This magnification is simply a multiplier of the impulse generated by CONWEP based on comparisons to loads generated in the BlastX software which accounts for ground reflection. This multiplier is applied as a constant to the entire spatial impulse distribution. Therefore, it is simply increasing the magnitude of the entire load acting on the bottom of the deck. The concept of an impulse multiplier of CONWEP-generated blast loads is used as opposed to BlastX loadings in an effort to simplify load determination. The BlastX software is not used directly because of the need to define precisely the below-deck geometry to accurately compute loads, complicated user inputs, and the relative ease of using loading data provided by CONWEP. Furthermore, CONWEP utilizes well-known formulas to describe variation in blast pressure and impulse as a function of time. Thus, even if it is not possible for bridge engineers to obtain CONWEP or BlastX because they are available only to government agencies and their contractors, other publicly available software (e.g., AT Blast, available at www.oca.gsa.gov) can be used to determine blast loads in a manner that is consistent with the approach being recommended by the current research. A detailed derivation of the magnification factors used to modify the CONWEP loads is discussed later in this chapter.

To carry out an analysis of a bridge deck and girder system, blast pressures as a function of time at each girder location are first generated within CONWEP, and potentially magnified for below-deck reflection effects. A uniform pressure across the width of the deck between any two girders is calculated by averaging the pressure at each girder location and assuming it to act uniformly between them. The concept of averaging the pressure distribution across a deck section between two girders is shown in Figure 4.8.

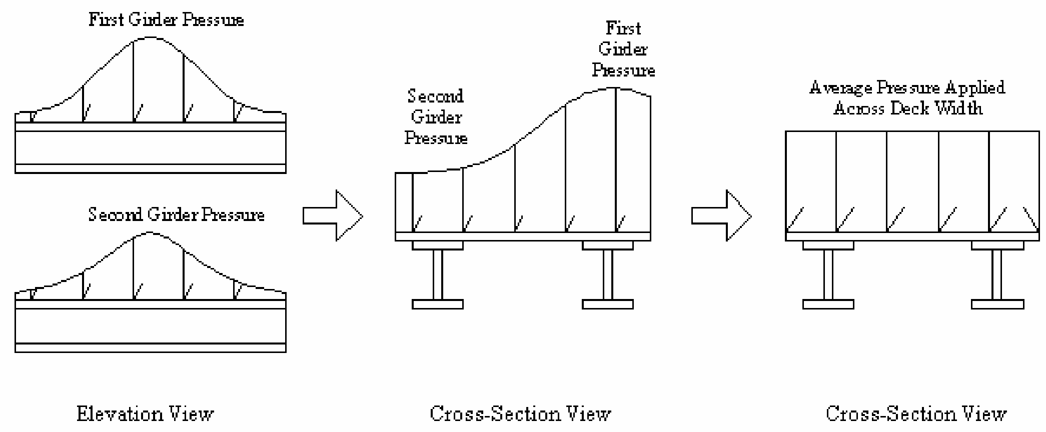


Figure 4.8 Averaging of Pressure Distribution Between Two Girders

The pressure acting on the area of each deck is assumed uniform over the width on a section between girders, and consists of three different magnitudes of distributed load over the length of a girder. Division of a blast load into three distributed loads is illustrated previously in Figure 4.6. This information is used to determine the magnitude of a uniform distributed load along the width of the deck between the two girders. A portion of the deck one foot in width is used to calculate the deck response under each distributed load. The representative deck section is subjected to the uniform distributed load, and the calculated reaction forces are used to load the girder along its length. The deck is assumed to be a fixed-pin supported one-way slab acting between the girders. The selection of these boundary conditions is based on the symmetry of deck spans between each girder. A deck section is assumed to be fixed over the center girder because no rotation will occur due to problem symmetry. A deck section is assumed pinned at the location of an adjacent girder because it is likely that the deck will be able to rotate. Because the amount of rotation possible is not known a conservative assumption is to assume the deck at this location can rotate freely. An illustration of the problem symmetry and associated boundary conditions is shown in Figure 4.9.

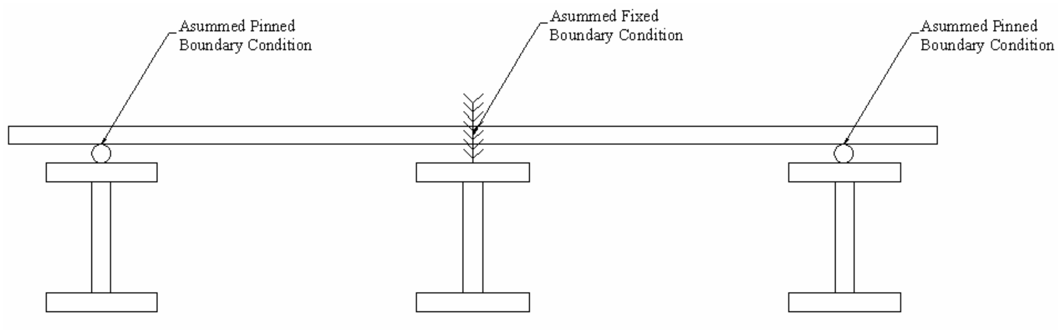


Figure 4.9 Deck Section Boundary Conditions

Deck properties such as stiffness, resistance limits, and load-mass factors are calculated using information obtained from Biggs (Biggs, 1964) in the same manner as discussed previously for pier systems. Deck reaction forces can be computed using dynamics principles, and will vary in time because of the dynamic nature of the load applied to the deck sections. As indicated earlier, a primary advantage to the load path approach is that it allows for the accounting of a failure of a portion of the deck. If the distributed load applied to the deck causes support rotations to exceed allowable limits as recommended by Conrath (Conrath, 1999), the deck is assumed to fail. At this point in time, the reactions created by that portion of the deck onto the girders are discontinued. The total impulse calculated to be acting on the girder in question from all deck section reaction forces is applied to the girder to compute its response.

The system properties for the girder such as the stiffness, resistance limits, and load-mass factor are calculated using concepts from "Introduction to Structural Dynamics" (Biggs, 1964), but specific formulas have been developed for a beam with three distributed loads (see Appendix D). For the purpose of the girder parameter studies, girder systems are assumed to be fixed at both ends as would be the case for girders in a continuous multiple span bridge, which would be the likely target of a terrorist attack. Analysis results are very sensitive to assumed support conditions. Therefore, it is necessary to investigate the effect of different options. In addition to continuous multi-span bridges, simply supported girder systems are quite common. Midspan deflections of a simply supported bridge span will be significantly greater than a continuous girder of equal size because of reduced stiffness and system strength. It is the use of this potentially unconservative estimation in girder parameter studies that necessitates a comparison of analysis results from each boundary condition type. Chapter 5 discusses an investigation of the effectiveness of recommended retrofit and design changes obtained from fixed-fixed girder parameter studies for use in simply supported girder bridges.

The equivalent single-degree-of-freedom girder used for analysis is subjected to the impulse loading caused by the deck reactions, and the dynamic response is calculated. Girder performance is judged on the basis of midspan displacement and compared to failure limits recommended by Conrath (Conrath, 1999). This information is used to formulate a relative comparison of design and retrofit effectiveness in blast mitigation.

4.4 BELOW-DECK MAGNIFICATION FACTOR DEVELOPMENT

Due to the ease of use and the provided spatial distributions of pressure and impulse, use of the CONWEP software is the preferred method of blast load characterization. As described above, however, the use of this software does create the necessity to find a means of accounting for pressure and impulse magnification due to shock wave reflections which are not already included within the software.   The method of determining this magnification can be accomplished through a comparison of CONWEP-generated loads with BlastX (USACE, 2003) generated loads. The BlastX software accounts for wave reflections and more accurately predicts blast load histories within enclosed or partially enclosed spaces. Because of its more complicated usage requirements and increased required amount of input data, the use of BlastX is less preferable to CONWEP for use in a large number of parameter studies.

A comparison of peak blast pressures and the pressure-time histories generated by BlastX and CONWEP illustrating similarities in peak pressure and variation in impulse is shown in Figure 4.10. Figure 4.10 also demonstrates that the impulse is different for the two software packages.

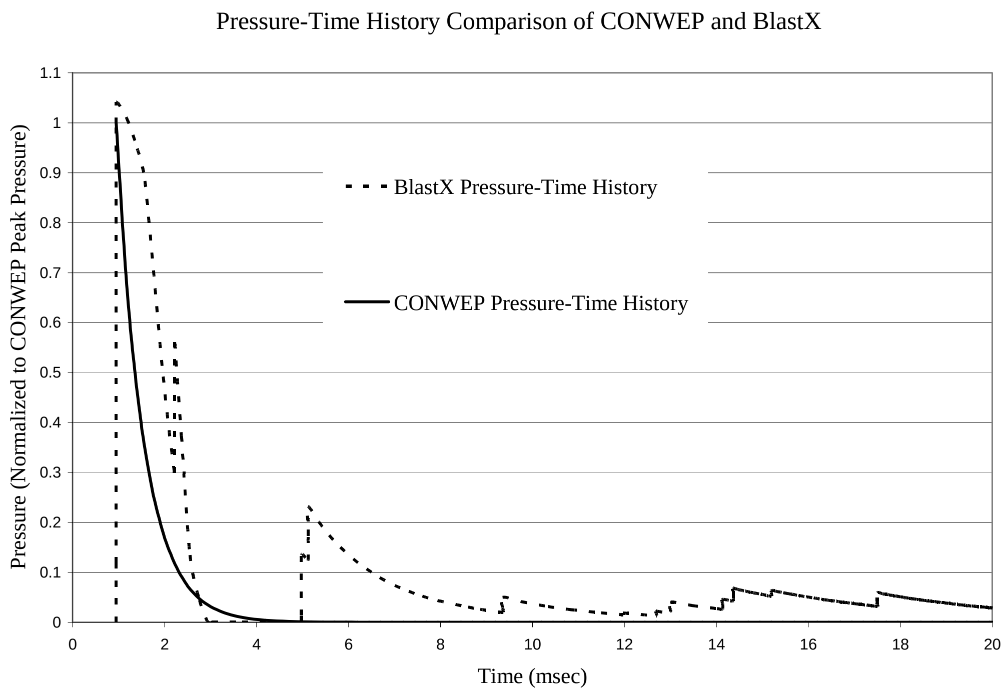


Figure 4.10 Pressure-Time Histories Developed by BlastX and CONWEP

Due to the fact that a blast load is an extremely impulsive event, and the natural period of a girder system is significantly larger than the blast duration for the scenarios considered in this research as most likely terrorist threats, the shape of the blast pressure time history is of little consequence. Because of this, as long as total impulse is retained, the same response will be compute even without following the exact load history given by BlastX. If the same peak pressure is retained and the assumption is made that the load decays linearly, the time needed to achieve the same impulse as given by BlastX can be calculated. This load definition gives the same structural response as the one that would result from the use of the BlastX load history directly. For the purposes of this research, it is assumed that the applied blast is a negative sloping line progressing in time from the peak pressure to zero pressure as shown in Figure 4.11. A magnification factor determined for each case in question is used to increase the peak impulse value provided by CONWEP.

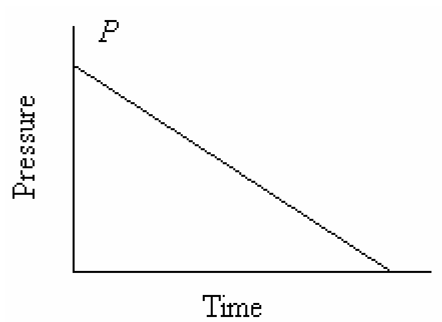


Figure 4.11 Assumed Pressure-Time History of an Applied Blast Load

Through comparison of loads generated by each computer program, including both straight and sloped abutments, it is observed that for deck and girder systems the peak blast pressure distribution is nearly identical. Through further investigation, it can be observed that the blast magnification due to reflections occurs in the duration of the blast load because of the increased number of reflected waves striking the target area over a longer period of time. This observation allows for formulation of a relationship between several geometric properties of a deck and girder system and the impulse magnification factor. Using an empirically based approach, it can be shown that the impulse magnification factor is related to charge weight, standoff distance, and span length. This relationship is shown in Figure 4.12 in which several different systems' magnification factors are shown with a regression equation demonstrating the observed relationship. The relationship is shown for straight abutments only because it is abutments of this type that produce the largest impulse magnification, and therefore are the critical case to study to improve blast mitigation of a bridge system.

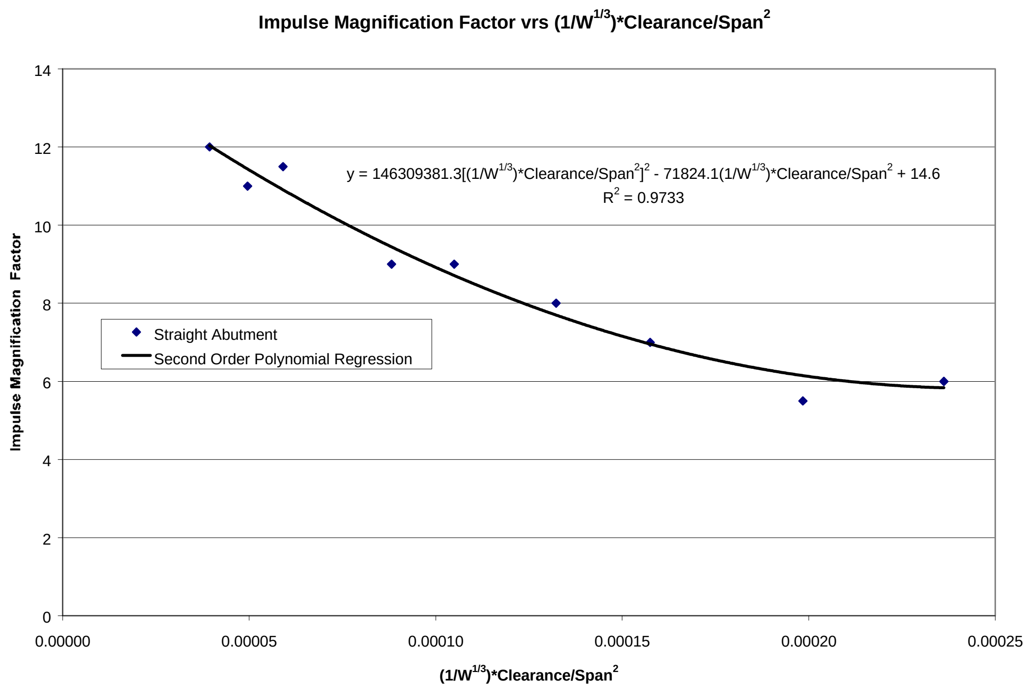


Figure 4.12 Relationship of Geometry and Impulse Magnification Factor for Girder Systems with Straight Abutments

It is this increased impulse that is used to calculate the duration of the blast load having a peak pressure given by the CONWEP spatial pressure distribution. The impulse of this function is calculated using Equation 4.1 shown below where I is the magnified impulse, P is the peak blast pressure, and t is the duration of the blast load.

$$I = \tfrac{1}{2} P\, t \tag{4.1}$$

The relationship shown above can be used to calculate the blast durations needed to perform the analyses of the girder systems.

4.5 GIRDER PARAMETERS INVESTIGATED

The purpose of this research is to identify retrofits and design changes useful in mitigating risk of terrorist attack to structures and to provide guidelines to design engineers that may be unfamiliar with blast design. Development of these guidelines requires a study of relative parameter effectiveness in improving structural response of bridges to blast loads. Parameters are chosen for investigation based on their effect on model properties such as strength and stiffness, their effect on applied blast loads, and on their presence in typical bridge design. Parameters investigated include girder size, span length, girder spacing, deck thickness, and girder steel strength. A complete list including the coupling of various parameters can be found in Appendix E.

4.6 GIRDER SYSTEM PERFORMANCE-BASED STANDARDS

The concept of performance-based standards has been introduced both in previous chapters of this research, and in “Design of Critical Bridges for Security against Terrorist Attack” (Winget, 2003). This section provides information about their use specifically for girder systems.

The failure limits used for girder systems are based on values reported by the USACE in the Security Engineering Design Manual (ASACE, 2004). As discussed previously, the research reported by the USACE is based on response of building components and modification are required to reflect the expected differences in response between bridge and building systems. For the purposes of this study, deformation limits for steel girders are shown in Table 4.2.

Table 4.2 Deformation Limits for Steel Girders

Failure Deformation Limits of Steel Girders (Ductility Limit)	**Failure Deformation Limits of Steel Girders (Ductility Limit)**
Event Magnitude	**Event Magnitude**
Large	Mid-Size
16	8

The design recommendations developed based on performance-based standards provide specific guidelines as to retrofit options and design advice to achieve a specified performance level. Design recommendations for girder systems are provided in chapter 5, however a general example is shown in Figure A.4 of Appendix A of this report.

4.7 GIRDER SYSTEM INVESTIGATION SUMMARY

Analysis of girder systems as single-degree-of-freedom systems subjected to blast loads requires several assumptions about system and load properties. Certainly, identical results will not be obtained through a different set of assumptions, but it is quite likely that a very similar relative comparison of retrofit effectiveness would be obtained. To this end, the use of the reasonable and conservative assumptions outlined in the previous sections of this chapter provides an efficient method for determining structural response.

The analysis procedure summarized in this chapter is tailored specifically to the determination of the dynamic flexural response of a girder system. Investigation of shear response of girders under blast loading utilizes the same load generation procedure, however dynamic shear failure is best determined using the procedure proposed by Norman Jones (Jones, 1995). Because of the large span lengths and high shear capacity of the sections studied, shear response proved not to be the controlling failure mechanism. Jones' procedure utilizes pressure and impulse to determine shear wave propagation through a beam. Failure is determined based on wave velocity and shear strength. It was determined that shear failure was not critical by applying the largest impulse to and pressure acting over each span to the smallest section studied for each load case in this research. Data demonstrating each section's ability to resist the applicable impulses is omitted for security reasons. One significant differences between analysis of flexure and shear response of girders lies in the choice of load location and the parameters selected to characterize structural response. In the case of flexure, the load location most critical to dynamic response is at midspan, and the parameter used to track response is midspan displacement. In the case of shear, the critical blast location is near a pier or abutment, and the parameter used to track response is the single-degree-of-freedom spring internal resistance, which using dynamics principles can be converted into a shear force.

It should be noted that these described critical locations are not necessarily critical for every girder system because of the variability in applied load due to failures in the deck. However, it is expected that the discussed critical locations will provide the most severe response for the largest number of girder systems which will provide the most useful relative comparisons of parameter effectiveness.

In addition to shear and flexural considerations, it is also important to consider other possible failure modes for girders. Specifically, the potential for local buckling of the flanges has not be investigated under blast loading, but rather guidelines are presented in the next chapter to help prevent this failure mode from controlling girder response.

CHAPTER 5

Superstructure Results & Recommendations

5.1 INTRODUCTION

Chapter 4 of this report outlines the methods and assumptions involved in the analysis of steel superstructure systems. The chapter also provides a description of the concept of performance-based standards used to formulate recommendations to improve steel girder system response to blast loads. This chapter presents the results of the completed analyses, makes comparisons of retrofit performance, discusses observed trends in analysis results, and provides recommendations for mitigation of terrorist threats to steel substructures.

5.2 SUMMARY OF RETROFITS AND DESIGN CHANGES INVESTIGATED

Chapter 4 focuses on the determination of flexural response of bridge superstructure systems because it is expected that the primary mode of failure for impulsively loaded long-span steel girders will be due to flexure. A typical girder subjected to a significant blast will lead to the development of one or more plastic hinges (as discussed in Chapter 3) and potentially large plastic deformations. Accordingly, design change and retrofit options for steel girders to improve bending performance were investigated. It should be noted that the effects of shear were also studied, but it was determined that improving flexural response would be most effective in mitigating blast effects.

As previously discussed, structural response was calculated under two different load magnitudes for the current research project. Two separate sets of three different steel plate girder cross-sections were examined, one set for moderate level blast loads, and one set for large threats. As a further measure to quantify the benefits of section strength increases, steel yield strengths of 50 and 75 ksi were used for calculation of single degree-of-freedom system parameters. Different section strengths, shapes and sizes will effect the performance of girder models subjected to blast loads by varying the stiffness, system mass, and resistance limits.

Structural layout is an extremely significant component of blast dynamics. When considering blasts, geometry affects pressure and impulse magnitudes, variations in time, and distributions in space. In addition to varying section strength and geometry, three different span lengths were studied. Spans selected for this research were 80, 120, and 160 feet in length. Changes in span of a girder subjected to a blast also have an effect on the distribution of pressure along the length, and the magnification of below-deck pressures as discussed in Chapter 6. Just as with section shape and strength, changes in span length effect girder model flexural stiffness, mass and resistance limits.

Chapter 3 includes a discussion on blast properties and the effects of reflections and standoff distances on load magnitude. The importance of these concepts requires inclusion of bridge clearance as a parameter to be studied. Changes in clearance lead to changes in standoff distances from blasts to girders for below-deck scenarios, which have a significant effect on the applied peak blast pressure. Also, as discussed in the previous chapter, the magnification of impulse to account for blast wave reflections is dependant on standoff. For this study, clearance distances between the ground and bottom of a girder were assumed to be 16, 20, or 24 feet.

Another important aspect of modeling structural response under blast loads is the internal load path of the forces. The flexural model of superstructure systems used for this research assumes that the blast pressure strikes the deck surface, and the deck response to that pressure generates a load as a function of time for the supporting girders. The amount of load transferred to the girders from the deck is dependant on the performance of the deck sections. The models used in this research assume that the deck reactions are transferred to the supporting girders until the deck portion in question fails under load. Failure of the deck is assumed to occur when selected end rotational limits are exceeded. This limiting end rotational limit was selected to be a peak deflection of five percent of half of the transverse deck span length (Conrath, 1999). When considering that the load to which the girders are subjected is dependant on the integrity of the deck and its ability to transfer force to a supporting girder, it can easily be seen that deck strength is an important parameter which requires investigation. In this research, deck strength was investigated through variation of deck thickness. Unlike the retrofit or design options considered and discussed above for which increases in strength and stiffness will improve girder performance, deck strength should be minimized to allow for failure early in time so that venting of loads will improve overall system performance. Concrete decks with thicknesses of eight, ten, and fourteen inches were considered.

Just as with girders, span length also affects the response of a deck section. In this case, the spacing between supporting girders represents the deck span. Variation of girder spacing will affect deck response and change the single degree-of-freedom system parameters, and it will also affect the load distribution and magnitudes acting on the deck section in question. For these reasons, girder spacing is another critical parameter which must be considered as a potential design change or retrofit recommendation. This research considered girder spacings of eight and twelve feet.

5.3 RESULTS AND OBSERVATIONS

In order to develop recommendations for the most effective design change and retrofit options, a relative comparison of different systems' flexural response must be obtained. As mentioned in the introduction of this chapter, flexural response was determined to be the controlling mode of failure for these reasonably long-span built-up steel sections. Shear response was investigated for the most severe load cases for critical scenarios; however an all-inclusive relative comparison of system configurations was not necessary because it was verified that shear is not the controlling failure mode. For this reason, results generated from shear response analyses were not used to formulate recommendations. Results for the analysis cases including shear effects can be found in Appendix H.

5.3.1 Bridge Clearance

Bridge clearance affects girder response because the standoff distance from the explosion source to the target changes with height of the bridge. With changing standoff, both the applied blast pressures and impulses vary. The difference in impulse generated by a blast at different clearances beneath a deck and transferred to the girders depends on several factors such as span length, deck thickness, and charge weight. It is for this reason that the importance of clearance for blast mitigation is difficult to quantify. Because of the success seen by increasing standoffs for improving column response to blast loads, it is expected that girder clearance would be of critical importance. Parameter studies on steel girder systems do not necessarily demonstrate these results however. Shown in Table 5.1 are response calculations for various system configurations at different threat magnitudes.

Table 5.1 Results of Selected Parameter Studies for Clearance

Clearance	Standoff	Charge Wt	Span	Sect Depth	Steel Fy	Girder Spacing	Deck Thickness	Displ.	Failure Displ.	Failure
(ft)	(ft)		(ft)	(in)	(ksi)	(ft)	(in)	(in)	(in)	
16	10	Large	80	72	50	8	14	72.8	27.9	Yes
20	14	Large	80	72	50	8	14	56.6	27.9	Yes
24	18	Large	80	72	50	8	14	38.4	27.9	Yes
16	10	Large	80	72	75	8	14	50.0	41.9	Yes
20	14	Large	80	72	75	8	14	39.0	41.9	No
24	18	Large	80	72	75	8	14	26.9	41.9	No
16	10	Large	80	72	50	8	14	46.1	27.9	Yes
20	14	Large	80	72	50	8	14	35.9	27.9	Yes
24	18	Large	80	72	50	8	14	24.5	27.9	No
16	12	Small	120	72	75	8	10	31.2	47.1	No
20	16	Small	120	72	75	8	10	27.1	47.1	No
24	20	Small	120	72	75	8	10	22.4	47.1	No
16	12	Small	120	66	50	8	10	76.2	34.3	Yes
20	16	Small	120	66	50	8	10	63.0	34.3	Yes
24	20	Small	120	66	50	8	10	50.2	34.3	Yes
16	12	Small	120	66	75	8	10	53.3	51.4	Yes
20	16	Small	120	66	75	8	10	45.2	51.4	No
24	20	Small	120	66	75	8	10	36.7	51.4	No

Table 5.1 shows that the reduced load acting on these systems does have an effect on peak displacements, however the reduced displacements caused by increased standoffs prevented failures in only a small number of scenarios. The fact that standoff increases did not deter failure leads to the observation that the magnitude of change in impulse is not alone significant enough within the range of clearances studied to be a primary method of blast mitigation. Table 5.1 does show some situations in which increased clearance did prevent failure, and decreased displacements within other sets of data does validate increased standoff as a potential design change for new bridges. These results, combined with knowledge of blast effects on structures indicate that maximizing bridge clearance is appropriate for improvement of girder performance.

5.3.2 Section Size and Geometry

Use of stronger and stiffer cross-sections is a logical and direct method of improving girder performance under dynamic loads. Because failure is determined through comparison of peak midspan displacement to an acceptable limit, larger sections will inevitably offer improved response. As mentioned above, three different sections were examined for each threat magnitude. These different sections were selected to represent typical plate girders in use, or an increased size to improve performance. Table 5.2 below shows cross-sectional properties of the girders studied, and Tables 5.3 and 5.4 compare the properties of the larger sections studied for each threat level relative to the "base" cross-section (the base cross section is section 3 for "small" loads, and section 1 for large loads).

Table 5.2 Section Properties of Girders Used in Parameter Studies

Section Number	Cross Sectional Area	Moment of Inertia	Plastic Section Modulus
	(in^2)	(in^4)	(in^3)
1	83.5	47007	4680
2	133.5	94716	3274
3	168	150408	1811
4	183	181396	5487
5	204	203106	6161

Table 5.3 Ratios of Section Properties for Small Loads

Section Number	Cross Sectional Area	Moment of Inertia	Plastic Section Modulus
1	1.00	1.00	1.00
2	1.60	2.02	1.81
3	2.01	3.20	2.58

Table 5.4 Ratios of Section Properties for Large Loads

Section Number	Cross Sectional Area	Moment of Inertia	Plastic Section Modulus
	(in^2)	(in^4)	(in^3)
3	1.00	1.00	1.00
4	1.09	1.21	1.17
5	1.21	1.35	1.32

The purpose of this research is to determine system configurations which mitigate risk to bridge structures under blast. Change in section size offered a large improvement in performance in many scenarios because size increases were chosen to be adequate to prevent flexural failure. Because of the large increases in size leading to improved performance, these larger section sizes offer a method of blast mitigation. Figure 5.1 shows the geometries of the studied cross-sections, and Table 5.5 illustrates the variation in displacements and failures obtained from changes to girder size and geometry.

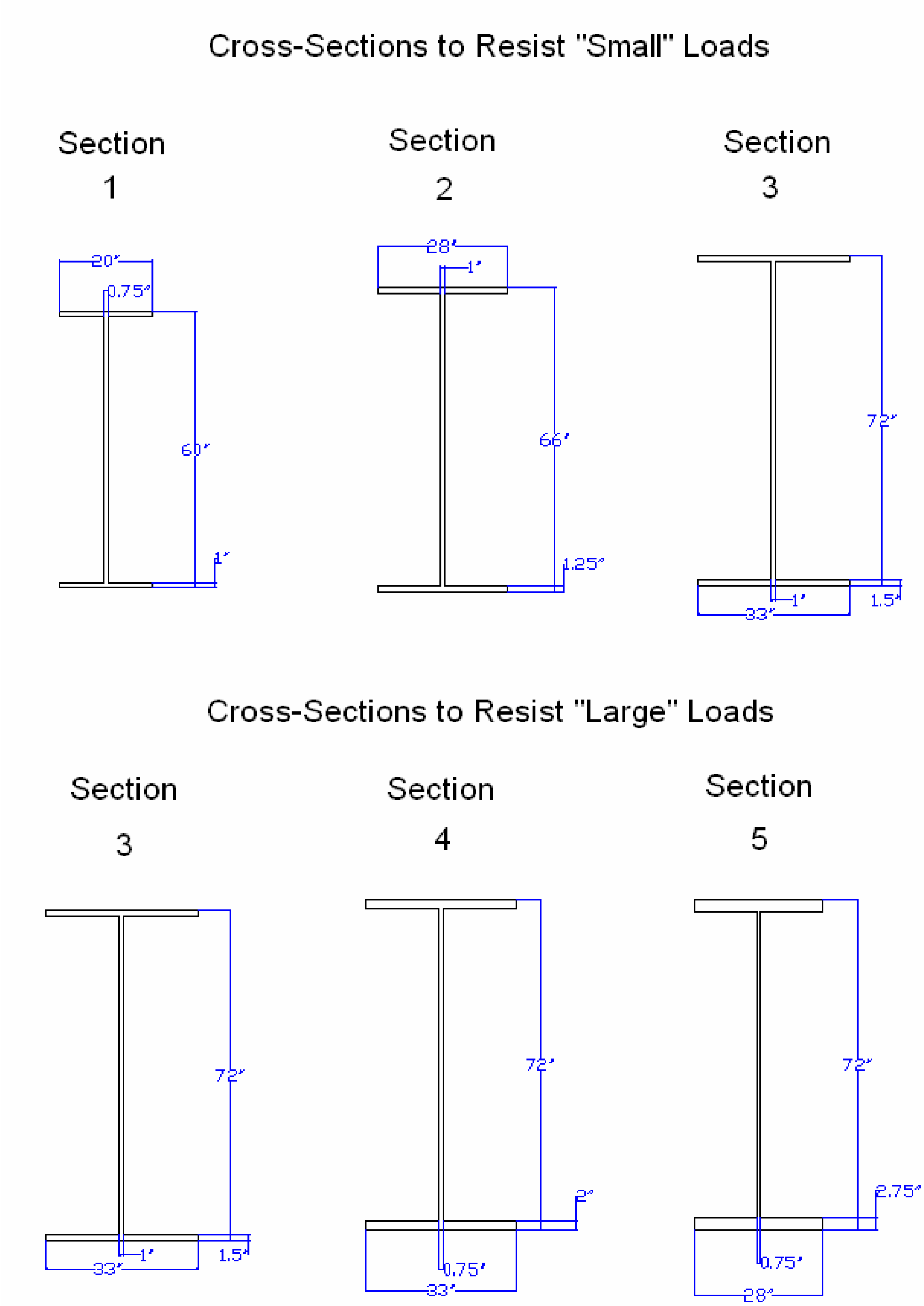


Figure 5.1 Cross-Section Geometries Studied

Table 5.5 Results of Selected Parameter Studies for Girder Size

Section #	Sect Depth	Charge Wt	Clearance	Standoff	Steel Fy	Deck Thickness	Girder Spacing	Span	Displ.	Failure Displ.	Failure
	(in)		(ft)	(ft)	(ksi)	(in)	(ft)	(ft)	(in)	(in)	
1	60	Small	16	12	75	14	12	160	436.3	100.5	Yes
2	66	Small	16	12	75	14	12	160	156.2	91.4	Yes
3	72	Small	16	12	75	14	12	160	89.9	83.8	Yes
1	60	Small	20	16	50	8	8	160	175.0	67.0	Yes
2	66	Small	20	16	50	8	8	160	64.2	60.9	Yes
3	72	Small	20	16	50	8	8	160	37.8	55.8	No
1	60	Small	Above	4	50	10	8	120	65.3	37.7	Yes
2	66	Small	Above	4	50	10	8	120	25.1	34.3	No
3	72	Small	Above	4	50	10	8	120	15.3	31.4	No
1	60	Small	16	12	75	8	8	120	115.7	56.5	Yes
2	66	Small	16	12	75	8	8	120	42.8	51.4	No
3	72	Small	16	12	75	8	8	120	25.3	47.1	No
1	60	Small	Above	4	75	10	12	80	49.0	25.1	Yes
2	66	Small	Above	4	75	10	12	80	18.4	22.8	No
3	72	Small	Above	4	75	10	12	80	11.0	20.9	No
3	72	Large	16	10	50	14	8	160	159.0	111.7	Yes
4	72	Large	16	10	50	14	8	160	125.6	111.7	Yes
5	72	Large	16	10	50	14	8	160	101.4	111.7	No
3	72	Large	20	14	75	10	12	160	140.1	167.5	No
4	72	Large	20	14	75	10	12	160	111.4	167.5	No
5	72	Large	20	14	75	10	12	160	90.7	167.5	No
3	72	Large	20	14	50	10	8	120	71.0	62.8	Yes
4	72	Large	20	14	50	10	8	120	56.2	62.8	No
5	72	Large	20	14	50	10	8	120	45.6	62.8	No
3	72	Large	Above	6	50	14	12	80	33.1	27.9	Yes
4	72	Large	Above	6	50	14	12	80	26.2	27.9	No
5	72	Large	Above	6	50	14	12	80	21.2	27.9	No
3	72	Large	24	18	50	8	12	80	59.4	27.9	Yes
4	72	Large	24	18	50	8	12	80	46.8	27.9	Yes
5	72	Large	24	18	50	8	12	80	37.7	27.9	Yes

The large improvements in girder performance suggest that increasing mass to increase inertial resistance, plastic section capacity, and moment of inertia of a section either through retrofit by addition of cover plates, or through new design is a useful hardening technique.

5.3.3 Steel Strength

Another method of increasing girder strength is through increases in material strength. Changing steel strength of existing girders is not an option for blast mitigation; however use of higher strength cover plates for existing bridges as a retrofit, or using higher strength steel for new construction, will have a positive effect on bridge performance. This research considered steel with yield strengths of 50 and 75 ksi (prior to magnification for material over strength and dynamic strength increase). Results shown in Table 5.6 demonstrate the effectiveness of strength increase to decrease displacements and the number of girder failures for a given bridge configuration.

Table 5.6 Results of Selected Parameter Studies for Steel Yield Strength

Steel Fy (ksi)	Charge Wt	Clearance (ft)	Standoff (ft)	Span (ft)	Sect Depth (in)	Girder Spacing (ft)	Deck Thickness (in)	Displ. (in)	Failure Displ. (in)	Failure
50	Large	16	10	120	72	8	14	123.1	62.8	Yes
75	Large	16	10	120	72	8	14	85.0	94.2	No
50	Large	16	10	120	72	12	8	153.3	62.8	Yes
75	Large	16	10	120	72	12	8	105.1	94.2	Yes
50	Large	Above	4	120	72	12	8	28.5	62.8	No
75	Large	Above	4	120	72	12	8	21.6	94.2	No
50	Large	20	14	80	72	8	10	135.8	111.7	Yes
75	Large	20	14	80	72	8	10	95.3	167.5	No
50	Small	16	12	80	66	8	10	40.8	15.2	Yes
75	Small	16	12	80	66	8	10	28.5	22.8	Yes
50	Small	Above	4	80	66	8	10	17.1	15.2	Yes
75	Small	Above	4	80	66	8	10	12.7	22.8	No
50	Small	Above	4	160	60	8	8	58.9	67.0	No
75	Small	Above	4	160	60	8	8	45.0	100.5	No
50	Small	20	16	160	60	8	8	175.0	67.0	Yes
75	Small	20	16	160	60	8	8	122.3	100.5	Yes
50	Small	24	20	160	66	8	8	54.3	60.9	No
75	Small	24	20	160	66	8	8	41.2	91.4	No

The large number of failures prevented through increase in steel strength suggests that use of higher grade steel is an effective design change option. It is important to consider that a bridge designed for the same gravity loads with higher grade steel would consist of smaller section sizes than that of a bridge consisting of lower strength steel if designed with no consideration of blast resistance. Thus, when protecting a bridge from blast, these higher magnitude loads must be accounted for so that girders can be sized appropriately.

5.3.4 Girder Span

Both stiffness and ultimate flexural resistance of a bridge system are heavily dependant upon the inverse of span length. For this reason, it would be expected that shorter span lengths would perform better than longer spans under blast loads. Data from parameter studies however, suggest that span length does not strongly influence the computed response. In fact, results from the single degree-of-freedom analyses show, through a modest decrease in the number of failures, that girders of larger span length actually performed slightly better than shorter girders. The reason for this result is that the failure criterion is based on ductility and midspan displacement normalized by the girder length. As such, longer girders can undergo larger magnitude displacements than shorter girders before failure occurs. Also, regardless of span length, the blast distribution over the center portion of the span, the portion where the greatest amount of impulse occurs, is the same for both long and short girders. The combination of these factors limits decreases in system stiffness and strength from being detrimental to performance. Table 5.7 illustrates that span length was not an important factor in system performance, and it also shows that some configurations were able to perform better with larger span lengths.

Table 5.7 Results of Selected Parameter Studies for Span Length

Span (ft)	Charge Wt	Clearance (ft)	Standoff (ft)	Steel Fy (ksi)	Sect Depth (in)	Girder Spacing (ft)	Deck Thickness (in)	Displ. (in)	Failure Displ. (in)	Failure
80	Large	16	10	50	72	8	14	72.8	27.9	Yes
120	Large	16	10	50	72	8	14	123.1	62.8	Yes
160	Large	16	10	50	72	8	14	159.0	111.7	Yes
80	Large	16	10	50	72	8	10	32.4	27.9	Yes
120	Large	16	10	50	72	8	10	56.3	62.8	No
160	Large	16	10	50	72	8	10	69.9	167.5	No
80	Large	Above	4	75	72	8	8	8.4	41.9	No
120	Large	Above	4	75	72	8	8	10.9	94.2	No
160	Large	Above	4	75	72	8	8	14.2	167.5	No
80	Small	24	20	75	66	12	14	109.7	25.1	Yes
120	Small	24	20	75	66	12	14	228.9	56.5	Yes
160	Small	24	20	75	66	12	14	311.1	100.5	Yes
80	Small	20	16	75	66	8	10	23.6	22.8	Yes
120	Small	20	16	75	66	8	10	45.2	51.4	No
160	Small	20	16	75	66	8	10	61.0	91.4	No
80	Small	16	12	75	60	8	8	13.1	20.9	No
120	Small	16	12	75	60	8	8	25.3	47.1	No
160	Small	16	12	75	60	8	8	35.8	83.8	No

5.3.5 Deck Thickness

Deck thickness is a critical parameter for bridge systems subjected to blast loads. The reason for this importance is derived from the method of transferring loads from a blast into girders. Deck sections are subjected to blast pressures, and the corresponding reactions are resisted by girders. When a deck section fails, the reactions are no longer transferred, and therefore weaker deck sections can transfer less load by failing earlier in time relative to stronger deck sections. In effect, the deck can be viewed as a sacrificial portion of the system, and its failure can limit the damage that occurs to critical structural components. This research investigated deck thickness of eight, ten, and fourteen inches as a method of determining the effects that increased deck strength has on increasing blast loads transferred. Table 5.8 below shows the magnitude of increase in impulse for a variety of different systems subjected to the same load. Each row represents one system configuration, and the data are normalized such that the impulse transferred by an eight inch deck section is taken to be one.

Table 5.8 Impulse of Selected Systems for Different Deck Thicknesses

Deck Thickness	Load Case	Standoff	Clearance	Girder Spacing	Span	Impulse (Normalized to 8 in deck Impulse)
(in)		(ft)	(ft)	(ft)	(ft)	
8	Large	14	20	8	120	1.00
10	Large	14	20	8	120	1.16
14	Large	14	20	8	120	1.40
8	Large	10	16	12	120	1.00
10	Large	10	16	12	120	1.13
14	Large	10	16	12	120	1.32
8	Small	4	Above	8	160	1.00
10	Small	4	Above	8	160	1.18
14	Small	4	Above	8	160	1.35

As is illustrated above, deck thickness is a very important parameter for blast mitigation. Table 5.9 illustrates the benefits gained by reduction in deck thickness. For this reason minimizing deck thickness is a very effective method of reducing loads which must be resisted by critical bridge components. Although the use of a thin deck will allow for deck damage to propagate further, it is better for overall bridge performance to reduce the load acting on the important structural system.

Table 5.9 Results of Selected Parameter Studies for Deck Thickness

Deck Thickness (in)	Charge Wt	Clearance (ft)	Standoff (ft)	Steel Fy (ksi)	Sect Depth (in)	Girder Spacing (ft)	Span (ft)	Displ. (in)	Failure Displ. (in)	Failure
14	Large	16	10	50	72	8	160	159.0	111.7	Yes
10	Large	16	10	50	72	8	160	108.7	111.7	No
8	Large	16	10	50	72	8	160	82.9	111.7	No
14	Large	Above	4	75	72	12	120	27.6	94.2	No
10	Large	Above	4	75	72	12	120	23.4	94.2	No
8	Large	Above	4	75	72	12	120	21.6	94.2	No
14	Large	20	14	50	72	8	80	56.6	27.9	Yes
10	Large	20	14	50	72	8	80	40.9	27.9	Yes
8	Large	20	14	50	72	8	80	29.9	27.9	Yes
14	Small	16	12	50	72	8	160	82.2	55.8	Yes
10	Small	16	12	50	72	8	160	59.0	55.8	Yes
8	Small	16	12	50	72	8	160	46.8	55.8	No
14	Small	24	20	75	72	12	120	47.9	47.1	Yes
10	Small	24	20	75	72	12	120	41.5	47.1	No
8	Small	24	20	75	72	12	120	34.2	47.1	No
14	Small	16	12	50	66	8	80	63.3	15.2	Yes
10	Small	16	12	50	66	8	80	40.8	15.2	Yes
8	Small	16	12	50	66	8	80	31.3	15.2	Yes

5.3.6 Girder Spacing (Transverse Deck Span)

Just as with reduction in loads achieved through minimizing of deck thickness, decreasing girder spacing will reduce the amount of blast impulse per girder. A particular blast load will have a finite distribution over a flat area, and decreasing the girder spacing provides additional girders within that effected area to resist loads. Clearly, an increase in the number of girders resisting a blast load will lead to improved system performance. This research focused on commonly used girder spacings of eight and twelve feet. Reduction of girder spacing provided one of the most effective methods of blast mitigation observed in the parameter studies for this research. The reduction in failures experienced is demonstrated in Table 5.10.

Table 5.10 Results of Selected Parameter Studies for Girder Spacing

Girder Spacing (ft)	Charge Wt	Clearance (ft)	Standoff (ft)	Steel Fy (ksi)	Sect Depth (in)	Deck Thickness (in)	Span (ft)	Displ. (in)	Failure Displ. (in)	Failure
8	Large	16	10	75	72	10	80	35.4	41.9	No
12	Large	16	10	75	72	10	80	87.4	41.9	Yes
8	Large	20	14	50	72	14	120	80.8	62.8	Yes
12	Large	20	14	50	72	14	120	173.2	62.8	Yes
8	Large	16	10	50	72	8	120	51.8	62.8	No
12	Large	16	10	50	72	8	120	120.7	62.8	Yes
8	Large	Above	4	50	72	14	160	30.3	111.7	No
12	Large	Above	4	50	72	14	160	46.1	111.7	No
8	Large	24	18	50	72	8	160	63.2	111.7	No
12	Large	24	18	50	72	8	160	135.2	111.7	Yes
8	Small	16	12	75	72	14	80	24.9	20.9	Yes
12	Small	16	12	75	72	14	80	29.3	20.9	Yes
8	Small	20	16	75	66	8	80	18.1	22.8	No
12	Small	20	16	75	66	8	80	38.8	22.8	Yes
8	Small	Above	4	50	66	10	120	25.1	34.3	No
12	Small	Above	4	50	66	10	120	33.6	34.3	No
8	Small	20	16	75	66	14	160	84.0	91.4	No
12	Small	20	16	75	66	14	160	135.3	91.4	Yes

5.3.7 Cable Restrainers and Girder Seat Requirements

Because some girders subjected to blast loads undergo large deformations, the ends of these sections may be pulled away from the supporting bent cap. If the movement on the supporting bent is large enough, the girder sections may be pulled off of the edge. This occurrence is referred to as seating loss, and retrofits available for prevention include restraint of the blast-loaded girders by cables that connect sections in adjacent spans, and extensions of the supporting structure. Figure 5.2 illustrates the use of cable restraints and seat extensions to mitigate seating loss risks to bridges.

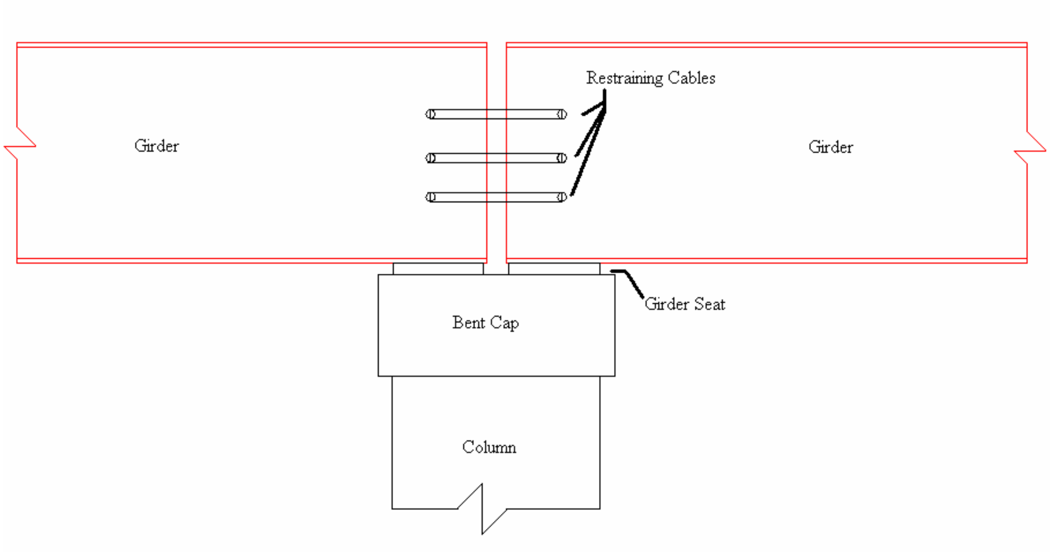


Figure 5.2 Cable Restrainer and Girder Seat Layout

Based on the results of the parameter studies described in the preceding sections, the amount of required seat extension can be determined for the most severe cases and recommended for bridges of varying criticality to prevent span loss. Seat widths for a steel girder on existing bridges are in the range of 16 inches (TxDOT, 2004). A summary of required seat widths for different systems is shown in Table 5.11. Typically for girders which do not experience failure, no seat extension is necessary, and the required seat width is less than 15 inches. To ensure that the cause of failure is not seating loss, it is recommended for large girders (e.g. those selected for large spans and designed and retrofitted for blast mitigation) that seat widths of 18 inches per girder are provided. The provision of this seat width may be in the form of a larger bent for new bridges, or in the use of seat extensions for retrofitted bridges. An alternative to seat extension retrofits would be provision of a cable restraint system tied to the bent cap or adjacent girders.

Table 5.11 Seating Width Requirements

Span	Scenario	Displacement	Required Half Seat Width
(ft)		(in)	(in)
80	Failure	41.9	1.8
120	Failure	94.2	6.2
160	Failure	167.5	14.7

5.4 SUMMARY OF BEST PRACTICES FOR BLAST MITIGATION

Several useful design techniques for blast mitigation were known prior to this research. The combination of that information with the specific parameter studies performed for this investigation allows for a set of best practices for blast mitigation for steel girder bridges to be assembled. These best practices include the use of ductile materials and connections, as well as redundant and systems which are over-designed compared to typical live and dead loads. These types of design philosophies will result in structures which are able to withstand overloads and maintain integrity through large deformations imposed by extreme events such as a terrorist attack. In addition to these essential elements to design, specific recommendations as to system configuration can be provided based on the performed parameter studies. In the previous sections of this chapter, it was shown that increased girder clearance, decreased deck thickness, and decreased girder spacing will minimize loads acting on girders as a result of a blast event. As expected, these reduced loads aid in decreasing midspan displacements and improving girder performance. These techniques are excellent options for new construction, but they are not likely cost-effective as a retrofit option for existing bridges. Parameter studies also demonstrated that increased section strength will offer relief from blast overloading. As a retrofit option, the addition of cover plates will provide strength increases, and for new designs, large section sizes and higher strength steels are recommended.

5.5 INFLUENCE OF COST ON RECOMMENDATIONS

The economic aspects of design changes or retrofits are a very important consideration in the provision of suitable recommendations for use by bridge engineers. Typically, bridges are designed to be efficient and cost effective. This concept must not be lost on a bridge designed to withstand overloads caused by terrorist attack. Clearly, major retrofit or design changes will have an effect on cost; however several of the investigated mitigation measures can be implemented with little to no effect on cost. As summarized in the section describing best practices for blast mitigation, measures which reduce the applied blast load will be amongst the most effective in preventing failure of a girder or girder system. These measures come with little associated cost increases.

Reduction in girder spacing will require additional girders. However, the girders in this case will be of a smaller size, and the reduced spacing will promote the use of a smaller deck thickness. These design changes will logically be utilized together, and increased cost will be minimal. Increases in bridge clearance will also serve to reduce applied blast pressures and impulses due to below-deck threat scenarios at a low cost. Increase in height of a pier system where no site restrictions prohibit this modification will be a very low cost design change. For the range of pier parameters considered in this research, the small amount of additional concrete and reinforcing steel required for a larger pier will cost little compared to the benefits gained by reduction in necessary girder cross-section size for blast resistance. Further research would be needed if very large clearances are available to determine if increases are warranted. Among the retrofits and design changes investigated in the parameter studies for this research, those with the highest associated costs deal specifically with the girder cross-section and material. In the studies discussed above, it is important to note both the large weight of the sections, and the large change in weight between cross-sections selected. Referring back to Tables 5.2 and 5.3, it can be seen that these increased sizes will require greater than usual quantities of steel, and therefore have significantly higher costs. Combining the cross-sectional information with cost information obtained from the West Point Bridge Designer 2004 software (USMA, 2004), a reasonable estimation of associated cost increases can be determined. Table 5.12 shows the cost data used for the cost analyses for this research. Prior to use of any recommendations for new bridge designs, the engineer should take into account more current and realistic data for determination of the use of larger section sizes and higher strength steels.

Table 5.12 Steel Prices Used for Cost Comparisons

Steel Type	Cost per lb
Regular Carbon Steel	$1.90
High Strength Steel	$2.40

Table 5.13 Cost of Girder Sections Studied

Section Number	Section Depth	Cross-Sectional Area	Cost per foot (Regular Carbon Steel)	Cost per foot (High Strength Steel)
	(in)	(in^2)	($/ft)	($/ft)
1	60	83.5	539.85	681.92
2	66	133.5	863.12	1090.25
3	72	168.0	1086.17	1372.00
4	72	183.0	1183.15	1494.50
5	72	203.9	1318.27	1665.18

Combining the information in Table 5.11 with the cross-sectional properties from Tables 5.2, a relative cost of each section, in each steel strength, can be generated. A summary of the cost of each section is shown in Table 5.13. The data above, combined with knowledge of the relative performance of each section (from Tables 5.5 and 5.6), provides the basis for a recommendation of section size and steel strength.

5.6 DESIGN CHANGES AND RETROFIT RECOMMENDATIONS

Each of the items investigated in this research has some value in improving bridge performance as discussed above. The portion of this report which outlines best practices in blast mitigation discusses the use of load reducing techniques and increase in section strength and stiffness to improve girder dynamic response. Since the purpose of this research is to provide design guidelines to engineers unfamiliar with blast design, design and retrofit options have been formulated into a set of performance-based design recommendations.

Performance-based design recommendation charts for steel girder bridges are shown in Figures 5.3 through 5.6. The chart is divided into different quadrants, and each quadrant provides guidelines for bridges of different criticalities. As discussed earlier, bridges are allowed to sustain different amounts of damage based on criticality. In addition to design change or retrofit information presented in this chapter, additional good practices such as improved lighting and security are included. These countermeasures are intended to be applied to bridges of any criticality, and implementation of these recommendations must be based on the risk manager's assessment of risk and availability of resources.

Category 1 (Very Important)
Concept: Each structural element designed to withstand 2 separate cases, large loads with repairable damage and smaller loads with negligible damage

Design Loads - Case 1 (Small-sized Loads)
Acceptable Damage - Case 1 (Small-sized Loads)
local deck failure, support system still intact with negligible damage
still capable of supporting design loads
no unrepairable foundation instabilities, no span loss

Design Loads - Case 2 (Large-sized Loads)
Acceptable Damage - Case 2 (Large-sized Loads)
local deck failure, support system still intact with minor damage
not capable of supporting design loads but easily repairable
no unrepairable foundation instabilities, no span loss

Design or Retrofit Options
Minimize girder spacing (recommended girder spacing of 8 feet)
Maximize above ground clearance of superstructure
Minimize deck thickness (recommended deck thickness of 10 inches)
Use of ductile connection details (if any)
Use of ductile steel (suggested A992) or higher strength
Use of minimum toughness standards for weld metal in any welding detail
Stiffener spacing such that full plastic capacity of girder sections can be realized
Stiffener detailing to account for the possibility of load reversal

Minimum girder depth 72 inches
If Span is ≤ 120 ft
Minimum flange width 28 inches
Minimum flange thickness 1.5 inches

If Span is > 120 ft
If Girder Spacing is ≤ 8 ft
Minimum flange thickness 1.5 inches
Minimum flange width 28 inches
If Girder Spacing is > 8 ft
Minimum flange thickness 2.75 inches
Minimum flange width 33 inches

Figure 5.3 Design Recommendations for Very Important Bridges

<u>Category 2 (Important)</u>
Concept: Designed to withstand smaller loads with repairable damage

Design Loads - (Small-sized Loads)
Acceptable Damage
local deck failure, support system still intact with minor damage
not capable of supporting design loads but easily repairable
no unrepairable foundation instabilities, no span loss

Design or Retrofit Options
Minimize girder spacing (recommended girder spacing of 8 feet)
Maximize above ground clearance of superstructure
Minimize deck thickness (recommended deck thickness of 8 inches)
Use of ductile connection details (if any)
Use of ductile steel (suggested minimum A992) or higher strength
Use of minimum toughness standards for weld metal in any welding detail
Stiffener spacing such that full plastic capacity of girder sections can be realized
Stiffener detailing to account for the possibility of load reversal

If Girder Spacing is ≤ 8 ft
Minimum girder depth 72 inches
Minimum flange thickness 1.5 inches
Minimum flange width 28 inches

If Girder Spacing is > 8 ft
Minimum ductile steel yield stress 75 ksi
Minimum girder depth 72 inches
Minimum flange thickness 1.5 inches
Minimum flange width 28 inches

If A992 steel used
Minimum flange thickness 2 inches

Figure 5.4 Design Recommendations for Important Bridges

Category 3 (Slightly Important)
Concept: Designed to withstand smaller loads with moderate damage

Design Loads - (Small-sized Loads)
Acceptable Damage
local deck failure, support system still intact with repairable damage
no more than one span loss
no unrepairable foundation instabilities

Design or Retrofit Options
Minimize girder spacing (recommended girder spacing of 8 feet)
Maximize above ground clearance of superstructure
Minimize deck thickness (recommended deck thickness of 10 inches)
Use of ductile connection details (if any)
Use of ductile steel (suggested A992)
Use of minimum toughness standards for weld metal in any welding detail
Stiffener spacing such that full plastic capacity of girder sections can be realized
Stiffener detailing to account for the possibility of load reversal

If Girder Spacing is ≤ 8 ft
Minimum girder depth 66 inches
Minimum flange thickness 1.0 inch
Minimum flange width 28 inches

If Girder Spacing is > 8 ft
Minimum girder depth 66 inches
Minimum flange thickness 1.5 inches
Minimum flange width 28 inches

Category 4 (Unimportant)
No Standard

Figure 5.5 Design Recommendations for Slightly Important and Unimportant Bridges

<u>Nonstructural Options for Improved Security</u>
(Any Bridge Criticality)

Planning and coordination measures to improve detection of and response to threats
Information control to prevent identification of system weaknesses
Improved lighting and sight cleanup to remove hiding locations for threat preparation
Increased standoff to below-deck bridge areas
Elimination of parking beneath bridges or on bridge decks
Police patrol or closed-circuit television monitoring
Emergency phones to report incidents or suspicious activity

Figure 5.6 Nonstructural Recommendations for Improved Security

5.7 COMMENTS AND ADDITIONAL INFORMATION

The parameter studies summarized in the tables shown previously in this chapter demonstrate some possible combinations of retrofits or design changes which offer blast mitigation. A full summary of all of the parameter studies performed for steel girders is provided in Apendix H. This Appendix more completely illustrates the trends in response results which provide the foundation for the conclusions reached in this research.

Results presented in this chapter focused on the performance of fixed supported centerline girders. Additional parameter studies were performed on simply supported girders and girders away from centerline. As discussed in Chapter 6, study of fixed systems will provide unconservative results for simply supported systems. Although results of fixed girder studies are not conservative for simple systems, the same trends in results are expected. Because the same trends in data exist, the same retrofit options are appropriate for improving girder performance. The significant difference between systems with different support conditions is that larger sections or more severe use of the recommended options (such as further decreasing girder spacing) must be implemented.

Analyses of girders away from centerline were carried out in the same manner as those for centerline sections. In general it was observed that for centerline girders that fail by a small margin, no additional girders failed, and if large displacements (much greater than failure) occurred at centerline, adjacent girders were found to fail. For this reason, results and trends for centerline girders are appropriate to generate recommendations as to appropriate retrofits or design changes. A summary of the analyses performed to verify behavior of simply supported girders, and those away from centerline is provided in Appendix H.

CHAPTER 6

Truss Bridge Investigation

6.1 OVERVIEW

Due to the relative ease of access to load-bearing members of a truss bridge, risk and safety assessment of these structures is a critical issue in the overall attempt to improve bridge performance to potential terrorist attacks. Several possible threat scenarios exist for a truss bridge including ones which are applicable to other bridge types such as vehicle-delivered explosives, vehicle impact, and hand-placed explosive charges. These threats pose a risk of potential loss of one or more truss members which in turn may cause immediate or progressive collapse of a bridge structure. In general, a truss bridge could be built in a large variety of configurations, and because of this great diversity, it is difficult to assess the exact risks of any one bridge under an array of possible attack scenarios. To provide a measure of a truss bridge's ability to withstand an attack, several different representative truss configurations must be examined to determine benefits gained in the event of member losses under a variety of possible circumstances.

6.2 SCOPE AND INVESTIGATION METHODS

A truss bridge's response to member loss could be affected by many parameters including the number of missing members, location of the missing members, the degree of redundancy of the truss system, truss element connection properties, and the overall geometry and loading of the truss. To investigate the effect of these parameters on a truss's ability to resist terrorist attacks, representative truss configurations were subjected to member removal and analysis under load to determine how remaining truss members were affected.

Analysis of these damaged trusses can be performed in a variety of ways. The purpose of these analyses is to investigate the possibility of progressive collapse of a truss bridge. This investigation requires an understanding of progressive collapse and its associated analysis methods. The American Society of Civil Engineers Standard 7-02 Minimum Design Loads for Buildings and Other Structures defines progressive collapse as ”the spread of an initial local failure from element to element, eventually resulting in the collapse of an entire structure or a disproportionately large part of it” (ASCE, 2002). Analysis of this type of event requires investigation of the performance of a structure not only to externally applied loads as with most analysis types, but rather the response of the structure under normal load cases such as wind, or dead and live loads with the removal of one or more members. Progressive collapse is a dynamic event involving the redistribution of internal member loads. Several options exist for progressive collapse analysis. The selected method can be either direct or indirect. With indirect methods, an attempt is made to provide sufficient overall structural integrity so that the potential for progressive collapse is minimized. No specific load case is considered with indirect methods. Rather, overall structural integrity is of primary concern. ASCE 7-02 presents general guidelines for improving structural integrity of building to aid in the prevention of progressive collapse, however some of these guidelines may be good recommendations for bridges as well. These guidelines include good plan layout, an integrated system of ties (providing a load path between structural elements to allow for redistribution to the strongest elements), load-bearing interior partitions, catenary action of the floor slab, redundant structural systems, ductile detailing, additional reinforcement for blast and load reversal, and compartmentalized construction

(ASCE, 2002). Direct methods, unlike indirect ones, take into consideration the behavior of a structure subjected to a localized load. Steps can be taken to protect individual elements deemed to be at risk so that component failure is prevented under the load cases considered. Alternatively, the performance of a structure as a whole can be considered in the event that a member or several members fail. Such an analysis seeks to determine whether or not the loads carried by the elements that fail can be safely redistributed to the remaining part of the intact structure. The analysis method can range from a simple linear-elastic static approach to a nonlinear dynamic method (Marjanishvili, 2003). Methods suggested for use in this regard by the Department of Defense in the Unified Facilities Criteria in UFC 4-023-03 range from the Alternate Path method which requires a structure to be capable of bridging over a missing element with only localized damage to the Specific Load Resistance method which requires that a structure be capable of resisting a specific threat (DoD, 2004). The type of investigation and analysis approach used is dependant on knowledge of loads which will initiate a collapse, and on required accuracy. The method may incorporate the use of the loads which initiate the first failure, or as is the case with this research, may be independent of the load which initiated the first failure and instead focus on the response of a system after a failure has occurred. This research uses a load path approach incorporating a static analysis procedure. This approach allows for the solution to be valid without knowledge of the exact hazard which causes an initial failure. For truss structures the potential for propagation of local damage into other parts of the structure ultimately leading to collapse can be prevalent (Malla, 2000).

In addition to exploration of the effect of truss properties on overall structural performance, the manner in which removed members are treated during analysis also requires investigation. The nature of the possible threat scenarios to a bridge in question suggests that member loss will be sudden, and therefore it is necessary to look at the effect of the rapid unloading of a damaged member or members. A comparison of the effect of applying a dynamic increase factor to the static force in a removed member and applying that force statically to the remaining structure with no accounting for sudden member unloading can be made. In effect each truss configuration and member removal is examined in two ways, once with a dynamic increase factor on removed member forces, and once with no increase factor. The two different analyses allows for a quantification of the effect of the dynamic nature of the unloading without performing an actual dynamic analysis. In the case of the analyses using the dynamic increase factor the initial member failure was consider sudden and dynamic in nature, but it is also true that subsequent failures will be sudden. It is because of this likelihood that the forces in members which fail as a result of the initial assumed failure were also magnified using the same dynamic increase factor. For the truss progressive collapse analyses presented in this research, a dynamic increase factor of two was used. Different increase factors could be considered reasonable, however the use of two is common practice. This value of dynamic increase is derived from basic structural dynamics. In the case of a simple mass and spring system with a statically applied force, the calculated maximum displacement would be F/k (force/spring stiffness). In the case of a simple mass and spring system with a dynamically applied impulsive force, the calculated maximum displacement would be $2F_o/k$ (two times initial force/spring stiffness). Comparison of a statically and dynamically applied force yields an increase of two times (Paz, 1997). The inclusion of this dynamic amplification factor is consistent with recommendation of the General Services Administration (GSA, 2000). The GSA has outlined procedures for progressive collapse which includes a linear static method applicable to low-rise buildings. Due to the large variation in truss configurations, and their potential complexity, the use of this increase factor on the removed member forces is intended to account for the dynamic nature of a structure's response to blast loading without the use of more sophisticated methods which are likely to provide results specific to one truss layout and attack scenario. The procedure used in this research for progressive collapse is demonstrated in Figure 6.1. This figure shows an intact truss, and then shows the same truss with a removed member and the applied force to demonstrate the unloading of that member. In Figure 6.1 the dynamic increase factor of two is applied, but as described the same scenario was investigated without an increase factor. The next portion of the Figure 6.1 illustrates failures which took place as a result of the assumed failure.

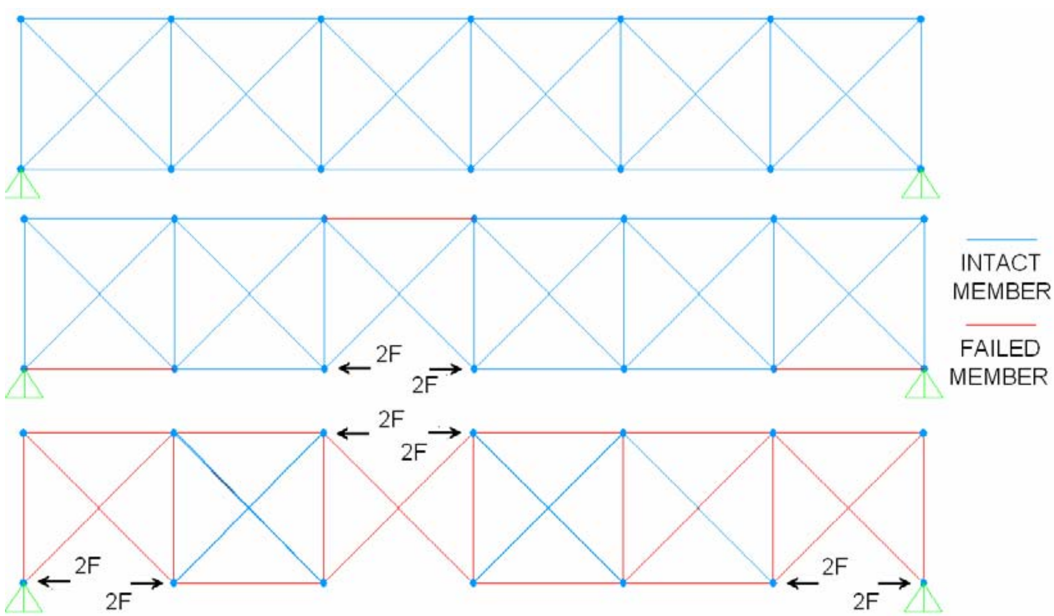


Figure 6.1 Truss Analysis Procedure Demonstration

In addition to investigation of unloading behavior of a truss, the redundancy and connection properties also require consideration. A truss will traditionally be designed as a system of pinned, two-force members, but because of detailing, truss connections may actually possess some amount of moment capacity which may have an effect on the response to member loss. Modeling of a truss separately with ideal pins as well as with moment resisting connections allows for comparisons to be made to quantify benefits gained from this connection capacity. The level of redundancy of a truss system may also influence the ability of the system to redistribute loads which will have a major impact on the truss response. Evaluation of the performance of different trusses with different degrees of indeterminacy to member loss will allow for some method to quantify of the benefits of a system redundancy for resisting collapse following the failure of one or more members.

6.3 ANALYSIS PROCEDURES AND OBSERVATIONS

The issue of redundancy within a truss system logically will have a large impact when comparing truss behavior after the loss of one or more members. The additional members providing this redundancy will allow for redistribution of loads which less redundant systems would be incapable of, and this ability for load redistribution may prevent other members from reaching their capacity and causing a failure that may lead to a progressive collapse scenario. As with other potentially important parameters under investigation, the method of analysis will affect overall performance of the truss system being studied. To provide a clear picture of a truss system's behavior, each example case was analyzed under simple removal of a member, and with a magnification factor of the missing member forces to account for sudden unloading of the member. The first truss investigated was one span of a three-span, statically determinate truss bridge used by the State of Ohio Department of Transportation. The truss is composed of rolled wide-flange shapes and miscellaneous channel sections, and has a span of 130 feet (see Figure 6.2 and Figure 6.3).

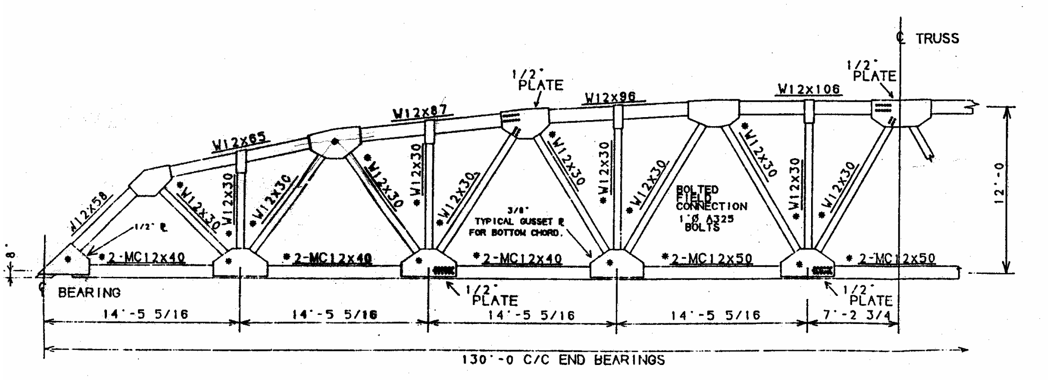


Figure 6.2 Ohio Truss Bridge Used for Investigation

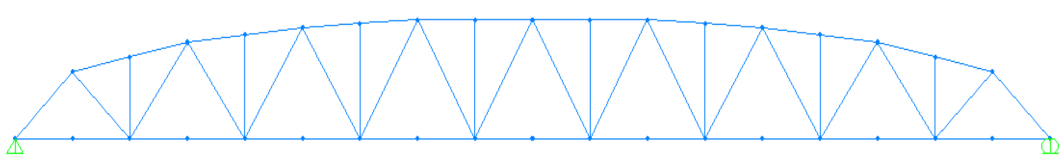


Figure 6.3 SAP Model of the Ohio Truss Bridge Used for Investigation

To perform an analysis, the truss was loaded with the AASHTO Bridge Specification (AASHTO, 2003) lane load of 0.1 kips per linear inch. In addition to this lane loading the self weight of the truss members was also considered to be present. Initial values of member forces were obtained, and it was verified that the structure was capable of carrying the code-specified loads. Members were checked to ensure that they provided adequate strength under axial load for pinned members, combined axial load and moment for partially or fully restrained members, and buckling for all compression members. The first case investigated for response to missing members was then analyzed with the removal of the bottom chord member at the centerline of the truss. A member near midspan was chosen because it is likely that use of a highly loaded member in this location will lead to collapse due to axial force and bending moment interaction. The specific member in question was selected because of its relatively high axial load compared to that of nearby members. This member removal, even without magnification for sudden unloading, leads to the failure of several adjacent members, causing total collapse of the truss. These failures occur because of large increases in the axial force in these nearby members causing buckling of a compression member, and yield of a tension diagonal. To determine sensitivity of truss failure from single member removal to location of the removed member, another case was studied with the removed member located only one bay from the truss support. Again the specific member in question was chosen based on its relatively high load compared to nearby members. As with the previous scenario, the truss failed immediately with or without the application of a dynamic increase factor for sudden member unloading. Immediate failure after the removal of only one member indicates that for a system with low redundancy, the truss cannot easily redistribute internal forces. Results of these analyses are shown graphically below in Figure 6.4 with intact members shown in blue and failed members shown in red.

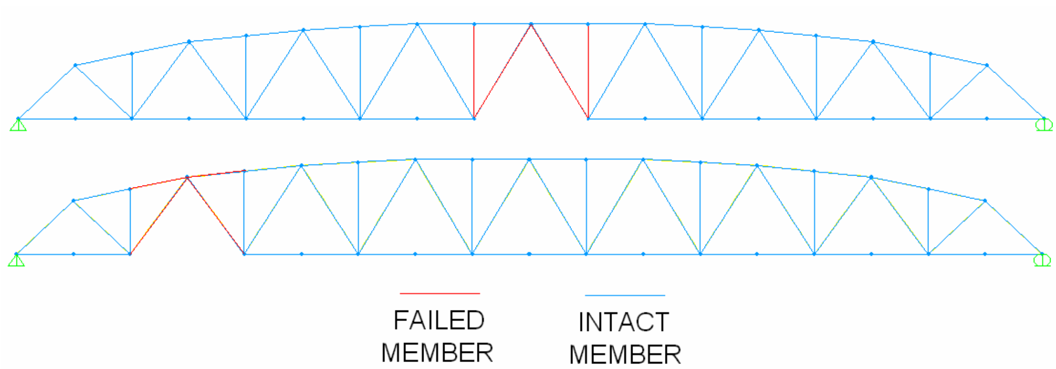


Figure 6.4 Truss Failures Due to Lack of Redundancy

To get a measure of the advantages to a more redundant truss system, a highly indeterminate structure of cross-braced bays was investigated. The truss has a 72-foot span with six bays, and initial member sizing was done using the same AASHTO lane loads used to determine initial forces for the statically determinate truss considered previously. The entire truss was composed of rolled wide flange shapes. Members were chosen based on the most critical bottom chord, vertical member, diagonal and top chord members, and not varied in size over the length of the span. A diagram of the truss configuration used to determine response of a redundant truss is shown below in Figure 6.5.

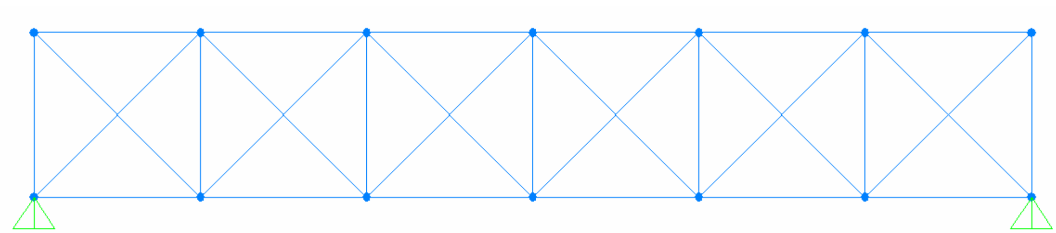


Figure 6.5 SAP Model of Redundant Truss Used for Investigation

Analysis of this truss was then performed after the removal of the bottom chord member from one of the two center bays. The specific member chosen for removal was selected because of its relatively high load compared to adjacent members and the similarity in location to the removed member of the Ohio bridge truss. The use of a member in similar locations in both trusses allows for the best comparison of the benefits gained by the additional redundancy present in this truss. Analysis of the system with a missing member and no adjustment for the unloading of the member indicates that no other members are overloaded to the point of failure. These results likely do not depict the true behavior of the truss, but it is interesting to note that the same scenario investigated in the less redundant truss led to immediate failure. This difference in behavior does give some indication of the expected gains from the use of a more redundant system. Results showing the removed member, and lack of other failed members are shown graphically in Figure 6.6.

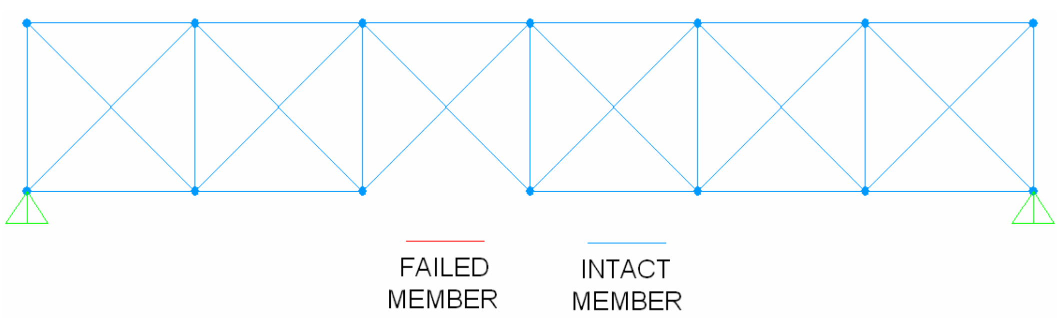


Figure 6.6 Redundant Truss Showing No Failures Upon Unmagnified Member Removal

The truss system was also analyzed in the same configuration with the same missing member, but with a magnification factor applied to the internal force in the removed member to account for the sudden unloading expected in a system subjected to a blast or impact. This analysis provided a more likely prediction of the actual truss behavior taking into account the sudden nature of the initial member loss. In this scenario, the initial member loss causes failure of several additional members. Results of the analysis with the initial member loss, and the progressive failure caused by this loss are shown graphically below. Although the analysis including the sudden member loss magnification factor causes additional member loss, it does not cause the truss to fail until the analysis is carried further to include the magnified effects of members lost due to the initial member removal. Although failure of the truss ultimately occurred, the fact that the failure required a large number of member failures to occur is another indication of the benefit of redundancy. The progression of the redundant truss failure under magnified member removal is shown below in Figure 6.7.

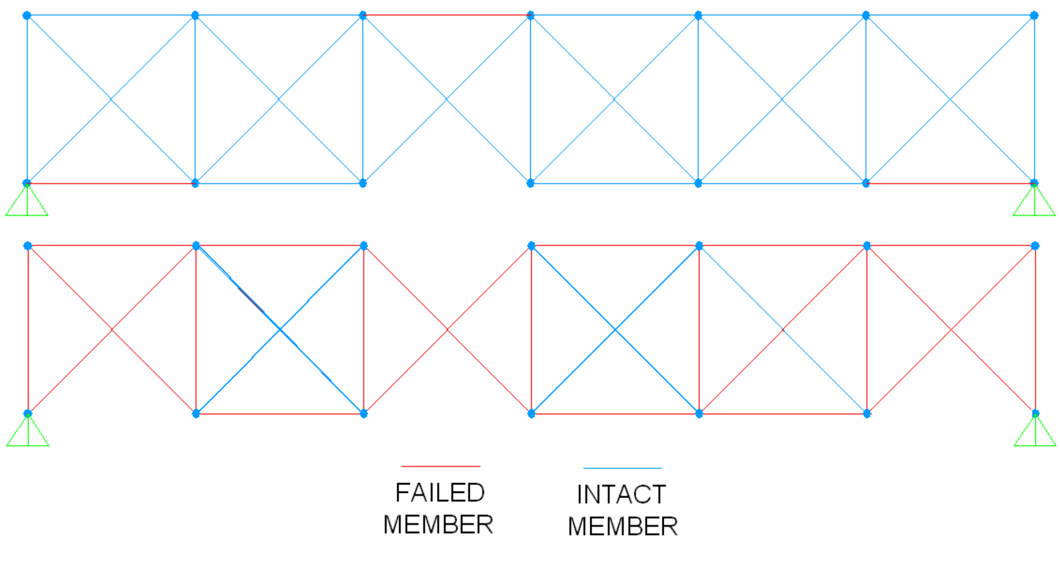


Figure 6.7 Redundant Truss Showing Progressive Failures Upon Magnified Member Removal

In addition to the case involving loss of a bottom chord member, a scenario involving loss of a diagonal member near the support was also examined. The results of this analysis showed that even with a highly redundant truss, it is possible to cause failure even without consideration of the sudden member loss magnification factor. For this truss, removal of the diagonal framing into the support causes buckling of the vertical member framing into the same location, which in turn causes the truss to collapse. The results of this analysis are shown graphically below in Figure 6.8.

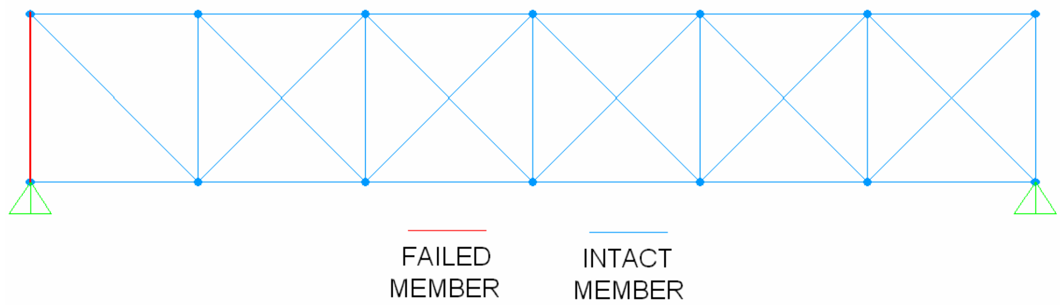


Figure 6.8 Truss Failure Due to Member Removal Location

6.4 Connection Assumption Effects

In addition to truss redundancy and location of the initial truss failure the effects of member connection properties was examined. As mentioned above a truss is typically designed as a system of two force members connected together using ideal pins. In actual construction it is more likely the case that the ideal pins assumed to be connecting truss members actually posses some amount of moment resisting capacity. To consider the effects of this moment capacity the analyses discussed previously were actually performed several times. Each truss was analyzed twice, once using pin connections, and once using fixed connections. The load distribution within a truss using both connection types was nearly identical. For this reason it was determined that connection type was not a significant parameter in the determination of internal force distribution, and therefore for progressive collapse analysis.

6.5 Initiation of Truss Progressive Collapse

The analyses above investigate the response of a truss to member loss and ensuing progressive collapse. The method used to perform this investigation was specifically chosen because is not dependant on the event which causes the initial failure. This flexibility allows for improvement of truss bridges to guard against a variety of terrorist attack scenarios. The cause of truss member loss could include hand-placed explosives used to cut a specific member, intentional vehicle collision with one or more members, vehicle delivered explosives, or a combination of explosives and collision. Also recommendations generated by this research could be used to improve truss performance in the event of an accidental vehicle collision with critical members, or even fires from traffic accidents or other causes.

6.6 TRUSS BRIDGE CONCLUSIONS AND RECOMMENDATIONS

It can be seen from the analysis results presented above that redundancy is a critical issue when discussing the vulnerability of a truss system to terrorist attack. It is not unexpected that a non-redundant system consisting of several large load bearing members would perform poorly with the removal of one or more of these members, and in turn a system with the ability to redistribute loads readily throughout a truss will perform well in the same situation. It is this observation that is at the root of terrorist threat mitigation for truss bridges. Certainly, it would be ideal to prevent damage to a truss member by means such as ensuring sufficient standoff, member jacketing or use of doubler plates, but in the event that that is not practical or even possible to prevent member failure, the best alternative would be to provide a truss system that is not highly dependant on the load carrying capacity of one member. It will be more difficult to remove a large number of truss members, and it will also be more difficult to isolate critical members in a redundant truss system. The recommendations for improvement of truss performance in the event of a terrorist attack of this research are first to make every effort to protect critical truss members, and secondly to make use of redundant systems to allow for improved truss performance. One method of protection of truss members is through the use of restricted access to load bearing members by using security guards or closed-circuit television. Increasing the strength of critical truss members is another alternative to increase the difficulty of damaging these members through blasts. This approach, however, is likely not an effective option because hand-placed charges are likely able to destroy truss members even with modest increase in strength. The most effective method in preventing progressive collapse of a truss bridge is to make use of a highly redundant system with the ability to redistribute internal loads in the event of a member loss. Although it may be possible to add additional members to an existing bridge, doing so is not likely an efficient retrofit option but rather a design recommendation for future truss bridges. Another important measure that can be used to reduce the risk of progressive collapse of a truss bridge is the use of tough, ductile steel and structural details which allow for ductile response. Again, these details and material changes are not necessarily retrofit techniques, but rather recommendations for future designs to ensure sufficient load distribution is not hindered by design. The most important aspect of mitigation of terrorist threats to a truss bridge is a redundant and ductile design which enables a truss to handle unexpected loads.

CHAPTER 7
Comments & Future Research Recommendations

7.1 OVERVIEW

The preceding chapters of this report have provided information on risk assessment and management, blast dynamics, blast analysis methods, and parameter studies to evaluate usefulness of structural retrofits and design changes for improving bridge response to terrorist attacks. This report provides recommendations as to effectiveness of various structural retrofits and design changes in reducing the threat of damage of a critical bridge structure to an acceptable level. During the course of research for this report, several areas of future research were identified. This chapter presents a discussion of items related to bridge security and blast-resistant design which should be investigated to either validate or enhance understanding of information presented previously.

7.2 GIRDER PERFORMANCE RELATIVE TO ADDITIONAL FAILURE MODES

Investigation of girder systems for this research considered both shear and flexural modes of behavior. Consideration of these failure modes requires the assumption that lateral torsional buckling and local buckling do not control response. This assumption is based on the provision of adequate lateral bracing and stiffeners allowing the development of full section capacity. In order to be consistent with this assumption, recommendations have been made in previous chapters to provide the necessary stiffeners and bracing. Girder bracing must consider the possibility of load reversals caused by blast loadings that are not typically considered in general design.

7.3 Composite Behavior of Girder and Deck Systems

In addition to consideration of alternate failure modes for superstructure systems, future research should seek to determine the behavior of girder and deck systems considering composite action. The current research has used a load path approach which assumes the deck sections to respond as a separate system from the girders. As was briefly discussed in Chapter 4, if analyses include various degrees of composite action, a large range of response can be observed. A review of the load path approach and a study of the appropriate amount of composite action to consider for bridge superstructure systems subjected to blast loads would validate the performed research. Physical investigation and study of components, perhaps in the form of full-scale testing, would be useful to evaluate analysis assumptions and methods. Research of this type would be particularly valuable because a large number of the prior investigations involving blast-loaded structures considers building component response. The scale of structural members and the methods for which these components carry load is much different for bridges, and this study could provide a measure of applicability of previous research for different structure types.

7.4 Behavior of Partially Damaged Girders and Truss Members

With regard to the response of truss bridges considered in this study, member failure was assumed, and the propagation of that failure through the structure was investigated. The initial failure of a truss member was assumed to occur as the result of a terrorist event such as a localized blast, debris from a large blast, direct cutting with counterforce charges or other means, or vehicle collision. It is possible that these types of events would lead to one or more member failures. In addition, it is also likely that members in close proximity to the member that was assumed to fail would also suffer damage (though not necessarily complete failure). A model for the behavior of partially damaged members would improve the understanding of the internal demand and capabilities for stress redistribution.

This research also considers blast loadings to girder bridges. Response to blast loading for these systems has been studied through examination of centerline girders. If these girders undergo large rotations and large displacements, it is reasonable to assume failure. However, the effect that centerline girder failure has on the overall performance of the deck and girder system, as well as the behavior of adjacent girders, was not studied. Thus, additional research is needed to characterize bridge response following the failure of a girder. In addition, investigation into the behavior of partially damaged members is also needed to determine how diminished capacity influences overall response. This information could be gathered through analytical studies and be applicable to the definition of component response for problems in other areas of blast research.

7.5 Evaluation of Dynamic Analysis, Nonlinear Unloading and Progressive Collapse

Examination of truss systems for this research was carried out through the use of a static analysis approach intended to approximate the actual dynamic response of a structural system composed of a large number of members assembled in a variety of geometries. While this approach is commonly used, it may not accurately capture the true dynamic and nonlinear effects of all systems. Research has previously been conducted in this area by Dr. Griengsak Kaewkulchai and Dr. Eric Williamson (Kaewkulchai, 2004), and it has been demonstrated that an equivalent approach using a constant multiple of unloading forces in a static analysis may be unconservative in some situations, and very over-conservative in other cases. Research to determine more appropriate approximation methods for progressive collapse, and consideration of dynamic and nonlinear unloading effects would validate methods used in this research and provide a foundation for progressive collapse analysis used in other areas of secure design. Research needs encompass progressive collapse of truss structures, and also the response of other structural systems such as girder bridges.

7.6 TRUSS MEMBER FAILURE WITH REGARD TO ALTERNATE FAILURE MODES

Chapter 6 of this report discusses the analysis of truss bridges. The analyses include consideration of compression member buckling, as well as axial-moment interaction to determine failure. It is possible that failure of truss members could include alternate modes of response not studied such as lateral torsional buckling, local buckling, and connection failure. Further research is required to evaluate the actual behavior of these members under blast loadings, in progressive collapse scenarios where load reversals may occur, and for cased in which internal redistribution of force is required. Research in this area could be closely related to evaluation of member behavior in unloading and progressive collapse scenarios.

7.7 EFFECTS OF IMPACT

This research has primarily focused on improving structural resistance to blast loadings. It is reasonable to expect that a structure which will perform well under a blast load will be more redundant and ductile, which will improve its response to other severe events such as impact or earthquakes. It cannot be assumed however, that these improvements will be sufficient to protect a structure completely from these events. For this reason, further research is required to investigate structural response to intentional vehicle impacts, and to combinations of impact, blast, and potentially sustained fires. Interaction of several severe and complicated loading scenarios would require detailed analysis of different failure modes and structural response. Research including these more complicated loadings could provide valuable information for structural hardening and improved performance, but would likely lead to expensive solutions because of the severity of these "rare event" type loadings.

7.8 COMMENTS

The above research topics indicate the need for improving the knowledge base for structural behavior under severe loading scenarios. Improving the general knowledge of bridge component response to blast loads will allow for the formulation of additional recommendations for retrofit and design which are applicable to the specific cases which occur in bridge engineering. It is important to recognize that bridges under blast loadings is a specific and complicated issue, and although benefit can be gained from study in related areas such buildings or structures under other extreme loadings, specific research applicable to blast loaded bridges is needed.

This research provides guidelines for design and retrofit of bridge structures subjected to blast loading. Techniques adapted from current blast design practice, primarily from building structures, have been used to evaluate structural performance. These current practices are accepted to be reasonable accurate, however because of the general lack of knowledge on some subjects assumptions must be made based on engineering judgment. The research topics included above are important to the improvement of future bridge engineering to resist terrorist threats. Despite the need for further research in some areas related to this study, the recommendations provided will lead to safer more resilient structures.

APPENDIX A
Substructure Modeling & Analysis

A.1 IMPORTANCE

To diminish the risk of damage to a critical bridge, it is important first to investigate potential terrorist courses of action that may have significant impact on the behavior of a bridge system. The performance of a bridge substructure to a blast or impact scenario is crucial to maintaining structural integrity of the superstructure throughout the same event and beyond. Thus, substructure components (i.e., bridge piers) necessitate careful analysis because of their importance to overall structural response of a bridge system.

As shown through previous risk assessment procedures, a column and bent system would most likely be attacked using vehicle-delivered explosives, hand-placed explosives or vehicle impact. For below-deck blast events, it would be typical to expect that these systems will be placed in flexure created by loading perpendicular to the longitudinal axis of a column. This extreme loading will also cause large shears to be developed as well as potentially large horizontal deformations. Depending on the standoff distance of a blast, it is also likely that spalling and cratering of concrete will occur, and for extreme loads such as hand-placed charges, or large magnitude close-in blasts on smaller diameter columns, complete penetration through the column, known as breach, may be possible.

A.2 CONNECTIVITY & AXIAL LOAD EFFECTS

The connectivity of a column and bent system to each other, as well as to a girder and deck system, is an important detail to consider when investigating bridge behavior under blast loadings. It is likely, particularly in Texas, that bridge girders are not connected to a pier in such a manner that uplift forces created by an under-deck explosion can create tension in the piers. If this uplift were possible, an investigation of the effects of this tension force on the performance of bridge columns and bent would be needed. In the models presented in this report, however, these effects are assumed to be of little importance and therefore are not considered. In the case of an above-deck explosion, the effects on the substructure are assumed to be negligible because lateral loads against the columns due to blast effects are small. The primary effect of above-deck blast loading on piers is increased live loads due to forces being transferred from the deck and girder system. The variation in axial loads due to above-deck loading, for the purposes of this research, was ignored. Ignoring the additional compressive forces in the piers, for most bridges, is conservative due to the fact that these members are quite massive and have much greater axial capacity than required to support gravity loads. This observation can be demonstrated by considering a column interaction diagram that shows section capacity on a plot axial load versus bending moment. Given that the applied axial load in typical bridge columns, relative to the column capacity, is below the balance point (Figure A-1) (Kim, 2003), a slight increase in axial load will actually improve section performance as the moment capacity will increase from the additional confinement of the compressive loads. Ignoring this aspect of behavior is conservative for predicting column behavior. The effect of varying axial load in a column is shown graphically in Figure A.1.

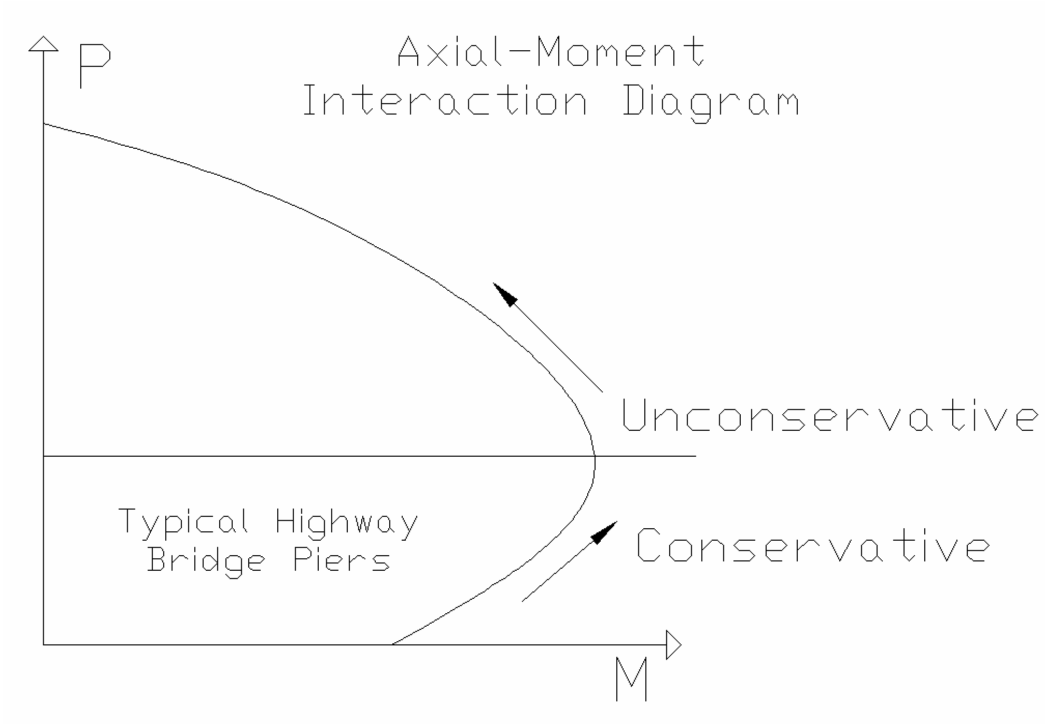


Figure A.1 Column Axial Load-Moment Interaction Diagram

A.3 SUBSTRUCTURE PARAMETERS

Models developed to study the performance of concrete pier systems are used to determine the effectiveness of retrofit options on mitigation of blast effects. A variety of parameters were selected in an effort to determine the most effective measures to improve pier performance. These parameters were chosen based on their influence on properties used within the component models and varied through an appropriate range to provide insight into the value of changing each parameter for a given column. The parameter studies for the pier systems included diameter, clearance (column height), longitudinal reinforcement ratio, spiral reinforcement, and use of steel and fiber reinforced polymer (FRP) jackets. In order to improve performance of pier systems, an investigation of parameters influencing shear strength, bending stiffness, axial strength and confinement was conducted.

An applicable range of pier parameters to be investigated was developed through review of the Texas Department of Transportation's website, and through coordination and discussion with Project Supervisor Dr. Eric Williamson and researcher Captain David Winget. The selected parameters were reviewed and approved by the Texas Department of Transportation to ensure that an appropriate and representative range of alternatives was used for analysis. Three column diameters, as well as three clearances, two reinforcement ratios, and steel and FRP wrap options were considered. A chart diagramming the parameters chosen for pier studies is available in Appendix E. It is important to note that varying of these parameters by coupling each of the different design change and retrofit options must be considered in order to fully evaluate the performance of the piers being studied. The purpose of this coupling is to provide an understanding of the potential increased benefits or limitations that result from the use of multiple retrofits. Furthermore, it helps provide a complete picture to show the improvement in performance for different retrofits to the varying substructures needed for different bridge design scenarios. The response to blast loadings resulting from either above- or below-deck explosions for the bent cap portion of the substructure was not considered because the bent cap was assumed to be sufficiently strong to carry the additional loads. Columns of these systems were assumed to be the more critical members and were the focus of the analytical studies.

A.4 MODELING APPROACH

There are a variety of different analysis approaches that could be utilized in the investigation of bridge components, and in this specific case, bridge substructure. The degree of accuracy is dependent on the analysis method, and it is important to choose an appropriate analysis technique based on problem definition. For this particular problem, it is important to balance quality of analysis and results with the need to investigate a large number of parameters and system properties. Due to the need for a large number of analyses and the fact that a relative comparison of the effectiveness of each retrofit or design option may be the most important data, a single-degree-of-freedom (SDOF) nonlinear dynamic analysis was chosen for reasons discussed in the previous chapter. This approach allows for a broad range of parameters to be used, and also provides an acceptable level of accuracy and consistency without requiring significant computational resources. Notably, this approach to analysis represents the state of practice in the design of structures to resist blast. Again the main purpose of these analyses is to provide a relative comparison of the effectiveness of each retrofit, or combination of retrofits and design changes.

A.5 MODEL & LOAD PROPERTY DETERMINATION

The approach chosen for flexural analysis involves the determination of the strength and stiffness properties of a column as a member loaded perpendicular to its longitudinal axis (i.e., behavior as a beam-column). Because axial effects are considered negligible for typical bridge systems (see discussion above), the columns of the pier can be analyzed as flexure dominated. The manner in which the member properties are obtained for analysis and the factors considered during this determination are discussed later in this chapter. It is important to note, however, that the properties of the columns are obtained while the column is under an appropriate amount of axial load. This point is important because, as seen previously, the axial load which the column is under has an influence on the moment capacity of the section, and will therefore influence its response to blast loads. For a column in a highway bridge, it would be typical to expect a low level of axial load compared to its maximum load carrying capacity. For the purpose of the analyses considered in this research, columns were assumed to carry ten percent of their axial load capacity in the absence of bending moment. This level of load is consistent with the range of typical axial load values for bridge columns (Kim, 2003). The value of ten percent was chosen because it was assumed that the dead load of a supported girder system and bent cap would provide at least this much load. The level of axial load in a bridge column would likely be greater than this amount, however this value was chosen to remain conservative if less than the expected amount of load is present. A decrease in axial load in this region of the column axial-bending moment interaction curve will actually lead to decreased moment capacity. Furthermore, because this value falls below the balance point, it remains a conservative choice when considering an increase in axial load will actually improve flexural strength.

The initial presence of axial load in a column under analysis is an important factor to consider due to its effect on cross-sectional behavior. The variation in axial load, however, may be ignored to provide conservatism as explained previously by Figure A.1 and with the assumption that the actual axial load present in a column would be some reasonable amount greater than ten percent. After determination of the single-degree-of-freedom flexural properties of a column, different blast loadings to which the system is to be subjected were generated using the CONWEP (USACE, 2003) computer software. This software has the ability to produce a detailed pressure-time distribution over the surface of the column subjected to a blast of varying type, magnitude, and standoff distance. The software is also capable of calculating an equivalent uniform pressure and time history. Because CONWEP doe not account for blast wave reflections which have the effect of increasing the impulse acting on the column, load magnification must be performed. For the purpose of this research the assumption was made that the peak pressure would be doubled. This pressure doubling has the effect of doubling the impulse, the critical parameter for this type of dynamic system under blast loads, applied to a pier. The value of two times the original pressure was chosen because of the unknown nature of the below deck environment. It is this magnified equivalent uniform pressure that was used in these analyses to determine the behavior of the column under a given blast load. The equivalent uniform pressure and impulse calculated within the software are determined using an equivalent work approach. The CONWEP software uses the full pressure and impulse distribution over the reflecting surface in question, in this case the area defined by the pier height and diameter, and an assumed displaced shape. This displaced shape within CONWEP accounts for support conditions on either two or four sides. Since this research is concerned with pier systems support conditions will be used on only two sides. These support conditions can be selected as either fixed or free. The boundary conditions selected for this research are discussed later in this chapter. The displaced shape, pressure, and impulse distribution are defined in the two dimensions of the reflecting surface. Figure A.2 is an illustration of the distribution of impulse over the surface of a column. A similar distribution of pressure would be expected to be acting on the same surface.

The equivalent impulse and pressure are then determined by integrating over the area either the pressure or impulse function multiplied by the assumed displaced shape, and then dividing by the integral over the area of only the assumed displaced shape. The formulas used within the CONWEP software are given by Equations A.1 and A.2. In these equations, P_E is the equivalent pressure, $P(x,y)$ is the spatial distribution of pressure, I_E is the equivalent impulse, $I(x,y)$ is the spatial distribution of impulse, and $\Phi(x,y)$ is the assumed displaced shape of the region under investigation.

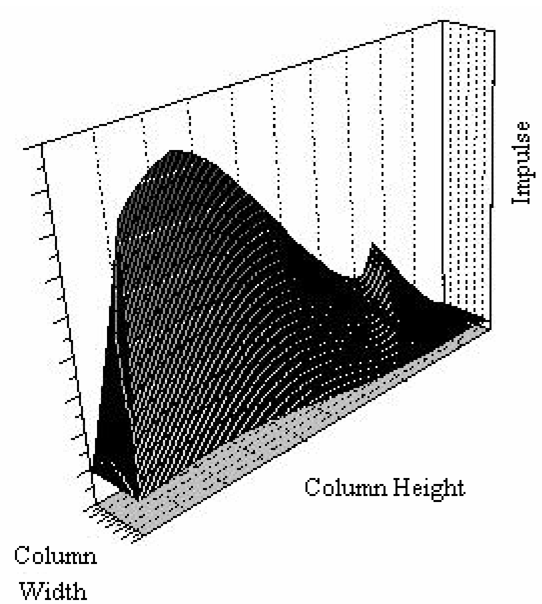


Figure A.2 Impulse Distribution Over a Column

$$P_E = \frac{\int_A P(x,y) \cdot \phi(x,y) \cdot dA}{\int_A \phi(x,y) \cdot dA} \qquad \text{(A.1)}$$

$$I_E = \frac{\int_A I(x,y) \cdot \phi(x,y) \cdot dA}{\int_A \phi(x,y) \cdot dA} \qquad \text{(A.2)}$$

Validation of the use of this equivalent load and this nonlinear SDOF dynamic analysis technique for columns was performed by investigating the response of a steel wide flange shape of representative column length to a blast load using the ABAQUS (ABAQUS Inc., 2003) computer software. Using the BlastX loads acting on the representative column, ABAQUS MDOF results were obtained which correlated nearly identically to SDOF results using the CONWEP equivalent load. The setup, calculations, procedure and results of this analysis and comparison are provided in Appendix F.

The single-degree-of-freedom dynamic analyses were performed on software developed specifically for this project. The computer program uses a Newmark time-stepping procedure to evaluate the nonlinear response of a single-degree-of-freedom system subjected to a forcing function that varies in time. The book "Introduction to Structural Dynamics" written by John Biggs (Biggs, 1964) and the Army Manual TM5-1300 (Department of the Army, 1990) were used to calculate properties of beam systems under different loading and boundary conditions for use in single-degree-of-freedom analyses as described in the previous chapter. A chart showing several systems and their corresponding properties, as well as an explanation and example of how these properties are calculated, are included in Appendix C. As mentioned previously, a uniformly distributed load is used in this analysis procedure, and the columns are assumed to be fixed at the base and pin-connected to the bent cap members. The use of a fixed-base boundary condition comes for the assumption of connectivity, through the use of continuous longitudinal reinforcement, with the foundation at the base of a stocky member. A pin connection to the bent cap is assumed because of this same continuity, but with an apparent allowed rotation. These assumptions define the system used and allow for calculations based on provided values; the important parameters for these analyses are given in Table A.1 below:

Table A.1 Single Degree-of-Freedom Analysis Parameters

Boundary Conditions	Loading Type	Elastic Stiffness	Elastic Limit	Elastic-Plastic Stiffness	Elastic-Plastic Limit	Plastic Stiffness	Load-Mass Factor
Fixed-Pinned	Uniformly Distributed	$\frac{185EI}{L^3}$	$\frac{8M_p}{L}$	$\frac{384EI}{5L^3}$	$\frac{12M_p}{L}$	0	.78

As is evident in the above table, calculation of system properties requires information about plastic moment capacity, moment of inertia, and Young's modulus. Due to the many different sections and retrofit options being considered for this research, it would be inconvenient and difficult to calculate these properties without the aid of computer software. In this case, RCCOLA (Inter-Tech Engineering Inc.) was chosen to perform cross-sectional analysis to provide the flexural stiffness and the plastic moment capacity of the piers being analyzed. RCCOLA has the ability to account for many factors in determining the cross-section behavior of a column including confinement effects and axial load. It allows for input of cross-section shape, reinforcement pattern and amount, spiral or other shear reinforcement amount, and material properties. The basic inputs were determined for each bridge column to be examined, with each section representing a different combination of parameters being studied for the current research. The RCCOLA program was then used to generate a moment-curvature relationship for both the initial unconfined section and for the confined core section of concrete. The actual response of a column section depends upon the behavior of the initial cross section and just that of the confined core. Only the initial portion of the curve was used to identify important parameters discussed in the next section. This portion of the curve is used because it defines the peak flexural capacity of the section as well as the actual flexural stiffness of the section under load. Information obtained as output from RCOLA was used in conjunction with dynamic increase factors accounting for strain rate effects of blast loads to define the cross-section properties used in analysis. Dynamic increase factors are multipliers of material allowable stress limits. The increase multipliers are a commonly used simplifying method to account for material strength increases due to the effects of high strain rate on a material. For this research a dynamic increase factor of 1.15 is used to modify the compressive strength of concrete (ASCE, 1997). A typical moment-curvature relationship given as RCCOLA output and the expected overall column response is shown in Figure A.3.

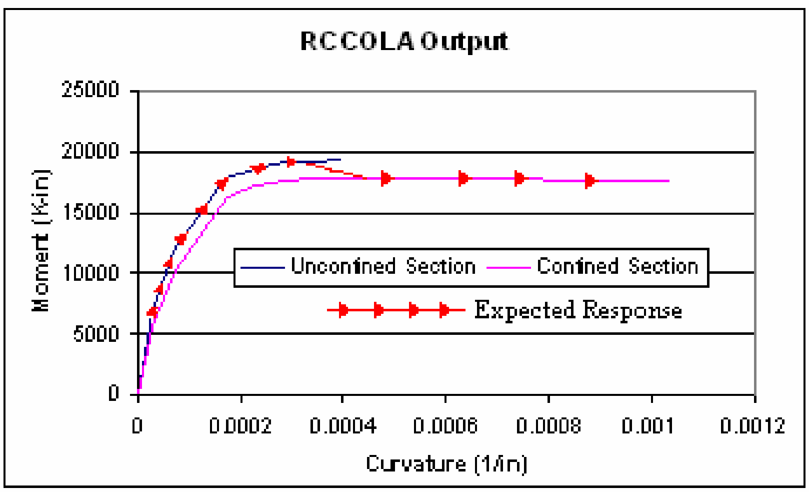


Figure A.3 RCCOLA Moment-Curvature Relationship Output

The shape of the moment-curvature diagram for a concrete column can be seen in the previous figure. This information is useful for identifying properties necessary for calculation of the overall column stiffness and strength parameters from Table A.1. In particular, the slope of the moment-curvature diagram is equal to the stiffness parameter of Young's Modulus times the moment of inertia, and the value at which the diagram reaches its maximum strength and begins to displace with very limited change in bending moment can be taken as the plastic moment capacity of the section. Identification of these values is somewhat subjective; however it is most important to obtain a relative comparison of retrofit performance so the consistency used in property determination should provide sufficient accuracy. RCCOLA typically outputs nine to ten data points along the moment-curvature relationship, and after four or five data points, the curve becomes nearly horizontal. The transition point between the horizontal portion of the curve and the portion with the positive slope is assumed to mark the end of the region that characterizes elastic response. This point was used, along with the preceding data points, to define elastic stiffness. A linear regression intercepting the origin of the moment-curvature plot and including data up to the end of the elastic region was conducted to calculate the slope of the elastic portion of the plot. This slope is used as the stiffness parameter EI for system property calculations. At some point, typically data points seven and eight, the moment capacity of the section levels off completely. It is this value that is taken as the plastic moment capacity of the section. These section properties, boundary conditions, and loading type allow for definition of the column behavior and calculation of all of the necessary properties for dynamic response analysis.

Although the typical method for determination of properties is discussed in the previous section, it is also important to discuss particular information relating to the calculations and methods of input into the computer software developed for this research to determine the section properties for different retrofit options. In particular, the method in which longitudinal reinforcement, steel jacketing and confinement is dealt with, and investigation of shear strength, must be examined and explained. The RCCOLA computer software uses two different types of longitudinal reinforcement. The program input allows for what it calls *primary reinforcement*, and also *secondary reinforcement*. In RCCOLA, primary reinforcement is defined as typical longitudinal bars which benefit from confinement effects. This reinforcement is used in determining the strength of the section being analyzed. Secondary reinforcement is longitudinal steel used in strength calculations but not accounted for in the confinement model used within the program. Through discussion with Dr. Richard Klinger (Klinger, 2003), it was determined that it would be appropriate to treat any typical longitudinal steel as primary confined reinforcement, and to treat any jacketing steel which would be contributing to both confinement for post yield performance and strength of the section as secondary unconfined reinforcement. This approach is appropriate because, as seen in Figure A.2 above, confinement effects become most important after the elastic stiffness range and into the transition to the confined core section of the moment-curvature response. In effect, the steel jacket would be providing confinement to improve after-yield performance if it were considered as primary confined reinforcement. The use of jacketing steel as secondary reinforcement is appropriate since information about post yield behavior is not used to determine required system properties (elastic stiffness and moment capacity) so that additional confinement effects which it provides are not necessary. The important contribution of the jacketing steel is the added strength to increase moment capacity and increased stiffness based on the increase in longitudinal steel from the actual jacket cross-sectional area. Because of the available input options for the RCCOLA software, actual steel jackets are not available, so this modified input route must be used. Due to the available location for placement of secondary reinforcing steel and the shape of the steel jacket, it was deemed appropriate to use only a portion of the total steel jacket cross-sectional area as secondary reinforcing steel. An investigation into the effects of varying the amount of steel used from the jacket to provide additional unconfined reinforcement was performed. The results of the analyses using different percentages of steel and corresponding strengths and stiffness can be viewed in Appendix G. RCCOLA developed moment-curvature relationships for column cross sections containing amounts of jacketing steel used as secondary reinforcement ranging from 0% to 100% of its actual cross-sectional dimensions were examined. The grouping of data for amounts of included steel between 30% and 80%, and reasoning based on the amount of steel away from the neutral axis, led to the assumption that fifty percent of the cross-sectional area of jacketing steel should be used as unconfined reinforcement to increase section strength, stiffness, and resistance in dynamic analyses. As mentioned previously, all necessary system parameters for flexural response calculation can be determined, and the structure portion of the problem can be defined. It should be noted that the analysis procedure outlined in this chapter refers only to flexural response and for a more complete understanding of a pier system's response to blast loads a model which includes concrete spall or complete breach of a column as well as direct and diagonal shear must be considered.

A.6 PERFORMANCE-BASED STANDARD USE FOR SUBSTRUCTURES

The remaining information for calculation of responses and relative effectiveness of different column designs and retrofits comes from the blast loads acting on the system under investigation. As discussed earlier, the most critical scenario for substructure elements, particularly columns, is a below-deck explosion causing a force perpendicular to the member's longitudinal axis and an increase in bending as well as transverse and diagonal shear stresses in the column. The below-deck explosion scenario has associated with it large variations in load magnitude and impulse depending upon the specific details of the threat being considered. For the purposes of this research, this threat definition is combined with a set of performance-based standards of response used to evaluate retrofit effectiveness and to provide recommended courses of threat mitigation. The method chosen to deal with the large array of possible threats and performance requirements determines the way in which information will be presented to design engineers for use in retrofit or design improvement of new structures. This topic is of central importance to this research and will be discussed in detail in the following section of this chapter, as well as others within this report. Information on the development and importance of these performance-based standards and design recommendations can also be found in the thesis prepared earlier in this research by David Winget entitled “Design of Critical Bridges for Security against Terrorist Attacks” (Winget, 2003).

In order to incorporate the criticality of a bridge with the expected magnitude of a potential terrorist attack into a set of guidelines for design engineers, the concept of performance-based standards was developed. These performance-based standards create categories of bridges based on criticality and then set a standard of required performance for different specified loads. The categories of bridges chosen for this research range from one (very important) to four (unimportant), and a performance standard is set for each bridge under a large or relatively small blast load. This required performance will allow for the assignment of specific mitigation techniques, as well as design and retrofit recommendations, to meet performance requirements.

For this research, both large and small explosive events were considered, and different structures were required to meet different performance levels based on these different loadings. Specific charge weights are not included in this report for security reasons. Maximum and mid-size credible load scenarios were formulated by accounting for the yield of explosive materials likely to be used by terrorists in conjunction with the payload capacity of different trucks. Hand-placed scenarios were based on the amount of explosive a person could carry over an extended distance. The large event is only to be considered for the most critical bridges, and damage limits appropriate for an attack of this magnitude have been assigned. These values were chosen through coordination with the Texas Department of Transportation, and through discussion with Project Supervisor Dr. Eric Williamson, Project Advisor Kirk Marchand and researcher David Winget. The large event considered in this research does not necessarily represent the absolute maximum event that a structure could possibly be subjected to, but rather was selected to balance risk and cost. Damage levels were first taken from Conrath (Conrath, 1999) and modified through discussion with the Project Advisory Panel to be applied to specific bridge components. The deformation limits used for the pier systems discussed in this chapter are shown below in Table A.2.

Table A.2 Deformation Limits Used in Pier Analysis

Failure Deformation Limits of Concrete Piers	
Event Magnitude	**Limiting Deflection/(Length/2) Ratio**
Large	10%
Small	6%

All structures are subjected to a small event, and based on relative importance, have been assigned acceptable performance standards. Figure A.4, taken from the thesis written by David Winget (Winget, 2003), provides an example of a typical categorization of criticality, list of acceptable damage, and design loads for bridges within the scope of this research.

Performance Based Design Standards for Bridges (Terrorist Threats)
<u>Category 1 (Very Important)</u> Concept: Each structural element is designed to withstand 2 separate cases, large loads with repairable damage and smaller loads with negligible damage.[1] **Design Loads – Case 1 (small loads):** "most-likely" threat scenarios using the following at worst possible locations for each structural element being designed: mid-size truck bomb [2] mid-size hand emplaced explosive scenarios mid-size static load for vehicle impact scenarios **Acceptable Damage – Case 1 (small loads):** local deck failure; support system still intact with negligible damage; truss / cables / piers *still capable of supporting design loads when considering structural redundancy*; no unrepairable foundation instabilities and no span loss; steel girders < 5% max deflection to length ratio, reinforced concrete girders < 4% **Design Loads – Case 2 (large loads):** "most-likely" threat scenarios using the following at worst possible locations for each structural element being designed: large truck bomb large hand emplaced explosive scenarios large static load for vehicle impact scenarios **Acceptable Damage – Case 2 (large loads):** local deck failure; support system still intact with minor damage; *not capable of supporting design loads but easily repairable*; no unrepairable foundation instabilities and no span loss; steel girders < 12% max deflection to length ratio, reinforced concrete girders < 8%
<u>Category 2 (Important)</u> Concept: Designed to withstand smaller loads with repairable damage. **Design Loads –** Same as Category 1, Case 1 **Acceptable Damage –** Same as Category 1, Case 2
<u>Category 3 (Slightly Important)</u> Concept: Designed to withstand smaller loads with no more than one span loss. **Design Loads –** Same as Category 1, Case 1 **Acceptable Damage –** no more than one span loss (no progressive collapse)
<u>Category 4 (Unimportant)</u> No standard

Note: 1. Design explosive loads for some Category 1 bridges may need to be increased based on a detailed threat assessment.

2. Exact design bomb sizes have been omitted for security reasons.

Figure A.4 Sample Categorized Design Recommendations (Winget, 2003)

The ductility limits provided in Figure A.4 were taken from those proposed in ASCE's "Structural Design for Physical Security – State of the Practice" based on typical structural members' observed level of damage under blast loads for a given deformation (Conrath 1999). For this specific portion of the current research, these deformation limits were modified slightly to account for the fact that the data reported in the ASCE document were based on research conducted for buildings. Due to the differences between expected behavior of bridge systems in comparison to typical buildings because of such parameters as span length, axial load acting on columns, etc., the deformation limits selected for the current study attempted to account for the expected deformation capacity of bridge components. The values selected were estimated based on engineering judgment and discussions with the Project Advisory Panel. The limits selected will likely require modification at a later date as data become available on the performance of bridges under blast loads. Because such data are not currently available, experience and judgment were used to establish appropriate limits. The deflection to length ratio limits used to define failure of a component for large loads was taken as 10%, and 6% was used for small loads. These limits are used for all bridges, and retrofits and design changes that meet the specified levels of acceptable damage for each criticality level are selected based on the results of the pier analyses. These values will vary in other portions of this research depending on material type, member type, and boundary conditions. To correlate with these deflection limits defining failure, physical descriptions of expected or acceptable damage are provided. Again, the purpose of these retrofit and design recommendations is to allow a design engineer unfamiliar with blast effects to take actions to mitigate the risk to a structure from terrorist attack. The specific actions to be taken, although certainly also based on available resources, will be presented in the next chapter based on bridge criticality and potential threat magnitude. The recommendations will provide not only structural retrofit options and design guidelines, but also suggest other measures such as access control, lighting, and security measures that will provide the design team with a complete resource for terrorist threat mitigation.

A.7 SUBSTRUCTURE IMPACT CONSIDERATIONS

In addition to damage to substructures from blast loadings, it is important to consider the potential of a terrorist attack to include, or consist solely of, a vehicle collision. The AASHTO Bridge Specifications (AASHTO, 2003) currently include a design load that must be accounted for in structures that do not provide an adequate barrier. This design load is based on full-scale crash tests involving an 80 kip tractor-trailer, and for piers must be considered as a 400 kip point load. This load is based on research of the deflection of tractor-trailers traveling at a moderate speed away from crash barriers, and not for head-on collisions with piers. It is for this reason that some modification to this load will be necessary to account for intentional high speed head-on vehicle collisions. Further research of this topic is required, but in the interim, it is the recommendation of this research that for piers unprotected by adequate barriers, use of the 400 kip collision load with magnification for intentional head-on collision under the designer's discretion would be most appropriate.

A.8 SUBSTRUCTURE ANALYSIS OUTLINE

This chapter has discussed information pertinent to analysis of substructure elements including a detailed layout of the analysis procedure and system property calculations. The procedure can be seen in diagrammatical form in Figure A.5 which illustrates the collection of data and analysis approach.

There has also been discussion of standards of performance for these components, and a presentation of what will be provided to engineers for use in blast design and threat mitigation. The next chapter presents results of these analyses and offers specific design recommendations and guidelines. There is also information presented about other nonstructural options to mitigate risk to a bridge and its substructure. The information in these chapters is closely related and should be viewed in conjunction with information presented from previous research on blast effects on substructures, pier and bent cap dynamic modeling, and performance-based standards for bridges in the thesis written by David Winget (2003).

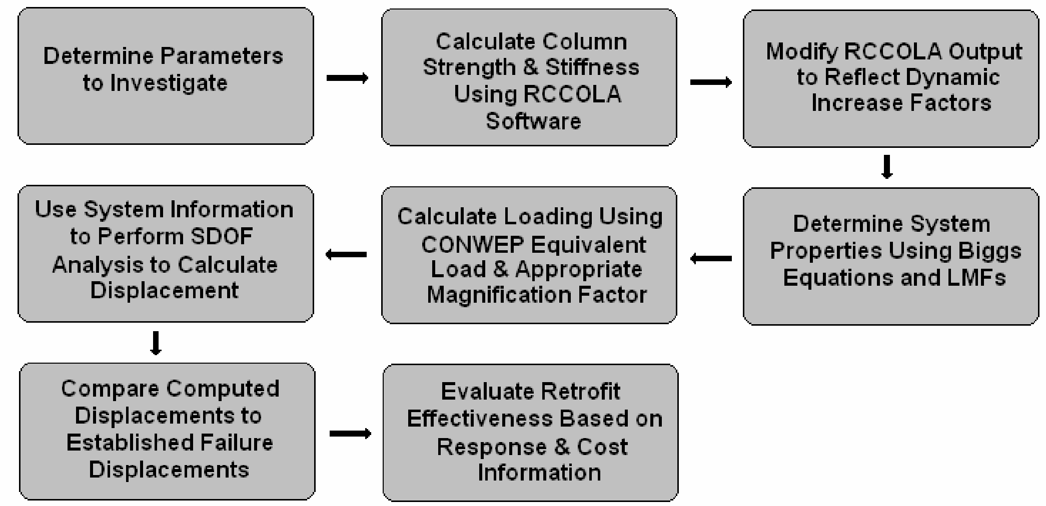


Figure A.5 Pier Analysis Procedure Outline

A.9 ADDITIONAL RESEARCH TOPICS

The analysis procedure described in the previous sections of this chapter provides a foundation for investigation of pier retrofit options; however there are other elements to this investigation that must still be considered. The analysis method described in this chapter deals only with flexural response of pier systems. Due to the short span length and large flexural stiffness of these systems shear response will play a very important role in retrofit or design change recommendation. Additional investigation of the effects of spall, and its corresponding reduction in pier cross-section is required. This determination is necessary due to the potentially large amount of concrete lost from a pier cross-section in the event of close-in explosions. Also, the benefit of fiber reinforced polymer wraps in improving ductility and overall column performance must be explored further. This exploration will require more knowledge as to the performance of FRP wrapping in the presence of blast projectiles, and of the overall effect of this wrapping as it relates to improving column performance under dynamic loads. Further investigation of the pressure and impulse magnification effects caused by reflections in a below-deck environment is necessary, as well as consideration of the response to blast of large-scale hollow piers such as those present in large overpass structures. Several of these research topics are currently under review, and results, observations, and recommendations to designers will be presented in an additional report accompanying this thesis.

APPENDIX B

Substructure Analysis Results and Recommendations

B.1 INTRODUCTION

Appendix A of this report outlines the procedure used to compute the flexural response of pier systems to blast loadings and also describes how to determine the parameters that are required to carry out the calculations. Appendix A also provides information on the use of performance-based standards to create recommendations for appropriate retrofits or potential design changes to pier systems. This chapter presents the results of the completed analyses and gives recommendations to improve column performance under blast loads. It is important to note that information presented in this chapter is derived specifically from flexural analysis of pier systems, and the recommendations given should be viewed in conjunction with recommendations provided in an additional report summarizing the findings of ongoing research. This additional research includes important factors such as concrete spall, and also alternate failure modes such as complete breach of a column cross section, diagonal shear, and direct shear, which are likely to be of critical importance to pier systems subjected to close-in blast loads.

B.2 SUMMARY OF RETROFITS AND DESIGN CHANGES INVESTIGATED

The analyses performed for this research considered the flexural response of pier systems. Because of this focus, the retrofits and design change recommendations were based on improving this mode of response. As discussed in Appendix A, important properties for flexural behavior include the ultimate moment capacity of a cross-section and the flexural stiffness (*EI*). Also important to the flexural response of a pier section is the severity of blast load to which it is subjected. Severity of a blast load acting on a surface is in part a function of geometry, and because of this, consideration must be given to the size and shape of a pier under analysis. This research focuses on finding combinations of the above parameters which effect flexural response, and in turn overall pier performance, that provide the most cost effective mitigation of severe blast events. For this portion of the research, column height, diameter, longitudinal reinforcement ratio, and amount of steel jacketing were considered. As was previously discussed, other significant modes of column response aside from flexure must be considered for safe design. Ongoing research by Captain Dave Winget (as yet unpublished) should be reviewed for additional design and retrofit recommendations for protecting piers against spall, breach, and shear modes of response.

B.3 Design or Retrofit Options Improving Pier Flexural Response

Consideration of only flexure as a mode of failure leads to recommendation of retrofit or design options which provide the largest stiffness and strength increases at the lowest cost. Although recent research (Winget, unpublished) has shown that improvement of flexural behavior may not be necessary to improve column behavior to blast loads because other modes of response may govern, retrofit options presented here may still have value. Investigation of flexural response of piers to blast loads without consideration of spall or breach of the cross-section demonstrates benefits of increased column size to improve performance. Although increase in cross-section size would increase flexural stiffness, and therefore require additional shear reinforcement, it is likely still necessary to provide low-cost resistance to cross-section loss. The improved flexural performance of a column with varying diameter is shown in Figure B.1. This figure shows the peak midspan displacement in inches for piers of different diameters subjected to a blast of consistent charge weight and standoff.

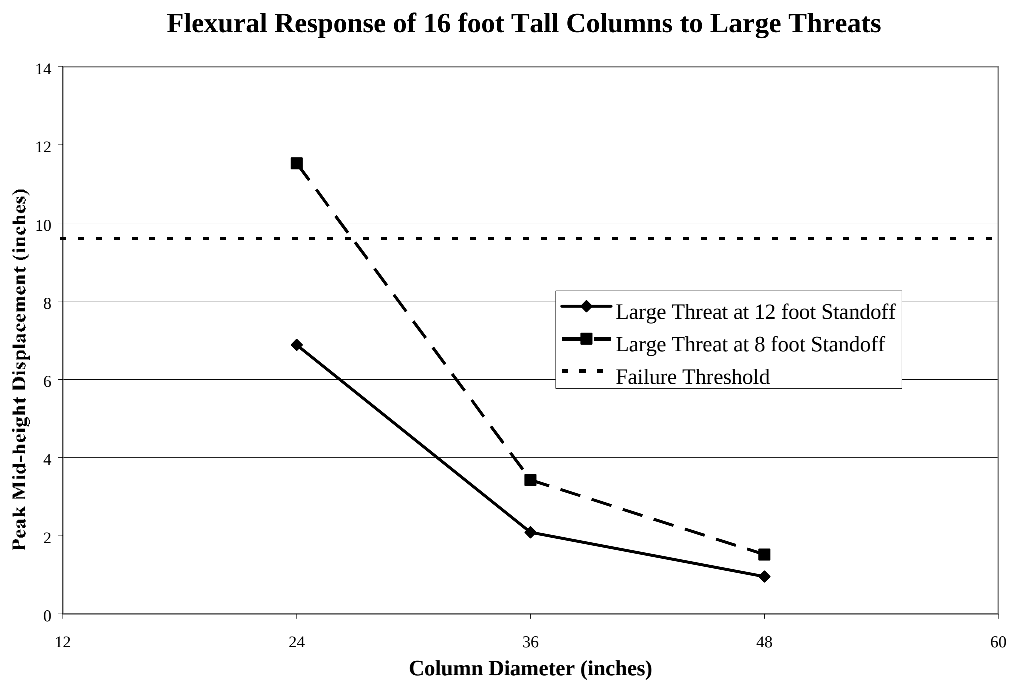


Figure B.1 Pier Performance Variation with Increasing Diameter

From this chart, it is possible to conclude that increasing the diameter of a column improves flexural performance. Increasing section size is a cost-effective measure when considering the low cost of the additional concrete which would be needed under most circumstances. In addition to improving flexural response, increasing column diameter is beneficial for improving resistance to the effects of concrete spall or even complete breach of a section. Other options which are better suited to resist spall or breach such as the use of steel jacketing may exist, but in situations where moderate standoff can be provided which will reduce the severity of expected spall, an increase in column diameter provides a cost-effective means of improving response to blast loads. In addition to this improved performance in flexure and better resistance to spall and breach, the increased diameter will help maintain axial load carrying capacity required to support a bridge superstructure which could be necessary due to a reduced cross-section that could result from localized damage from close-in charges.

As discussed in previous sections, the amount of provided standoff (distance from explosive charge to column face) is a critical parameter in determination of a column's performance under blast loads. An important recommendation for protecting columns against blast is to supply as much standoff as possible, though doing so may be difficult for bridges. As was detailed in Chapter 3 describing the dynamics of blast loads, charge yield and range to target (standoff) are related cubically. This cubic relationship means doubling the standoff will have the effect of reducing the charge yield by a factor of eight. The importance of such a load reduction is clear, and therefore use of the largest practical standoff for bridges with at least moderate criticality is recommended. Standoff can be achieved through creative use of landscaping and other physical barriers. Increased standoff reduces the total impulse acting on a column and therefore decreases the severity of the blast loading. There are limits to the benefits gained by increasing standoff, and further information on this topic can be found in accompanying research performed by Captain David Winget (Winget, As Yet Unpublished). The increase in provided standoff will improve column response in all failure modes; the improvements in column flexural response are demonstrated graphically in Figure B.2.

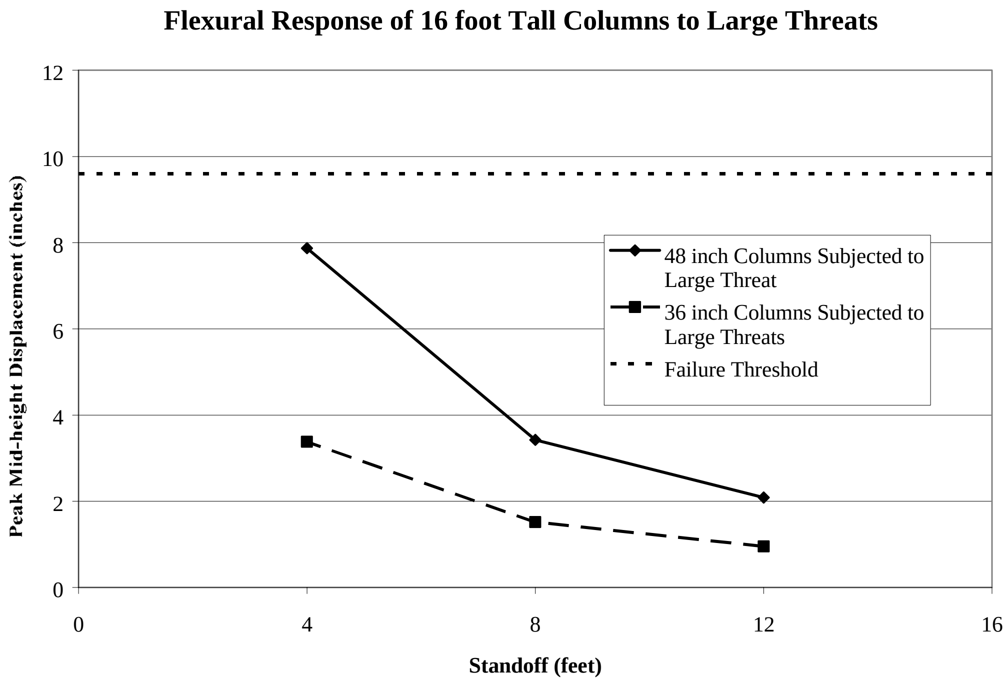


Figure B.2 Effect of Standoff on Improving Column Flexural Response

An investigation of the improvement in flexural performance to blast loads of a column with varying amounts of flexural reinforcement was also performed. A change in longitudinal reinforcement ratio increases flexural stiffness (*EI*) and maximum flexural capacity (M_p). As discussed earlier, these increases in flexural parameters may not be the best method of improving overall column performance because it may decrease effectiveness against alternate failure modes which may control column failure. Viewing the results of dynamic flexural analysis of blast loaded columns, and considering the possible effects on more critical failure modes, it is clear that modest increases in longitudinal reinforcement ratio is not an effective retrofit or design change recommendation. Figure B.3 illustrates the performance of columns with two different longitudinal reinforcement ratios.

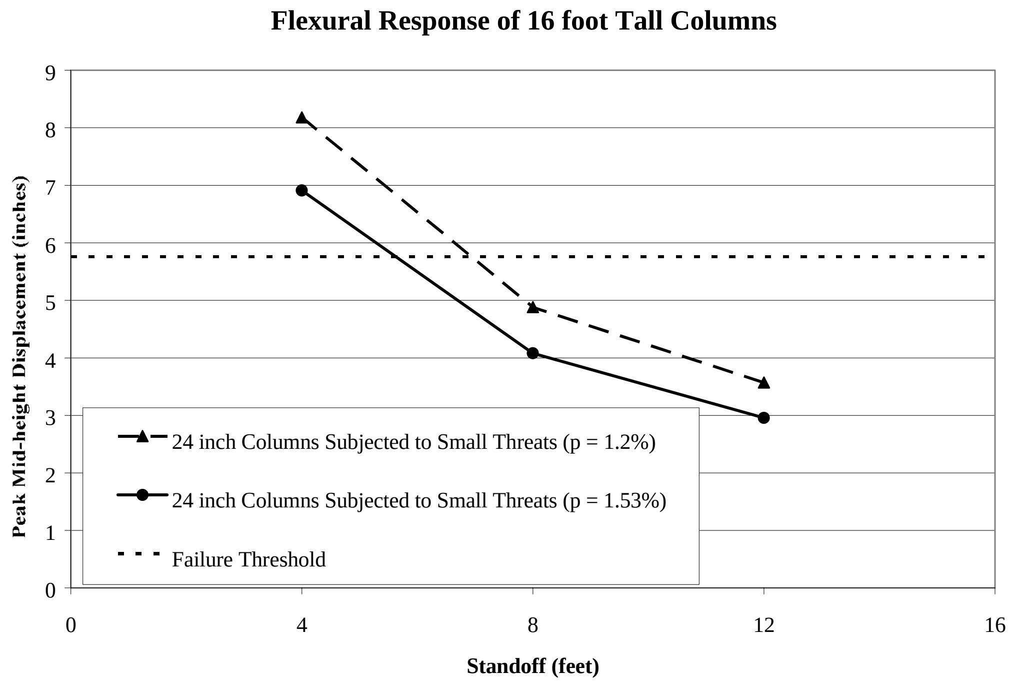


Figure B.3 Effect of Longitudinal Reinforcement Ratio of Flexural Response of Columns

The final parameter investigated for its effect on flexural response is the use of steel jacketing. Steel jacketing of columns is the most expensive retrofit option investigated, however it is also likely the most effective for cases where site restrictions prevent the use of large standoffs. The cost of a steel jacketed column ranges from \$500 to \$600 per foot, very comparable to other jacketing (such as FRP) retrofit costs (Coskun, 2003). This price is put into context when considering a plain concrete column costs approximately \$200 to \$250 per foot. Considering the relative cost of steel jacketed and unjacketed columns it is evident that jacketing is only economically feasible where larger columns, or increased standoff can not be provided, or in critical retrofitting situations. The use of these jackets not only improves the flexural response of a column, but it also dramatically reduces spall or possible column breach, and aids in the prevention of diagonal shear failure.

The results discussed in this section only reflect improvements in flexural response. To understand the benefits gained in prevention of other failure modes, accompanying research should be reviewed (Winget, As Yet Unpublished). For large-diameter columns (48-inch, and some 36-inch diameter) without accounting for concrete spall, there were typically no retrofits necessary to provide an adequate amount of protection against flexural failure. In the case of smaller columns, a retrofit was necessary, and the use of a steel jacket was very effective. The jackets used in this research were one-quarter inch in thickness, have a yield stress of 50 ksi, and considered to be fully bonded to the column to which they were encasing. The effectiveness of steel jacketing for different column diameters and heights is shown in Figure B.4 and Figure B.5.

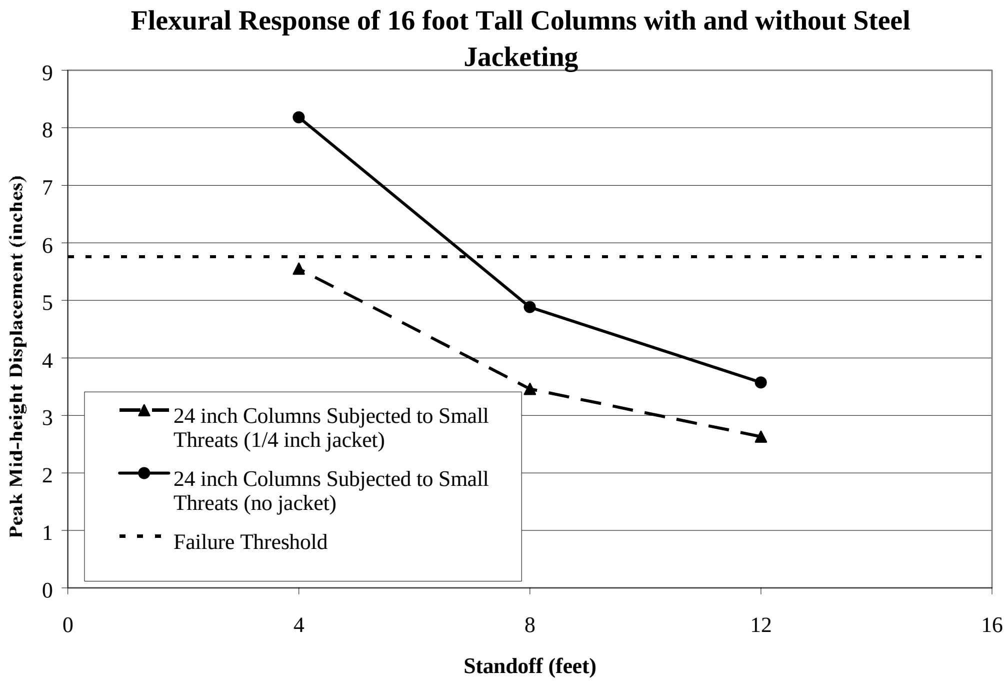


Figure B.4 Flexural Response of 24 inch Diameter Columns with or without Steel Jacketing

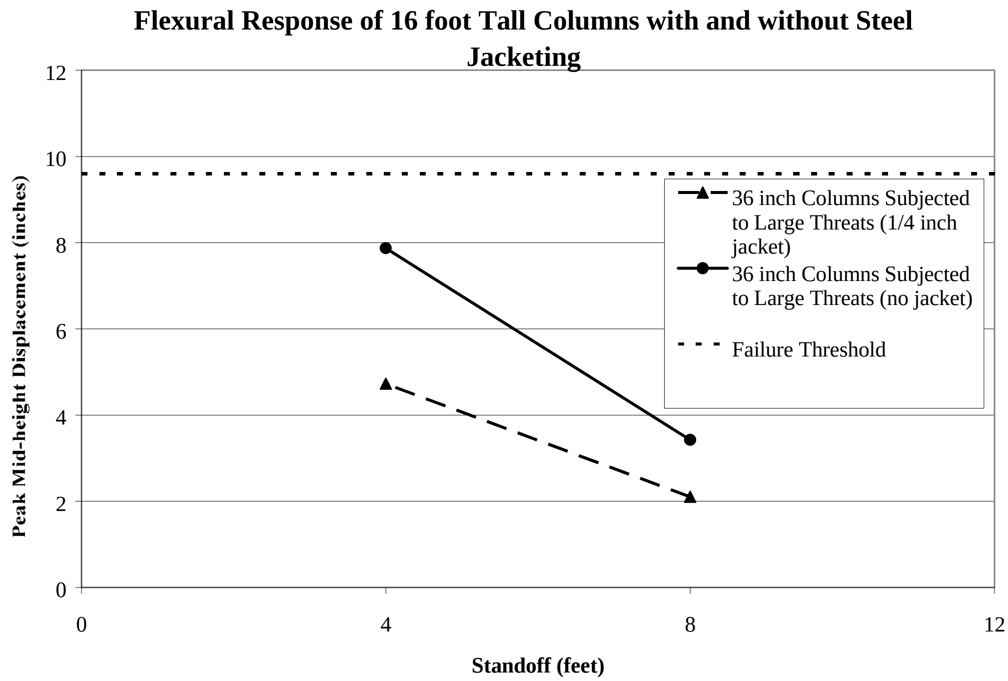


Figure B.5 Flexural Response of 36 inch Diameter Columns with or without Steel Jacketing

B.4 SUMMARY

The importance of standoff and several retrofit options available when adequate standoff cannot be provided are described in the previous sections. Again, it must be made clear that these results reflected only flexural analysis of columns and do not include other potentially very important failure modes. Before retrofitting of columns, it is essential to review the in-progress research of Captain David Winget (Winget, As Yet Unpublished). That research report provides recommendations of specific retrofit options which include improvements in column shear performance, as well as spall and breach prevention. The results and recommendations presented above can, however, be reviewed to provide some guidance as to important parameters in flexural response which can be an important factor in response to blast loads. An estimate of the usefulness of each particular design change or retrofit option with regard to potential benefits in other failure modes has been included with each set of results. Again, the most effective option to mitigate the potential for damage to piers from blasts is the reduction in blast loads that can be achieved through adequate standoff. No specific recommendations as to required standoff for a particular pier parameter configuration are provided in this section because it may conflict with results obtained through investigation of other failure modes. If sufficient standoff cannot be provided, then the most appropriate courses of action are to provide steel jacketing to prevent spall, breach, or diagonal shear failure of a column, or to provide additional shear reinforcement as recommended by Captain David Winget (Winget, Unpublished). Review of the aforementioned research (Winget, Unpublished), including critical failure modes, is essential and will provide recommendations for specific column parameter configurations.

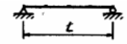
APPENDIX C

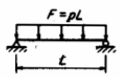
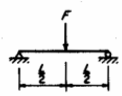
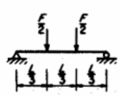
Dynamic System Parameter Calculation

C.1 NECESSARY SYSTEM PARAMETERS

Calculation of the dynamic response of a structural system requires determination of several properties. As explained in Chapter 3, structural stiffness, resistance limits, and conversion factors which account for differences in internal work performed between a real and an idealized system must be defined for analysis. These system properties are based on assumed structural response (e.g. assumption of a beam's static displaced shape as the dynamic mode of vibration), and are sensitive to items such as material properties, boundary conditions, cross-section properties, and loading conditions. Figure C.1 shows properties of simply-supported beams for dynamic analysis (Biggs, 1964), and Figure C.2 shows the same properties for beams with fixed supports.

Table 5.1 ***Transformation Factors for Beams and One-way Slabs***



Loading diagram	*Strain range*	*Load factor* K_L	*Mass factor* K_M: *Concentrated mass**	*Mass factor* K_M: *Uniform mass*	*Load-mass factor* K_{LM}: *Concentrated mass**	*Load-mass factor* K_{LM}: *Uniform mass*	*Maximum resistance* R_m	*Spring constant* k	*Dynamic reaction* V
	Elastic	0.64	....	0.50	....	0.78	$\frac{8\mathfrak{M}_P}{L}$	$\frac{384EI}{5L^3}$	$0.39R + 0.11F$
	Plastic	0.50	....	0.33	....	0.66	$\frac{8\mathfrak{M}_P}{L}$	0	$0.38R_m + 0.12F$
	Elastic	1.0	1.0	0.49	1.0	0.49	$\frac{4\mathfrak{M}_P}{L}$	$\frac{48EI}{L^3}$	$0.78R - 0.28F$
	Plastic	1.0	1.0	0.33	1.0	0.33	$\frac{4\mathfrak{M}_P}{L}$	0	$0.75R_m - 0.25F$
	Elastic	0.87	0.76	0.52	0.87	0.60	$\frac{6\mathfrak{M}_P}{L}$	$\frac{56.4EI}{L^3}$	$0.525R - 0.025F$
	Plastic	1.0	1.0	0.56	1.0	0.56	$\frac{6\mathfrak{M}_P}{L}$	0	$0.52R_m - 0.02F$

* Equal parts of the concentrated mass are lumped at each concentrated load.

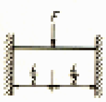
Source: "Design of Structures to Resist the Effects of Atomic Weapons," U.S. Army Corps of Engineers Manual EM 1110-345-415, 1957.

Source: "Introduction to Structural Dynamics" Biggs 1964

Figure C.1 Important Dynamic Analysis Properties of Simply-Supported Beams (Biggs, 1964)

$\mathfrak{M}_{Ps}$ = ultimate moment capacity at support
$\mathfrak{M}_{Pm}$ = ultimate moment capacity at midspan

Fixed ends

Loading diagram	Strain range	Load factor K_L	Mass factor K_M		Load-mass factor K_{LM}		Maximum resistance R_m	Spring constant k	Effective spring constant k_E†	Dynamic reaction V
			Concentrated mass*	Uniform mass	Concentrated mass*	Uniform mass				
	Elastic	0.53	...	0.41	...	0.77	$\frac{12\mathfrak{M}_{Ps}}{L}$	$\frac{384EI}{L^3}$	......	$0.36R + 0.14F$
	Elastic-plastic	0.64	...	0.50	...	0.78	$\frac{8}{L}(\mathfrak{M}_{Ps} + \mathfrak{M}_{Pm})$	$\frac{384EI}{5L^3}$	$\frac{307EI}{L^3}$	$0.39R + 0.11F$
	Plastic	0.50	...	0.33	...	0.66	$\frac{8}{L}(\mathfrak{M}_{Ps} + \mathfrak{M}_{Pm})$	0	......	$0.38R_m + 0.12F$
	Elastic	1.0	1.0	0.37	1.0	0.37	$\frac{4}{L}(\mathfrak{M}_{Ps} + \mathfrak{M}_{Pm})$	$\frac{192EI}{L^3}$	......	$0.71R - 0.21F$
	Plastic	1.0	1.0	0.33	1.0	0.33	$\frac{4}{L}(\mathfrak{M}_{Ps} + \mathfrak{M}_{Pm})$	0	......	$0.75R_m - 0.25F$

* Concentrated mass is lumped at the concentrated load.
† See Fig. 5.4.
Source: "Design of Structures to Resist the Effects of Atomic Weapons." U.S. Army Corps of Engineers Manual EM 1110-345-415, 1957.

Figure C.2 Important Dynamic Analysis Properties of Fixed-Supported Beams (Biggs, 1964)

C.2 Example Calculations

This section demonstrates the calculation of important dynamic analysis properties for a simply-supported beam subjected to a point load at midspan. Figure C.3 shows a diagram of the system for which the following calculations apply.

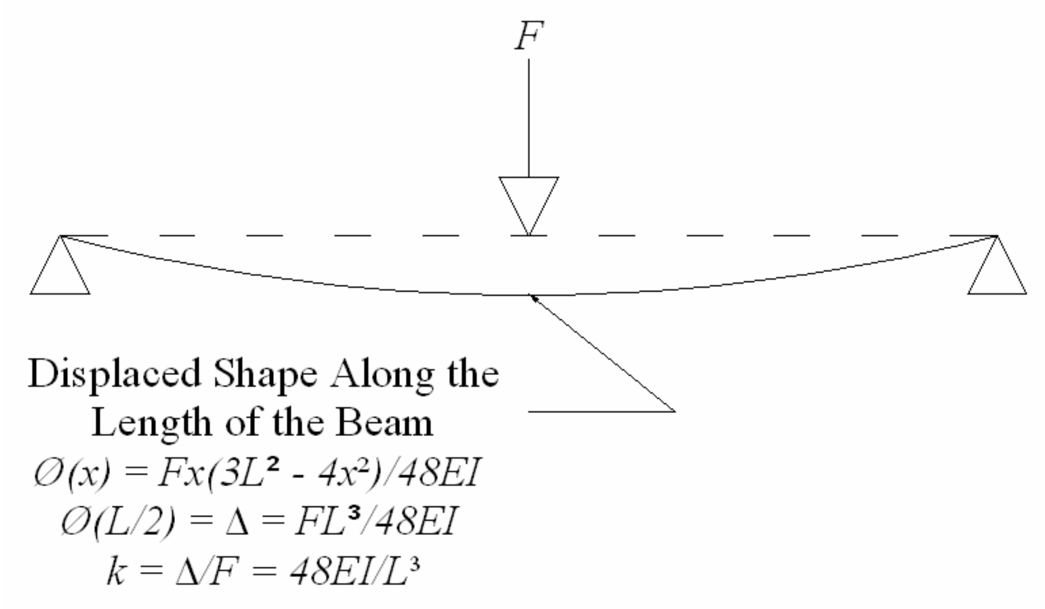


Figure C.3 Force-Displacement Relationship for a Simply-Supported Beam

C.2.1 Stiffness

Beam stiffness is determined by calculation of the displacement at the location of maximum deflection along the assumed displaced shape for a unit amount of force. For the system shown in Figure C.2, the critical displacement occurs at midspan, and the assumed displaced shape corresponds to the static displaced shape of a beam subjected to a point load. This assumption is a valid and commonly made choice, it would also be reasonable to chose a dynamic mode shape, however the more easily obtained static shape provides sufficient accuracy. Calculations involved in the determination of system stiffness are shown in Equations C.1 through C.4. In these equations $\nu(x)$ is the displaced shape of the beam, F is an applied point load at midspan, x is a distance along the length of the beam, L is the length of the beam, E is Young's Modulus, I is the beam's moment of inertia, Δ is the midspan deflection, and k is the beam's flexural stiffness.

$$\nu(x) = F \cdot x \cdot (3L^2 - 4x^2)/(48 \cdot E \cdot I) \tag{C.1}$$

$$\nu(L/2) = \Delta = F \cdot L^3/(48 \cdot E \cdot I) \tag{C.2}$$

$$k = F/\Delta \tag{C.3}$$

$$k = 48 \cdot E \cdot I / L^3 \tag{C.4}$$

C.2.2 Maximum Resistance

The internal resistance of a beam is required to determine dynamic response. Changes in internal resistance, and hence stiffness, occur due to the formation of plastic hinges. Figure C.4 shows the shape of a simply-supported beam after the formation of a plastic hinge at midspan. The internal resistance limit to the previously calculated stiffness (Equation C.4) is determined by equating internal and external work for the beam as described in Equations C.5 through C.7. Variables shown in these equations are consistent with those shown previously. In addition, W is the work performed on or by the system, θ is an assumed rotation, R_{max} is the maximum internal resistance of the system, and M_p is the plastic moment capacity of the beam.

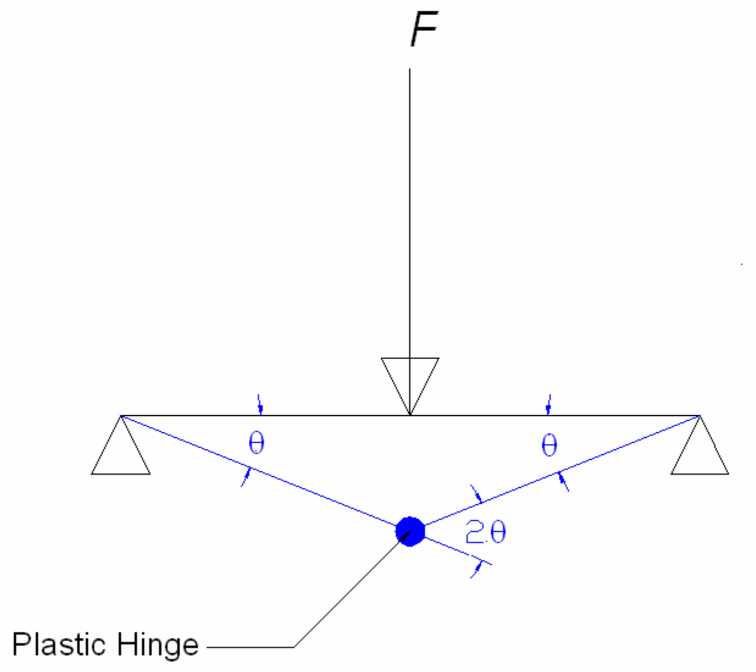


Figure C.4 Diagram for Resistance Calculation

$$W_{\text{internal}} = W_{external} \tag{C.5}$$

$$2 \cdot \theta \cdot M_p = F \cdot \theta \cdot L / 2 \tag{C.6}$$

$$F = R_{\max} = 4 \cdot M_p / L \tag{C.7}$$

C.2.3 Transformation Factors

As described in Chapter 3, transformation factors must be calculated to complete the conversion from a real system to an idealized dynamic system. These transformation factors are based on equating the work between the critical point on the idealized structure and the work done by the entire actual structure. These calculations require use of the beam's displaced shape normalized such that the critical deflection is a unit amount, the distribution of mass along the structure, and the applied loading conditions. The calculation of important transformation factors is shown in Equations C.8 through C.13 for the example considered previously. In these equations, M_e is the mass transformation factor, m is the mass per unit length of the beam, $\phi(x)$ is the beam's normalized displaced shape, L_f is the load transformation factor, L is the beam length, and L_{mf} is the load-mass transformation factor. All other variables are as previously defined.

$$\phi(x) = \nu(x)/\nu(L/2) \tag{C.8}$$

$$\phi(x) = [F \cdot x \cdot (3L^2 - 4x^2)/(48 \cdot E \cdot I)]/[F \cdot L^3/(48 \cdot E \cdot I)] \tag{C.9}$$

$$M_e = \int m \cdot \phi(x)^2 \tag{C.10}$$

$$M_e = 2 \cdot \int_0^{L/2} m \cdot (3/L - 4 \cdot X^2/L^3)^2 = .49 \tag{C.11}$$

$$L_f = \frac{\sum_r F_r \cdot \phi_r}{\sum_r F_r} = 1 \tag{C.12}$$

$$L_{mf} = M_e / L_f = .49 \tag{C.13}$$

C.3 COMMENTS

This appendix demonstrates the calculation of important system properties required for dynamic analysis. Tables similar to those provided in Figures C.1 and C.2 are readily available in dynamics textbooks, but may vary depending upon selection of a displaced shape. As previously discussed, other choices of displaced shape are available, however the simply obtained static displaced shape is acceptable for sufficient accuracy. These properties have been used in this research for calculation of dynamic response for systems in which the loading can be reasonable approximated as uniformly distributed. The derivation of system properties in this chapter was provided for a point load system to provide mathematical simplicity, however the same procedure is applicable to the loading conditions used in this research. Specific reference to their use can be found in Chapter 4 and Appendix A when describing assumptions made for deck sections and columns.

APPENDIX D

Dynamic System Parameter Calculation for a Beam with Varying Length Distributed Loads

D.1 USE OF SYSTEM PROPERTIES

As explained in Chapter 3, dynamic response requires information about structural stiffness, resistance limits, and conversion factors which account for differences in internal work performed between a real and an idealized system. These system properties are based on assumed structural response, and are sensitive to items such as material properties, boundary conditions, cross-section properties, and loading conditions. Appendix C shows how these system properties have been calculated and organized into charts such as those found in structural dynamics textbooks (e.g., Biggs, 1964). The current research studies blast loads on beams, and as described in Chapter 3, this loading can be approximated using a series of varying length distributed loads. Because the dynamic system properties and transformation factors have not been derived for beams under these loading conditions, their development was required for this research.

D.1.1 Development Procedure

The first step in determination of required dynamic system properties is the assumption of a displaced shape of the loaded beam. Figure D.1 shows the loaded beam for which the displaced shape is to be calculated.

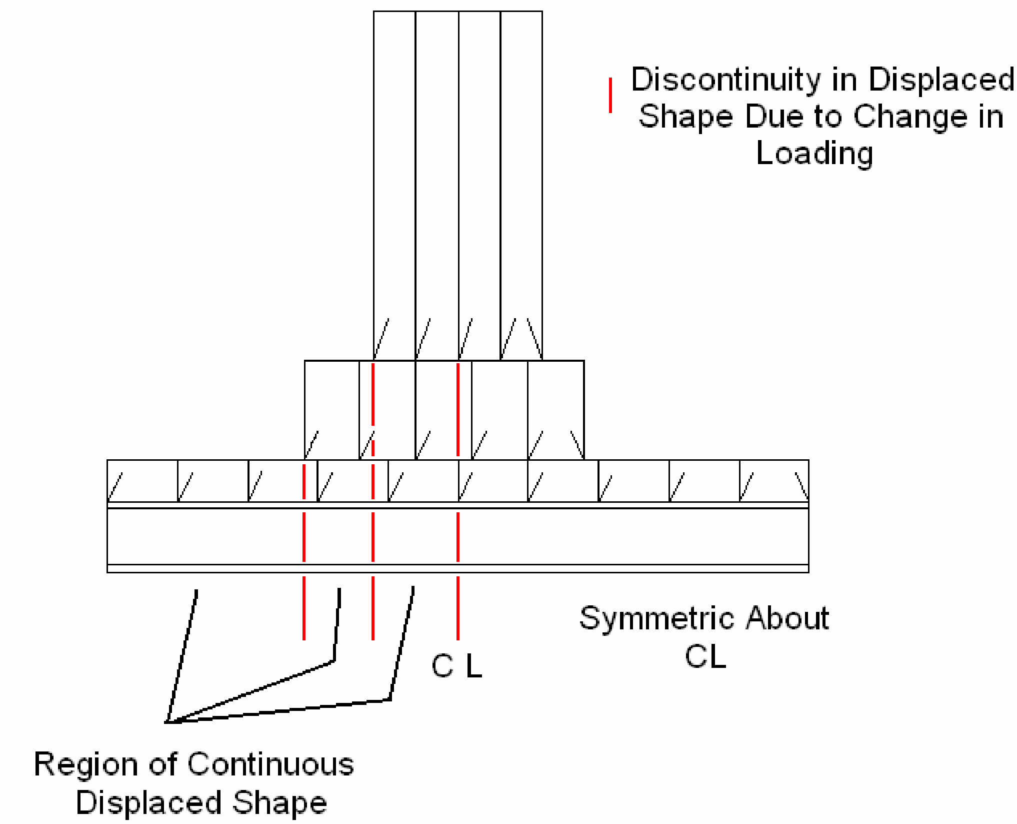


Figure D.1 Diagram for Displaced Shape Calculation

The displaced shape required for analysis is the deflection that the beam will undergo in dynamic response. The use of the static displaced shape under similar loading is an acceptable approximation of the actual response. It is necessary to derive a general expression for the deflected shape corresponding to the assumed load profile. This research uses a beam subjected to a series of distributed loads of varying length. A beam is modeled as a sequence of events corresponding to the formation of one or more zero length plastic hinges. The model is modified to account for changing releases at each event. Although elasticity will occur leading up to the plastic hinging, it is sufficient to assume that the beam is elastic until hinge formation. Because no inelasticity occurs during each stage of deflection, the displaced shape of a beam under individual portions of the applied load can be combined using the principle of superposition. With this concept, displaced shapes for a beam under a full-length distributed load, and two separate partial-length distributed loads can be added together. The beam's deflected position under a full-length uniform load can be determined at any location using the formula provided in the AISC LRFD Manual (AISC, 1998). Because the loading is not continuous along the length, the displaced shape under the remaining two variable-length distributed loads must be defined piecewise. Figure D.1 illustrates the concept of defining the shape in several regions.

The displaced shape of the beam can be determined from compatibility after assuming that the shape in each region is of the form shown in Equation D.1. In Equation D.1, v_1 is the displaced shape as a function of the length along the beam, a_i is a constant to be determined through compatibility, and v_p is the particular solution to the differential equation which is dependant on the loading of the region in question.

$$v_1(x) = a_0 + a_1 x + a_2 x^2 + a_3 x^2 + v_p(x) \qquad \text{(B.1)}$$

After calculating the particular solution through the use of a differential equation defining the beam's response and the constants a_i through solution of the system of compatibility equations, the dynamic system properties can be determined. These system properties are calculated using an overall displaced shape formed by the summation of the three different shapes (one from each of the loaded regions) normalized such that the beam's peak midspan displacement is one.

The load and mass factors are calculated using the same equations as presented previously in Appendix C and shown again here for convenience in Equations D.2, D.3, and D.4. In these equations, M_e is the mass transformation factor, m is the mass per unit length of the beam, $\phi(x)$ is the beams normalized displaced shape, L_f is the load transformation factor, *w, m, and p* are the applied distributed loadings in each region, L is the beam length, and L_{mf} is the load-mass transformation factor.

$$M_e = \int m \cdot \phi(x)^2 \tag{D.2}$$

$$L_f = \frac{\int_0^{nL} w \cdot \phi(x) \cdot dx + \int_0^{(j-n)L} m \cdot \phi(x) \cdot dxx + \int_0^{(1-j-h)L/2} p \cdot \phi(x) \cdot dxx}{(w + m + p) \cdot L} \tag{D.3}$$

$$L_{mf} = M_e / L_f \tag{D.4}$$

Each integration involving the displaced shape $\phi(x)$ must be performed piecewise and correlated with the appropriate length over which that displaced shape is valid. The appropriate load acting over that displaced shape must also be used, and the resultant of the entire load is required in the denominator of Equation D.3.

An event to event analysis of a beam is performed in which the formation of plastic hinges causes stiffness changes. The stiffnesses and resistance limits of each stiffness are determined in a manner consistent with those calculated in Appendix A. Some differences occur between calculations of these properties under different loading conditions. The total load resultant of the load on a beam with only a single uniform load, and the load resultant for each region of the load on a beam under variable length distributed loads are used in each case for stiffness determination, and the calculation of work performed by each region of load in determination of the external work used for resistance limit calculation.

D.2 COMMENTS

The exact derivation of the displaced shape and system properties for a beam under three regions of applied loads is not shown because of the resulting lengthy mathematical expressions. The approach described previously, however, allows for the determination of system properties for each stage of deformation of a beam. For example, if a beam is assumed to be loaded as previously described and has fixed supports, the system properties must be calculated under these boundary conditions. Once plastic hinges form at the supports, a new stiffness for the system is needed, and the beam can be modeled as being simply supported. Finally, formation of a plastic hinge at midspan will cause the deflected shape to change again, and the new system parameters needed for analysis during this stage of response can be calculated as described above.

APPENDIX E

Selected Parameters and Coupling for Analysis

E.1 PURPOSE

As described in earlier chapters, the purpose of this research is to generate recommendations for retrofits, design changes and best practices for improving bridge performance under terrorist attack. The method used in this research for determining the most effective blast mitigation techniques is based on an examination of computed responses of a wide variety of different systems with different design parameters. The range of parameters chosen for this research is presented in this appendix.

E.1.1 Parameter Selection and Coupling

A description of the parameters studied and the benefits gained in blast mitigation through the use of each configuration are discussed previous chapters. This appendix provides a chart diagramming each selected system property and the other properties for which an investigation into possible coupling effects was carried out. Figure E.1 lists each bridge component, the studied variations, and illustrates in yellow the coupling within that bridge component investigation. A retrofit or design change option shown in yellow is coupled with all other parameters for the applicable component. Figure E.1 is a modified version of a figure created by Captain Dave Winget (Winget, 2003).

Steel Girders Bridges & Piers	Most-Likely COAs	
Design & Retrofit / COA	Above Deck	Below Deck
Girders		
Design Configurations	S	
x3 girder depth (60", 66", 72")	provides load comparison for retrofits and initial design parameters	
x2 girder spacing (8', 12')		
x 3 clearance (16', 20', 24')		
x2 material strengths (50, 75ksi)		
x3 deck thickness (8", 10", 14")		
x 3 spans (80', 120', 160')		
Piers		
Design Configurations		PSBC
x3 diameters (24", 36", 48")	provides comparison for above three retrofits and initial design parameters	
x 3 clearance (16', 20', 24')		
x2 reinforcement ratios (1.2%, 1.5%)		
x1 jacket thickness (0.25")		
P = prestressed concrete girder	Coupled with all others	
S = steel plate girder	Coupled only with yellow boxes	
B = concrete segmental box girder		
T = steel truss		
C = cable-stayed		

Notes:
specifically for selected 5 categories
under assumptions made in COA analysis
neglects foundations and abutments

Figure E.1 Selected Bridge Configurations (Winget, 2003)

E.1.2 Parameters Not Specifically Explored

Several design and retrofit options are were not analyzed. Examples of items not considered are lateral bracing for piers or girders, use of cable restrainers to prevent unseating of girders, and the use of Styrofoam panels between girders to reduce loads under a deck. Recommendations were provided in Chapter 5 and Appendix B to ensure that failure through modes related to those parameters would be prevented. These options do however warrant consideration for future research through the use of more detailed models accounting for more complex behavior and localized affects. For example, the recommendation of provision of ductile connections for steel girder or truss bridges which can develop 125% of member capacity and sustain large rotations will prevent connection behavior from controlling the capacity, however more information would be useful to assess the role of connection behavior on blast response.

E.2 COMMENTS

This appendix provides a graphical illustration of parameters studied for bridge component types included in this report. Figure E.1 provides the framework for which a body of data is developed to determine the system configurations which provide the greatest measure of blast mitigation. Figure E.1 represents the first step in determination of system properties which can be recommended for improvement in existing or newly designed bridges.

APPENDIX F

Comparison of Single Degree-of-Freedom and ABAQUS Models of Piers

F.1 CONCEPT OF COMPARISON

This research focuses on computing structural response to blast loads considering a large number of system configurations and parameter combinations. It is for this reason that an analysis method that provides suitably accurate results, yet is computationally efficient, be used so that a large number of cases can be considered. The scope of this work is to provide a basis for relative comparisons of retrofit and design change techniques, and to formulate appropriate recommendations as to system configurations to enhance blast mitigation. This scope cannot be achieved effectively through the use of a small number of complex and computationally intensive analyses. It is important however, that the less complex analyses provide a reasonable level of accuracy. To ensure that a sufficient level of accuracy is obtained, a comparison of the results of a single degree-of-freedom (SDOF) analysis of a steel wide flange column was made with the results of a more complex multiple-degree-of-freedom (MDOF) finite element model using the ABAQUS software.

F.2 SETUP

The comparison between the SDOF and ABAQUS models was made for a W18x76 wide flange column. The column was taken as fixed-supported at both ends with a length of 172 inches. A blast of charge weight and standoff on the order of the blasts studied for columns and superstructures in this research was used. Blast pressure loads for the SDOF column model were determined using the uniform equivalent load provided by the CONWEP software. The loading function for the ABAQUS model was determined from the Blast-X software at various locations along the height of the column, and the pressure at that location was used to generate a uniform load over the target's tributary area. An example of load determination through the use of tributary areas is shown in Figure F.1.

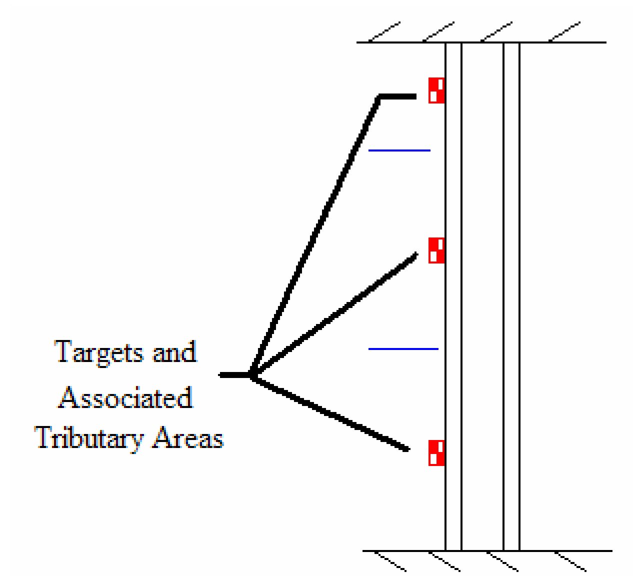


Figure F.1 Tributary Breakdown for Column Load Determinations

Because CONWEP does not account for wave reflections which increase the magnitude of the impulse acting on the column studied, the pressure applied to the ABAQUS model were scaled down by a factor of 25%. This reduction on load is consistent with the concept of increasing the impulse provided by the CONWEP software which was utilized and described in Chapter 4 and Appendix A of this research.

F.3 RESULTS AND COMMENTS

Investigation of a column using analysis methods of varying degrees of complexity allows for the verification that the selected single degree-of-freedom model provides sufficient accuracy and is properly modeling the desired mode of response. Figure F.2 shows the displacement-time histories obtained from each model. The accuracy of the single degree-of-freedom model is shown to be sufficient by comparison of the peak displacements, as well as the amplitude and the natural period of vibration of the component. Although scaling of the load was required to achieve agreement of the analysis approaches, this scaling is appropriate because it accounts for the difference in impulse of loading provided by the two different computer programs. Because the results generated by the different analysis approaches is in agreement, it is reasonable to use single degree-of-freedom models to perform a large number of parameter studies. One case does not by itself suggest that all cases will agree this well with the MDOF case, but the results are reasonable and can be used to asses relative improvements in performance.

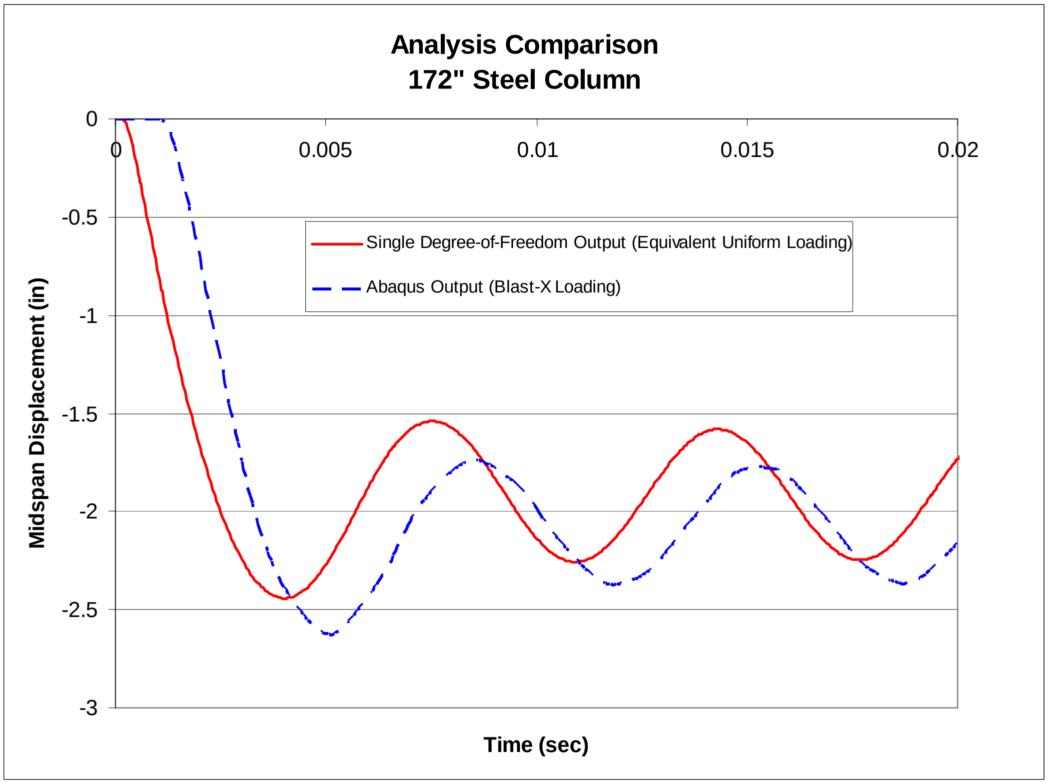


Figure F.2 Comparison of Displacement Histories of a Column from a Single Degree- of-Freedom Model and an ABAQUS Model

APPENDIX G

Determination of Steel Jacketing Benefits

G.1 CONCEPT

A portion of this research focuses on determining retrofit options appropriate for improving column performance under blast loads. One retrofit option investigated is the use of a steel jacket encircling a column. Expected benefits from the use of a steel jacket are improvement in column confinement, increased moment of inertia, increased flexural strength, resistance to concrete spall, and improved shear resistance. Because models for jacketed column behavior were not readily available, a procedure for determination of column cross-section properties was investigated.

G.2 INVESTIGATION

As previously discussed, the RCCOLA software was used to generate a moment-curvature relationship for each of the column sections studied in this research. RCCOLA does not allow for direct input of a steel jacket; therefore secondary longitudinal reinforcement was added to account for the important benefits gained in strength and flexural stiffness. Because all of the jacketing steel is not located in an effective area within the cross-section, only a portion of the jacketing steel area was assumed to contribute to improved section performance. The investigation considered variations in the percentage of contributing steel jacket area from 30 to 70% in ten percent increments. For each of the studied amounts, a moment-curvature relationship was developed to determine the flexural stiffness parameter *EI* and the section's ultimate moment capacity.

Figure G.1 shows the results of the moment-curvature analyses along with linear regression curves used to determine flexural stiffness.

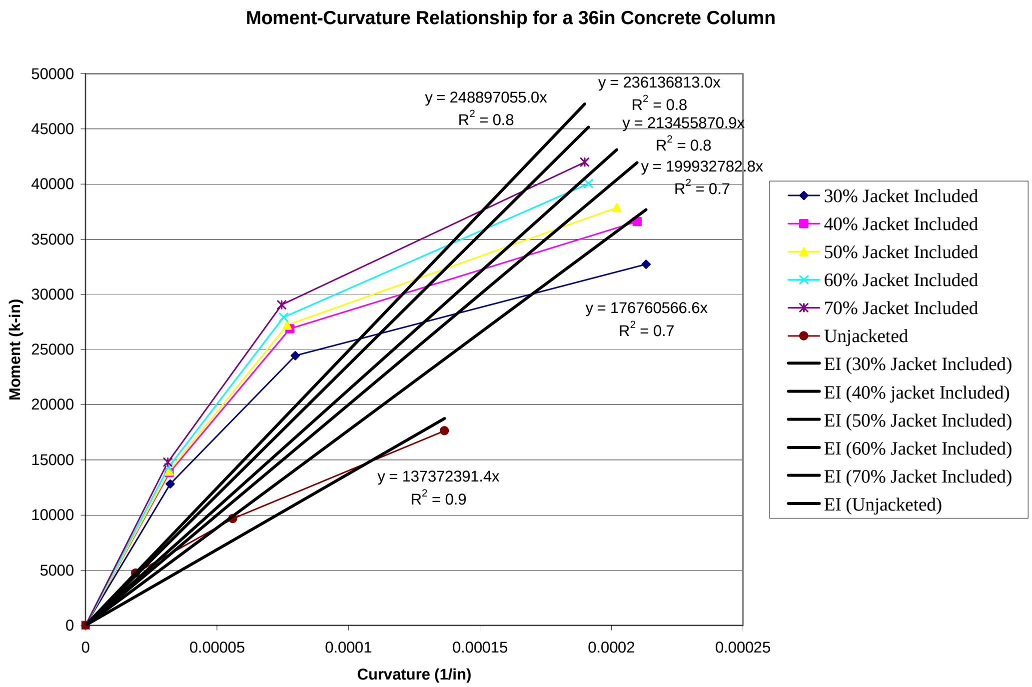


Figure G.1 Moment-Curvature Relationships for Columns Containing a Varying Percentage of Jacketing Steel

Figure G.1 demonstrates the change in both stiffness and strength of a column cross-section when additional reinforcing steel is applied within RCCOLA. Because a variation occurs in these important dynamic analysis parameters, an investigation into the effect of these changes on flexural response is necessary. The investigation for this research considered the behavior of a column subjected to a blast load consistent with those used for the substructure parameter studies. Figure G.2 shows the variation in displacement response histories for columns with varying amounts of jacketing steel included.

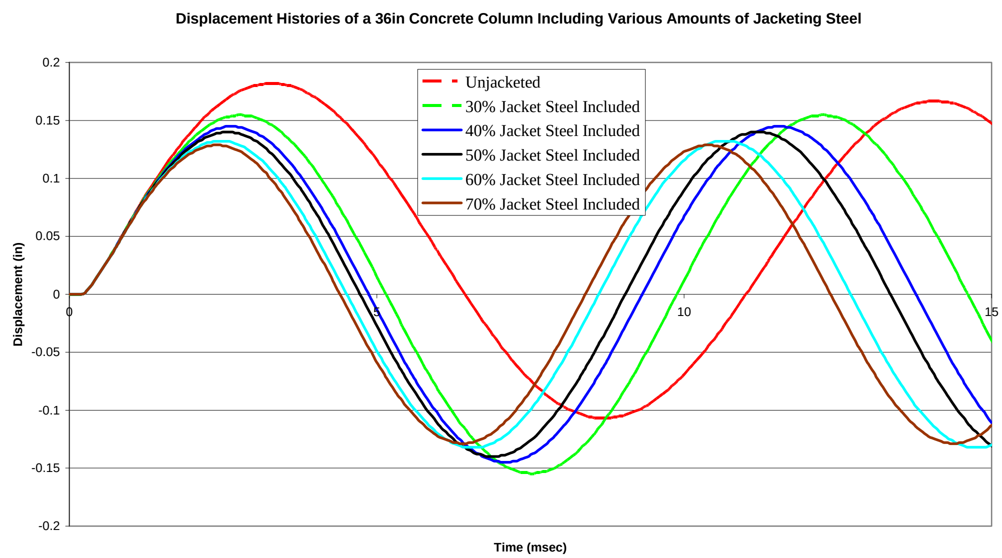


Figure G.2 Displacement Histories for Columns Containing a Varying Percentage of Jacketing Steel

Figure G.2 illustrates that a variation in maximum response is achieved when the extreme values of jacketing steel are considered. However, focusing on the middle range of jacketing percentage considered, the use of 40-60% of the jacketing steel leads to similar analysis results. In addition to the maximum response achieved, it is important to consider the actual amount of jacketing steel which will likely be contributing to column response. Although all of the jacket steel will be located at some distance from the neutral axis of a column, only a fraction of this steel will be located in the most effective regions near the extreme fibers of the cross-section. This research assumes that because the variation in the computed response of columns containing 40-60% of the jacketing steel area is not large, and that approximately this percentage of steel will be located in the most effective regions near the extreme fibers of a column, 50% of the jacketing steel is appropriate for modeling column performance in the retrofitted configuration. This assumption is subjective; however, since only relative comparisons of column performance are important, the assumption employed for the current study will provide an adequate basis for response comparisons to unjacketed columns.

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