BLAST PERFORMANCE OF BRMS STRUCTURES WITH SLIDING FOUNDATION & EFFECTS OF DYNAMIC INTERACTION OF STRUCTURAL MEMBERS ON ESDOF BLAST ANALYSIS

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BLAST PERFORMANCE OF BRMS STRUCTURES WITH SLIDING FOUNDATION & EFFECTS OF DYNAMIC INTERACTION OF STRUCTURAL MEMBERS ON ESDOF BLAST ANALYSIS

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To My Lovely Wife & My Parents

ABSTRACT

Growing terror threat and accidental blast incidents can have significant consequences including structural collapse, personal casualties, and financial losses. Therefore, blast performance of structures is becoming an important design consideration for not only military structures but also for structures located at or near potential explosion sites such as petrochemical industry. However, blast resistant structures can be extremely costly due to high magnitude of blast loads. To minimize cost, prefabricated blast resistant modular steel (BRMS) structures are becoming more common as economic solutions especially in petrochemical industry. Typically, these structures are anchored to their foundation, however this type connection results in large foundation reactions and as a consequence very large and costly foundations. Therefore, occasionally these structures are just left on top of their foundation without any connection to their foundation, which is known as "free-to-slide" foundation to minimize foundation reactions and cost.

In the first part of this study, blast performance of a two-module BRMS building was determined by performing 3D nonlinear dynamic blast analysis for different foundation cases including anchored and free-to-slide foundations with different sliding properties. Blast performance of structural members was compared for each foundation case to determine effects of foundation type on blast performance of structure and its members. Performing dynamic blast analysis with large 3D finite element models, where contact and separation is modelled requires significant computational resources. Therefore, this study investigated viability of using anchored foundation and neglecting sliding for conservatively evaluating blast performance of free-to-slide structures.

In the second part of this study, feasibility of using equations of rigid body motion for structures with free-to-slide foundation to determine their horizontal acceleration, velocity, and sliding histories under blast loads was investigated. For this purpose, a simple and computationally cheap nonlinear numerical integration method was developed and verified with FE models. Accuracy of using this rigid body approach to predict building sliding distance, velocity, and acceleration were determined as well as its foundation horizontal and vertical reactions.

In the last part of this thesis, effects of dynamic coupling of inter-connected structural elements on their blast performance and viability of using uncoupled single degree of freedom (USDOF) blast analysis method were evaluated by performing blast analysis of a girder-beam system. Effects of structural interaction (coupling) on blast performance of girder-supported beam was evaluated by performing blast analysis for both elastic and elastic-perfectly plastic girder-beam systems. The ratio of vibration periods of the girder and supported beam were changes from 0.125 to 4.0 to determine effects of structural coupling between the members for different relative stiffness of structural members. The results and findings were used to provide guidance on limitations for use of USDOF blast analysis method.

ÖZETÇE

Artan terör tehdidi ve patlama kazaları; bina hasarları, can kayıpları ve ekonomik kayıplar gibi kötü sonuçlar doğurabilmektedir. Bu sebeple, binaların patlama yükleri altındaki dayanım performansları günümüzde daha önemli bir dizayn unsuru halinde gelmektedir. Patlama dizaynı sadece askeri binaların dizaynında değil, aynı zamanda patlama kazası riski büyük olan petrokimya endüstrisinde kullanılmaktadır. Bununla birlikte; patlamaya dayanıklı binalar, patlama yüklerinin büyük olması sebebiyle çok maliyetli olabilmektedir. Bu maliyetleri azalttığı için, özellikle petrokimya endüstrisinde, prefabrik patlamaya dayanıklı modüllü çelik binalar, ekonomik çözümler olarak her geçen gün yaygınlaşmaktadır. Bu modüler binalar genellikle temellerine kalıcı olarak sabitlenmektedir ve bu sabitlenen temel bağlantıları çok büyük temel yüklerine, dolaylı olarak ta çok büyük temellere ve maliyetlere sebep olmaktadır. Bu sebepten, bu binaların temlerine bağlanmadığı veya patlama yüklerinde binanın yanal yer değiştirmelerini engellemeyecek tasarımlar yaygınlık kazanmakta. Bu şekilde modüler binaların, temelinin üstünde hareketi kısıtlanmamış olmakta, temel yükleri ve dolaylı olarak ta temel maliyetleri en aza indirilmiş olmaktadır.

Bu tezin ilk bölümünde, iki modüllü bir "patlamaya dayanıklı modüler çelik" binanın 3 boyutlu doğrusal olmayan dinamik patlama analizi ile patlama performansı incelenmiştir. Bu çalışmada sabit ve sabit olmayan (kayan) temel tipleri gibi farklı kayma özelliği olan temel tiplerinin yapının patlama yükleri altındaki performansına etkileri incelenmiştir. Temel tiplerinin, binaya ve yapı elemanlarına etkisini anlamak için; farklı temel bağlantı durumlarının yapının patlama performansına etkileri karşılaştırılmıştır. Yapı temel etkileşiminin modellendiği 3 boyutlu sonlu elemanlar

modelleri ile dinamik patlama analizleri yapmak, yüksek performanslı bilgisayarlar gerektirmektedir. Bundan dolayı bu çalışmada, kayan temel tipli yapıların sabit olarak incelenmesinin yapının patlama performansına etkileri ayrıca incelenmiştir.

Bu tezin ikinci bölümünde; yatay ivme, hız ve kayma değerlerinin, patlama yükleri altındaki sabit olmayan temelli binalarda, rijit cisim hareket formülleri kullanılarak hesaplanabilirliği incelenmiştir. Bu inceleme için, basit ve kolay hesaplanabilir bir doğrusal olmayan sayısal integral yöntemi geliştirilmiş ve bu yöntemin sonlu elemanlar yöntemi kullanılarak doğrulanması yapılmıştır. Rijit cisim yönteminin yeterliliği; kayma, hız ve ivme değerlerinin yanında yatay ve dikey temel tepki değerleri için de saptanmıştır.

Bu tezin üçüncü ve son bölümünde; patlama yükleri altındaki yapı elemanlarının, düğüm noktalarındaki dinamik etkileşimi incelenmiş ve tek serbestlik dereceli patlama analizi yönteminin uygulanabilirliği, birbirine bağlı bir üçlü kiriş sistemi üzerinde patlama yükleri uygulanarak incelenmiştir. Bu üçlü kiriş sisteminin patlama yükleri altındaki etkileşimi hem elastik hem de elastik-tam plastik malzeme özellikleri kullanılarak incelenmiştir. Ana kiriş ve destek kirişlerinin doğal titreşim periyotları oranı 0.125 ve 4.0 arasında değiştirilerek, farklı rijitlik derecelerine sahip yapı elemanları arasındaki yapısal etkileşim saptanmıştır. Sonuçlar ve bulgular, bağlı olmayan tek serbestlik dereceli patlama analizi yönteminin kısıtlamaları için yol gösterici olarak kullanılmıştır.

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ABBREVIATIONS

$\ddot{\theta}$	Rotational acceleration
$\dot{\theta}$	Rotational velocity
$\ddot{x}$	Acceleration
$\dot{x}$	Velocity
θ	Rotation
ϕ	Deflected shape function
Δt	Time step size
A_{front}	Building front wall area
A_{roof}	Building roof area
B	Building width
DIF	Dynamic increase factor
$F(t)$	Total load
FE	Finite element
F_H	Net horizontal force
Fv	Net vertical force
g	Vertical acceleration constant
H	Building height
I	Building inertia
K	Spring or member stiffness
K_L	Biggs' stiffness transformation factor
K_M	Biggs' mass transformation factor
K_{ML}	K_M/K_L
L	Width of building or member span length
M	Total mass
m	Mass density per length
M_O	Overturning moment
M_R	Restoring moment
P_{Front}	Front face blast pressure
P_{rear}	Rear wall blast pressure
P_{roof}	Roof blast pressure

R_u	Ultimate resistance
SIF	Strength increase factor
T	Vibration period, seconds
T_1	Vibration period of girder, seconds
T_2	Vibration period beam, seconds
USDOF	Uncoupled single degree of freedom
W	Building self-weight
w	Distributed load
x	Displacement
y(t)	Displacement
μ	Coefficient of friction

CHAPTER I

INTRODUCTION

Blast incidents, which are becoming more common and result from chemical, fuels, and terrorist attacks, can have extremely severe consequences including personnel casualties and financial losses. Blast events considered here include accidental explosion at process plants processing natural gas, chemical/hydrocarbons, fuel tanks, and explosions occurring due to terrorist attacks. In addition to direct effects such as personnel injuries and financial losses, blast events can also have significant and long-lasting psychological effects on societies. In particular, blast events that occur due to terrorist attacks such as September-11 and Oklahoma City Federal Building attacks can profoundly affect societies and change government’s security policies. Today, wide-spreading terror attacks and accidental explosions in the petrochemical industry and industrial plants can have extremely severe consequences including significant financial loses as shown in Figure 1. Therefore, it is becoming almost mandatory to take blast loads, which are generally ignored at design stage by engineers, into account for design of at least critical structural systems.

(a) Texas City (b) Hatay Reyhanlı (c) Ankara Ostim

Figure 1: Some examples of accidental explosions and terror attacks

However, blast resistant structures can be extremely costly due to high magnitude of blast loads. To minimize cost, blast resistant modular steel (BRMS) buildings are becoming common especially in petrochemical industry. The BRMS buildings are recognized as an economical solution to minimize blast effects and protect personnel working near potential explosion sites. These structures are widely used as control rooms, office buildings, and living quarters in areas with a high risk of explosion, fire, or danger from toxic materials in petrochemical industry, and temporary living shelters in military areas. Figure 2 shows BRMS building used as crew shelter at offshore platforms and as control room at petroleum refineries.

Figure 2: BRMS Building at Offshore Platform and Petroleum Refinery

An important design challenge for blast resistant buildings is the extremely high foundation reactions and the requirement for large foundations to anchor structures. Another problem with especially BRMS buildings is that they are typically placed at already existing plants, where digging for such big foundation is not practical. Therefore, to minimize foundation requirements and foundation reactions, sometime BRMS structures are not connected to their foundation for blast loads but only for environmental loads such as seismic and wind loads. This new design approach, which is called “free-to-slide” or “unanchored” foundation is gaining wide acceptance for blast-resistant modular steel structures in the industry.

Although it is known that leaving structures free-to-slide on its foundation will reduce foundation reactions, there is no detailed study giving relative reduction in foundation reaction as well as effects of sliding on blast performance of individual structural members. In this study, the effects sliding foundation on blast performance of a prototype building was determined by comparing blast performance of the structural members with anchored and unanchored foundations. In addition, effects coefficient of friction between the foundation and structure on blast performance of the building and its structural elements was studied.

Blast analysis of such structures is typically done using nonlinear finite element models with dynamic analysis. However, such nonlinear dynamic analysis can be very costly in term of computational requirements when the structure is also sliding on its foundation. Such a modelling analysis requires well trained engineers, who can properly model contact and contact separation and significant computational sources. Therefore, this study also investigated whether conservatively unanchored BRMS structures can be modelled as anchored to determine blast performance of structural members while building sliding acceleration, velocity, and sliding distance are approximated with a rigid body approximation.

Design of structures for blast loading requires a good understanding of nonlinear dynamic behavior of structural members subjected to these loads. For this purpose, two dynamic nonlinear time history analysis methods are widely used. These are three-dimensional (3D) finite element (FE) and uncoupled single degree of freedom (USDOF) method which is also known as simplified spring model method.

FE method is the most suitable analysis method for blast loading. However, the need for high performance computers, high cost of FE software and especially the need for experienced and well-educated engineers are the major disadvantages of this method. On the other hand, USDOF method provides great convenience for blast analysis of structures. In this analysis method, dynamic interaction of structural members under blast loading is ignored, and their dynamic performance is evaluated as uncoupled separate individual elements.

Effects of structural interaction on blast performance of structural members was evaluated by performing blast analysis of an elastic and elastic-perfectly plastic girder-beam system under blast loading. The computed blast performance of girder supported beam were compared with that determined from its uncoupled blast analysis.

1.1 Literature Review

There are numerous studies on blast performance of structures and structural elements as well as analysis method used. However, literature on blast performance of sliding structural systems is very limited.

Blast resistant modular steel-framed buildings are utilized in petrochemical facilities and becoming more common both for turnaround situations and as alternatives to conventional in-place construction These modular buildings are constructed typically using HSS members that 'seismically' compact' per AISC 341 [1]. Summers provided a detailed description of steel modular buildings design approach using both USDOF and FE analysis methods, acceptance criteria for different damage levels and foundation types. It was concluded that although both design approaches can be used, to include tension membrane effects, plastic strain limitations, and geometric and material nonlinearity effects FE analysis method should be preferred. Summers also mentioned that whether a building is anchored or unanchored for blast loads depends on the anticipated use of the building, whether or not potential down time is acceptable, amount of flexibility in the utility connections, and owner's tolerance of risk. Summers also stated that the maximum sliding displacement, velocity, and acceleration of unanchored buildings can be estimated using impulse-momentum first principles, simplified numerical integration methods or finite element analysis. However, these analyses or their results were not included. Finally, it was concluded that permissible sliding displacement is at the order of 300 mm, unanchored buildings have high margin against overturning, and in case of uplift, application of blast pressure on the underside of the building need to be considered.

Blast analysis and design of structures especially those located at petrochemical facilities are done typically based on ASCE guideline "Design of Blast Resistant Buildings in Petrochemical Facilities" [2]. The report provided general guidelines for the structural design of blast-resistant design and includes dynamic material strengths, allowable response criteria, analysis methods, and design procedures.

Erkmen [3] performed FE and USDOF blast analysis for structural members of a two-module steel blast-resistant building to evaluate adequacy of USDOF blast analysis method. Two blast load cases corresponding to building medium damage and high damage levels as defined per ASCE [2] were used for blast analysis. It was found that accuracy of USDOF depends on engineering judgment, and contrary to common assumption that USDOF is a conservative analysis method its results can be unconservative due to structural interaction of connected members. It was also found that dynamic interaction of structural members affects their blast performance by changing the assumed deformed shape for the USDOF analysis and consequently affecting Biggs' factors.

FE blast analysis method is a reliable and powerful approach, and it has been shown that blast performance of structural members determined using FE approach is reliable [1] [2] [3] [4] [5]. However, the need for high-performance computers to solve millions of equilibrium equations, high costs of FE software, and the need for well-trained and experienced engineers to build models, select and perform appropriate analysis method, and interpret the results are significant disadvantages of FE method for blast analysis [3].

The principals of using USDOF blast analysis method were established by John M. Biggs [5] in his book '*Introduction to Structural Dynamics'*. Biggs' showed that a simple spring-mass system can be used to properly predict dynamic behavior of structural elements. He introduced factors known as Biggs' factors to ensure that developed USDOF or spring model has the same total and kinetic energy as the structural element for which the USDOF is developed. The developed USDOF blast analysis method based on Biggs' factors is still being used today to determine blast performance of uncoupled structural elements. Biggs' also performed some limited study on dynamic interaction of multiple degrees of freedom and developing their spring model. He found that if the vibration periods of inter connected members differ by a factor of 2 or more then dynamic interaction of structural members can be neglected and USDOF analysis method can be used to predict their dynamic behavior. However, the study was done on a very limited cases in terms of structural system, boundary conditions, and blast loading.

Yokoyama [6] performed a detailed study using multi-degrees of freedom analysis and USDOF analysis of beams with different loadings and boundary conditions to verify Biggs' factors for USDOF blast analysis. Overall, errors at less than 1% were reported for USDOF analysis with Biggs' factors. However, it should be noted that the study did not include any dynamic interaction of structural members and was limited to uncoupled beams.

Baker et al. [7] also performed dynamic analysis on two-degrees of freedom systems and concluded that the USDOF dynamic analysis method is an approximate but on the safe side analysis method. The USDOF blast analysis method has been studied by many other researcher [2] [4] [8] [9] and shown that it is an approximate analysis method for dynamic analysis of structural systems.

1.2 Goals and Objectives

This thesis aims to expand understanding of blast performance of blast resistant modular steel structures, effects of anchored and uncharred (free-to-slide) foundation, and effects of dynamic coupling (interaction) of structural elements on their blast performance. In summary goal and objectives of this study are;

1. Determine effect of unanchored (free-to-slide) foundation on blast performance of individual structural members as well as horizontal and vertical foundation reactions.

2. Determine whether anchored foundation can be conservatively assumed for

structures with free-to-slide (unanchored) foundation to eliminate high cost of modelling contact and sliding between structure and foundation under dynamic blast loads.

3. Develop and verify a simple and computationally cheap numerical method to predict rigid body sliding motion of structures and their foundation reactions under blast loads.

4. Determine applicability of rigid body approach for structure with unanchored foundation under blast loading.

5. Determine effects of coupling (dynamic interaction) of interconnected elements on their blast performance and evaluate applicability of uncoupled single degree of freedom analysis method for coupled structural members.

1.3 Organization of Thesis

The thesis includes five chapters, “Chapter I” is mainly the introduction and literature review on three main topics. “Chapter II” is on effects of anchored and unanchored foundation types on blast performance of structural members and foundation reactions. “Chapter III” includes rigid body analysis of unanchored structures subjected to blast load, and development and verification of a numerical integration method to predict horizontal acceleration, velocity, and sliding distance and foundation vertical and horizontal reactions of unanchored structures under blast loads. “Chapter IV” includes a parametric study, where effects of dynamic interaction (coupling) of inter connected structural elements is investigated as well as adequacy of uncoupled single degree of freedom system for blast analysis of such systems. At last, “Chapter V” includes summary, conclusions, and future work.

CHAPTER II

EFFFECTS OF UNACHORED FOUNDATION ON BLAST PEFORMANCE OF BRMS BUILDINGS

Blast-resistant modular steel (BRMS) buildings are cost-effective and practical solutions to minimize blast effects and shelter personal in petrochemical and military industries. Typically, BRMS buildings are designed to be fixed (anchored) to their foundation. However, this design approach may result in very large dynamic anchorage and foundations reactions, which in turn result in large and uneconomic foundation sizes. Therefore, an alternative foundation approach which minimize foundation loads and known as “unanchored” or “free-to-slide” foundation has recently attracted much attention in the industry. In this approach, the building and foundation connections are only designed for seismic and wind loads, but they are assumed to break under blast loads allowing the building to slide free on its foundation.

Performing such dynamic finite element sliding analysis with very small-time steps using commercial finite element packages is computationally costly, requires advanced finite element knowledge including dynamic interaction and contact separation between model building and its foundation. Therefore, analysis approaches based on rigid-body motion of the structure are commonly used in industry to predict building velocity, sliding, and its interaction with the foundation. However, to determine blast performance of structural members still finite element analysis of structure under blast loads is needed.

This chapter discusses (1) effects of free-to-slide foundation on blast performance of a prototype BRMS building and its structural members including foundation reactions, and (2) viability of using anchored foundation (i.e., no sliding and no contact interaction) approximation instead of sliding foundation interaction to minimize computational costs for evaluating blast performance of structural members. The objective of this section is to determine whether conservatively anchored foundation can be assumed for BRMS buildings which are unanchored to their foundation. Another objective is to determine how building sliding affects foundation reactions and blast performance of the structure and its structural members.

To achieve above objectives, a prototype BRMS building was developed and its blast performance was evaluated by performing its blast analysis for several cases including anchored foundation and sliding foundation with different friction properties.

2.1 Prototype BRMS Building and Blast Loads

Blast resistant structures can be very costly due to magnitude of blast loads. Therefore, cost effective, portable, and prefabricated BRMS buildings are becoming widespread as control rooms, operator shelters, and office structures especially in petrochemical industry [2]. Figure 3 shows a typical three-module BRMS building during and after field installation. In this study, a prototype BRMS building was designed to evaluate effects of sliding on its blast performance. The prototype BRMS building studied consists of two rigidly connected building modules each with a length of 12 m, width of 3 m, and height of 3.5 m as shown in Figure 4 through 5.

Figure 3: Sample BRMS Prototype Building with three modules

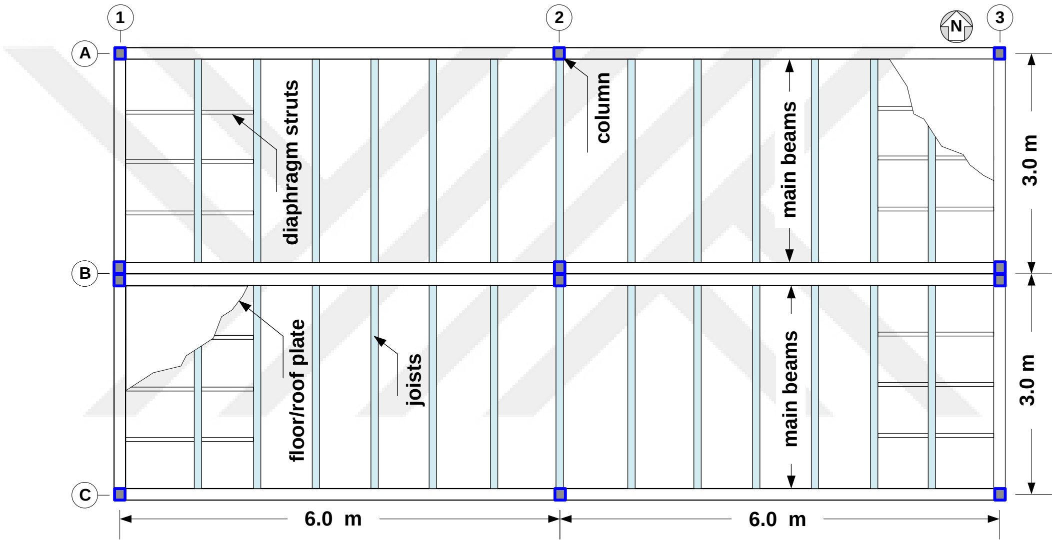
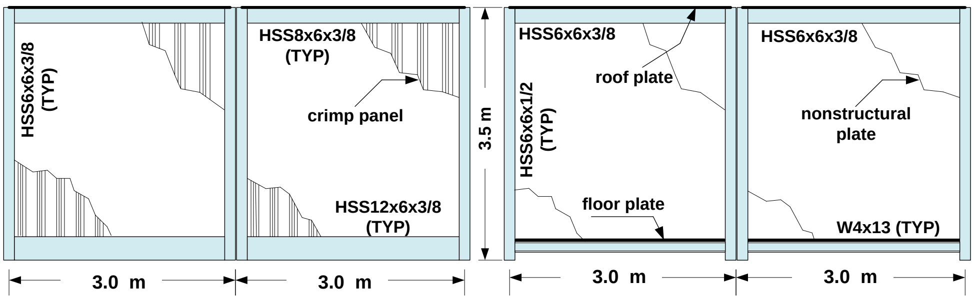


Figure 4: BRMS Prototype Building Floor Plan and Roof Plan



(a) (b)

Figure 5: Elevation Views at (a) column axes 1 and 3 (b) column axis 2

The building frame members were selected to be seismically compact hollow structural sections (HSS) per ASIC [10] so that their full bending plastic capacity can be reached before any local instability. The only non HSS section was the roof and floor joists, which was seismically compact W4x13 I-beams. Flat 5 mm thick steel plates were used for the roof and floor openings while 5 mm thick crimp panels (trapezoidal cross-section) with much higher ductility capacity were used for the walls. Details of structural members used for the building are given by Erkmen [3] and in Table 1.

Table 1: Section and Steel Grade of Structural Members of BRMS Building

Frame Member	Steel Grade	Section
Corner columns (total of 8)	A500 Gr. B	HSS6x6x3/8
Other columns (total of 4)	A500 Gr. B	HSS6x6x1/2
Top perimeter longitudinal main beams (total of 2)	A500 Gr. B	HSS8x6x3/8
Top intermediate longitudinal main beams (total of 2)	A500 Gr. B	HSS8x6x1/2
Top perimeter transverse main beams (total of 4)	A500 Gr. B	HSS8x6x3/8
Top intermediate transverse main beams (total of 2)	A500 Gr. B	HSS6x6x3/8
Bottom longitudinal main beams (total of 4)	A500 Gr. B	HSS12x6x3/8
Bottom transverse main beams (total of 4)	A500 Gr. B	HSS12x6x3/8
Roof and floor joists	A992 Gr. 50	W4x13
Roof and floor diaphragm struts at module ends	A500 Gr. B	HSS3x2x3/16
Roof and floor steel plate	A36	5 mm thick
Crimped steel wall plate	A36	5 mm thick and 100 mm deep

2.2 Blast Loads

The blast source was assumed to be on the building north side with blast wave directly striking its broad side. Blast loads are usually described in term of free field overpressure known as incident or side-on overpressure, which is the blast pressure before it is reflected from any surface. Typical blast peak side-on overpressure due to explosions in petrochemical industry is between 10-100 kPa with duration of 20-200 ms [1]. The blast pressure was assumed to have a triangular shape with a maximum peak side-on pressure of 80 kPa and duration of 140 ms. The selected blast load and its duration correspond to BRMS prototype building high damage level. Different blast damage levels are described for structures in petrochemical industry, and details of these damage levels are given in several references [2] [11]. The interaction of blast waves with building walls and roof is pretty complex. Therefore, a simplified method proposed by ASCE [2] was used to determine blast pressure curves, which are given in Figure 6, for each wall and roof as well as free-field overpressure, which is pressure of blast wave before it hits any surface.

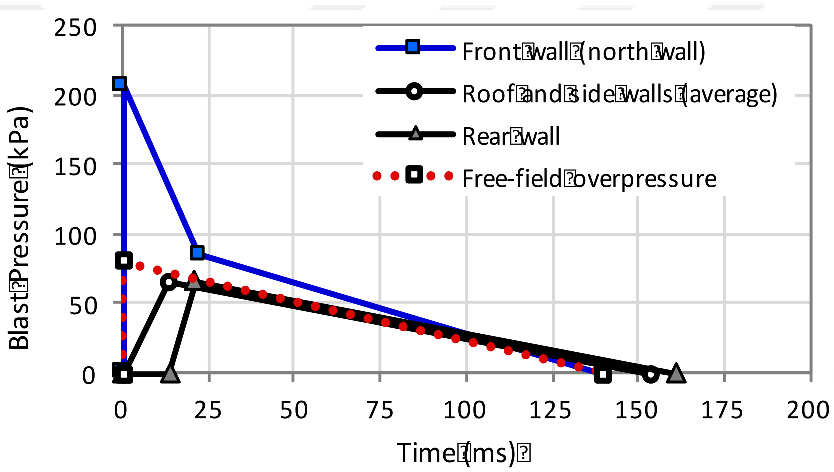


Figure 6: Free-field and Individual Wall and Roof Blast Pressures

2.2.1 Finite Element Model

The FE blast model of the building was developed using ABAQUS 6.14 general-purpose finite element software. The three-dimensional model shown in Figure 7 consists of approximately 40,000 nodes and 38,500 elements. All frame elements were modelled with Timoshenko B31 beam element, which is suitable for both thick and slender beams. All steel plates including crimp wall panels were modelled using S4R, which is a 4-node, quadrilateral shell element with reduced integration and large-strain formulation. A nominal mesh size of 300 mm was used for all elements. Elastic-perfectly plastic material model was assumed for all steel members.

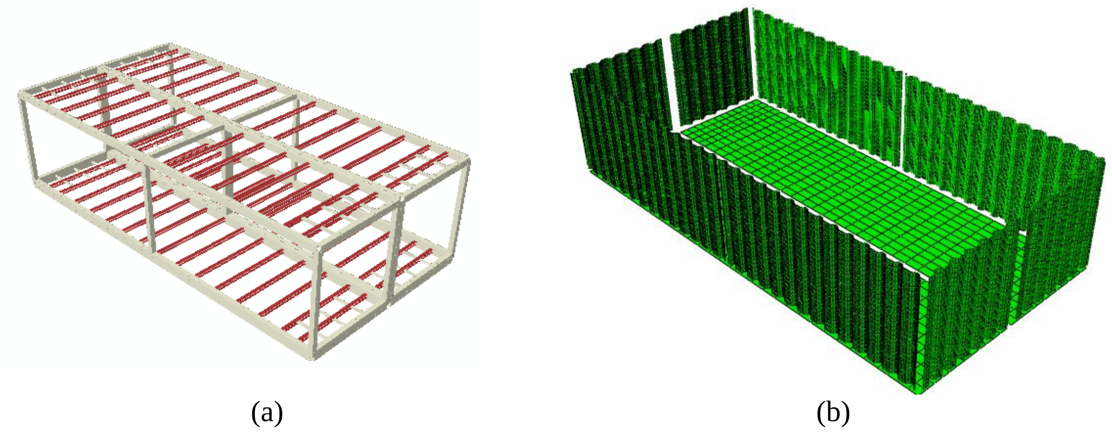
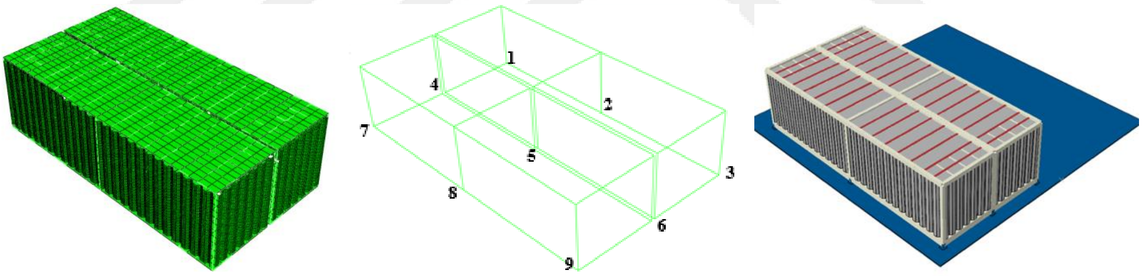


Figure 7: FE Model, (a) Frame elements (beams, column, joist, and girts), (b) Wall and floor elements (roof plate not shown for clarity)

To account for average yield strength of steel being greater than those specified in applicable standards and increase in yield strength due to high strain rate under blast loading, static yield strength was increased by strength increase factor (SIF) and dynamic increase factor (DIF). The values of SIF and DIF for different steel grade and loading were obtained from ASCE [2]. Because the wall and roof plates are only 5 mm in thickness, the main mechanism that they resist out of plane blast loads is membrane action. In addition, due to trapezoidal cross-section of the wall panels (i.e., crimped panels), the effect of nonlinear geometry (i.e., membrane action) is also important, and it was included. Blast analysis of the structure was completed in two steps. Initially, the static loads (i.e., self-weight) were applied, and the blast loads on individual walls and roof were applied at the subsequent implicit dynamic analysis step. No blast load was applied to short side walls since the loads on these walls does not affect sliding behavior of the structure.

Two types of boundary conditions were considered for the BRMS building based on whether the building is anchored or unanchored (free-to-slide) to its foundation as shown in Figure 8b & c. For the anchored foundation case, the structure was assumed to be anchored (pinned boundary condition) to its foundation at nine anchorage points located under the columns as shown in Figure 8b. For unanchored foundation, the building was placed on a rigid foundation and contact interaction was defined between the building column end-plates and rigid foundation. The defined contact interaction between structure and foundation allows separation of the building from its foundation (uplift). For horizontal motion of the building, static and dynamic friction coefficients for interaction between the building and its foundation were assumed to be equal, and three values of friction coefficient 0.2, 0.5 and 0.8 were considered for blast analysis of the building.



(a) (b) (c)

Figure 8: FE Model, (a) Whole Building, (b) Anchored Supports, and (c) Unanchored (free-to-slide) on Rigid Foundation

2.3 Analysis Results and Discussion

2.3.1 Foundation Reactions

One of the main reasons using sliding foundation for BRMS or blast resistant structures is the large magnitude foundation reactions and the corresponding large foundation size required. Table 2 shows maximum value of horizontal and vertical reactions for anchored and three unanchored (free-to-slide) foundation cases for the prototype BRMS building. The results show horizontal foundation reactions significantly decreased from 9203 kN to as much as 1348 kN when the structure was allowed to slide over the foundation. In other words, the horizontal foundation reaction decreased to 60% to 15% of that for anchored case when the structure was free to slide. Considering that reasonable value of coefficient of friction is approximately 0.5, the corresponding horizontal foundation reaction dropped to 37% of that corresponding to the case with anchored foundation. This finding shows why industry prefer to not connect such structures to expensive foundations when it is allowed or possible. On the other hand, effects of foundation type on maximum foundation vertical reaction were very limited. Foundation maximum vertical reaction dropped only between 2% to 4% of that corresponding to the anchored foundation case.

Table 2: Summary of Foundation Reactions

Foundation Type	**Anchored**	**Unanchored**					
		μ=0.8		μ=0.5		μ=0.2	
	R^* (kN)	R (kN)	R (%)	R (kN)	R (%)	R (kN)	R (%)
Maximum Horizontal Reaction, kN	9203	5480	60	3399	37	1348	15
Maximum Vertical Reaction, kN	7008	6856	98	6806	97	6750	96

R^* and R are the maximum total foundation reaction and R (%)= $(R/R^*)100$

Figure 9 and 10 shows history of foundation vertical and horizontal reactions with time, respectively. The maximum foundation vertical reaction occurs at the beginning of blast loading as expected and decreases significantly over a short period of time. The foundation type has negligible effects on vertical reaction both in terms of its magnitude and time it occurs. The horizontal reaction histories show that horizontal maximum reaction again occurs at the beginning of blast loading but having sliding foundation slightly delays the time that maximum reactions occurs. In addition, results show that even providing limited sliding (μ =0.8), the foundation horizontal reaction decreases significantly. This finding shows that if a foundation mechanism such as a foundation connection with a limited gap distance (e.g., slotted pin connection) or similar mechanism is provided to limit the sliding magnitude but allow some initial sliding, the foundation horizontal reaction can be significantly reduced while also the maximum sliding distance is controlled.

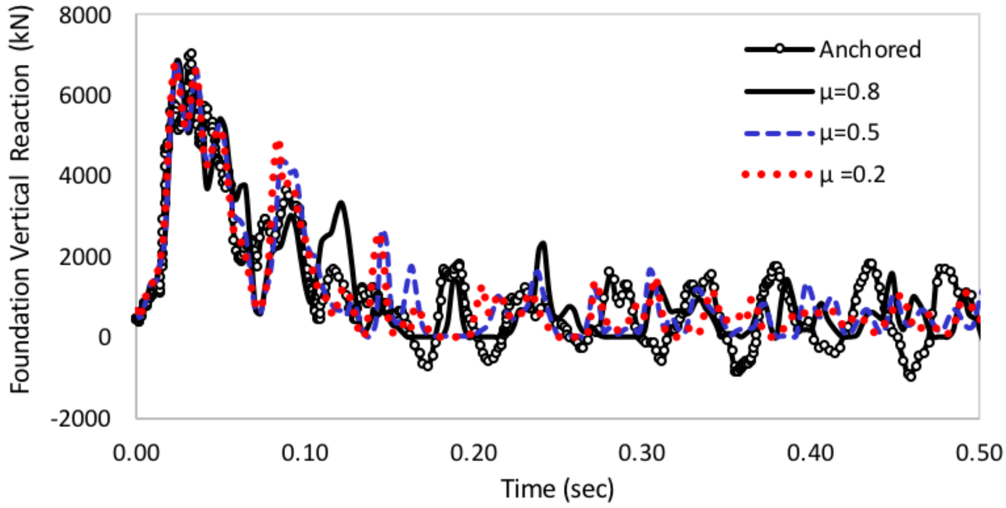


Figure 9: Foundation Vertical Reaction

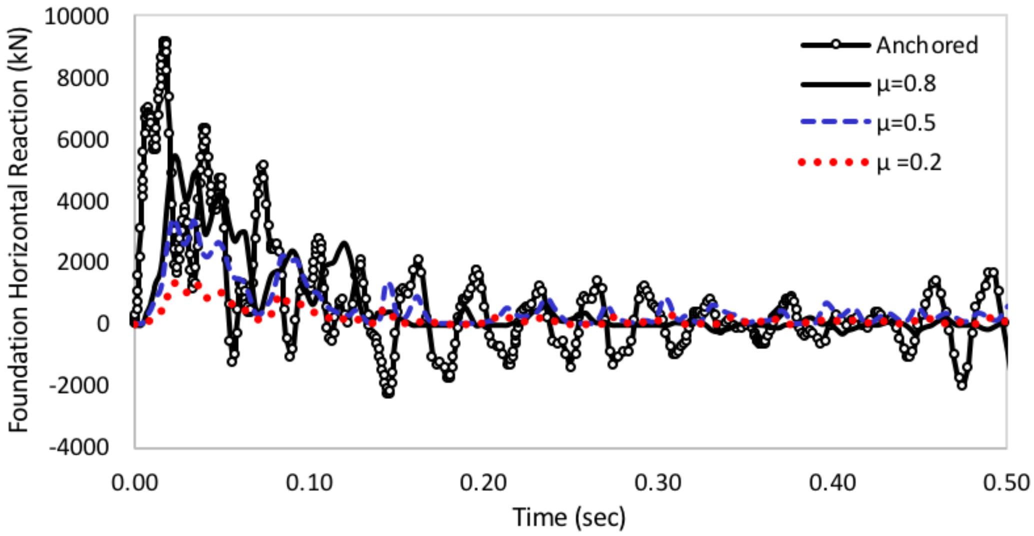
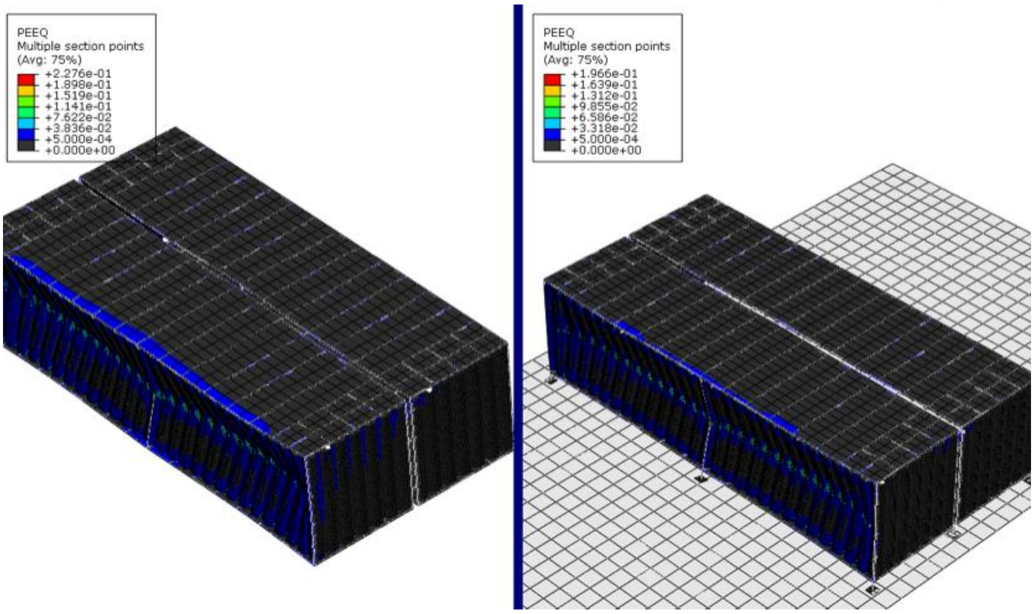


Figure 10: Foundation Horizontal Reaction

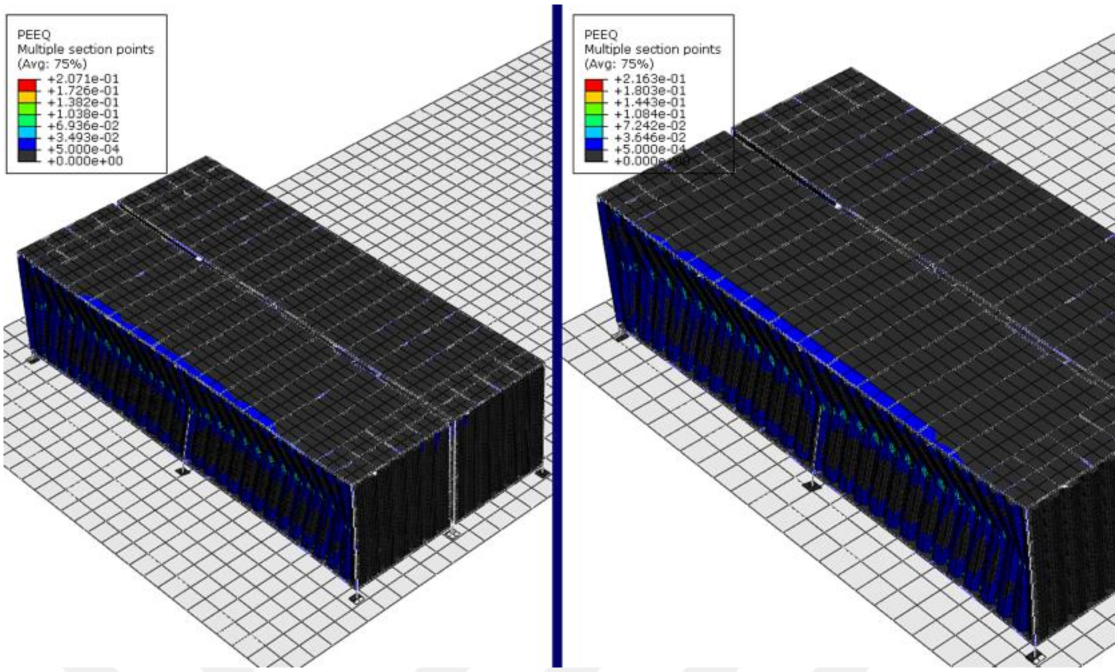
2.3.2 Structural Member Blast Performance

The effects of foundation type on blast performance (i.e., maximum ductility, deflection, support rotation) of the structural members is not fully cover in the literature. An important indication of structural damage is equivalent plastic strain (PEEQ), which allows to characterize at the measurement moment the structural state of the material that was formed as a result of the previous loading. In other words, PEEQ is the integral of plastic strain/deformation (accumulated plastic strain), and it is a good indication of failure [12]. Figure 11 and 12 shows the PEEQ at the end of blast analysis for four foundation conditions considered. The PEEQ values computed are 0.23, 0.20, 0.21, and 0.22 for anchored foundation and foundation with coefficient of friction of 0.2, 0.5, and 0.8, respectively. In other words, the structural damage was highest for anchored foundation while the damage decreased as coefficient of friction decreased. In other words, increased sliding decreased the overall structural damage under blast loading.



(a) (b)

Figure 11: Equivalent Plastic Strain (PEEQ) (a) Rigid Foundation (b) Sliding Foundation μ=0.2)



(a) (b)

Figure 12: Equivalent Plastic Strain (PEEQ) (a) Sliding Foundation μ=0.5 and (b) Sliding Foundation μ=0.8

Blast response of individual structural members is typically expressed in term of ductility and support rotation, which are function of deflection [1] [2] [10]. Because the only difference between the cases studied is foundation type, and same structural members are used for all cases, individual structural members blast performance is measured in terms of their maximum deflection instead of ductility and support rotation. For this purpose, the structural members with maximum deformation were determined for the structure with anchored foundation, and the exactly the same members were monitored for cases with sliding foundation for consistency. In addition, because the deflections of structural members with sliding foundation also have a component due to sliding displacement, the deflection histories of these members were corrected for their support displacement. Deflection histories of these members were computed by subtracting the average support displacement from the displacement of location along the member, where maximum deflection occurred with anchored foundation. This procedure ensured consistency of deflections of structural members for anchored and sliding foundation cases.

Computed maximum relative deflections of the structural members are given in Table 3 for each foundation case. The results indicate that blast performance of structural members improved (i.e., maximum deflection decreased) as the foundation sliding increased. In other words, demand for maximum deflection of structural members decreased as coefficient of friction between the structure and foundation decreased. The only exception was free-to-slide structure roof intermediate transverse beam, maximum deflection of which was between 93% and 120% of the that computed with anchored foundation.

The ratio of maximum relative deflection of the members for the building with free-to-slide foundation to the corresponding deflections of anchored foundation was between 59% and 120% (83% average) for μ =0.8, 52% and 107% (82% average) for μ =0.5, and 43% and 93% (77% average) for μ =0.2. In other words, the deflection demands of the structural members between approximately 20% to 25%. However, roof intermediate transverse beam was the only structural element with increased deflection demand when foundation was allowed to slide. The increased in defection demand of this member is likely due to complex dynamic interaction of structural members under blast loading. When deflection demand of this element is excluded, the average deflection demand for structural element decreased to 74% to 77% for structure with sliding foundation. This finding shows that providing even limited sliding between structures and their foundation improves blast performance of structural elements as much as 25%.

Table 3: Summary of Maximum Deflection Demand for Structural Elements

Structural Member	Anchored	Unanchored μ=0.8		Unanchored μ=0.5		Unanchored μ=0.2	
	Δ^* (mm)	Δ (mm)	Δ (%)	Δ (mm)	Δ (%)	Δ (mm)	Δ (%)
Roof joist	32	19	59	23	72	24	75
Roof plate	21	11	52	11	52	9	43
Roof intermediate transverse beam	15	18	120	16	107	14	93
Roof intermediate longitudinal beam	135	121	90	120	89	118	87
Broad side crimp wall	609	517	85	505	83	495	81
Center broadside column	408	367	90	355	87	346	85

Δ^* and Δ are maximum absolute relative deflections and Δ (%)$_=$ (Δ /Δ^*)100

The deflection histories corrected for support displacements are given Figure 13 through 19 for structural members for all four foundation cases. The results. The results show that maximum deflection typically occurs at early stages of the loading, and that is why providing some kind of damping does not significantly improve blast performance of structures. The results show that not only the maximum deflections decreased but also deflections remain smaller with free-to-slide foundation. The deflection history of roof intermediate transverse beam given in Figure 15 shows that not only the maximum deflection increased with free-to-slide foundation but also remain higher than that of anchored foundation throughout analysis.

In conclusion, the analysis results indicate that allowing structure to slide even a limit distance on its foundation improves structural members blast performance by 25%. This finding supports the conclusion in Section 2.3.1 that a foundation mechanism such as a foundation connection with a limited gap distance (e.g., slotted pin connection) or similar mechanism to limit the sliding magnitude but allow some initial sliding can significantly improve blast performance of structural members while the maximum sliding distance is controlled.

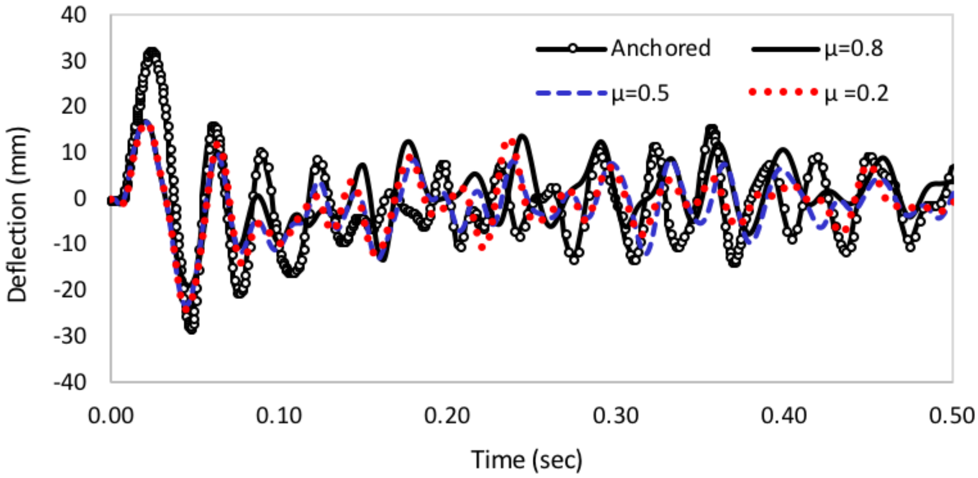


Figure 13: Roof Joist Deflection History

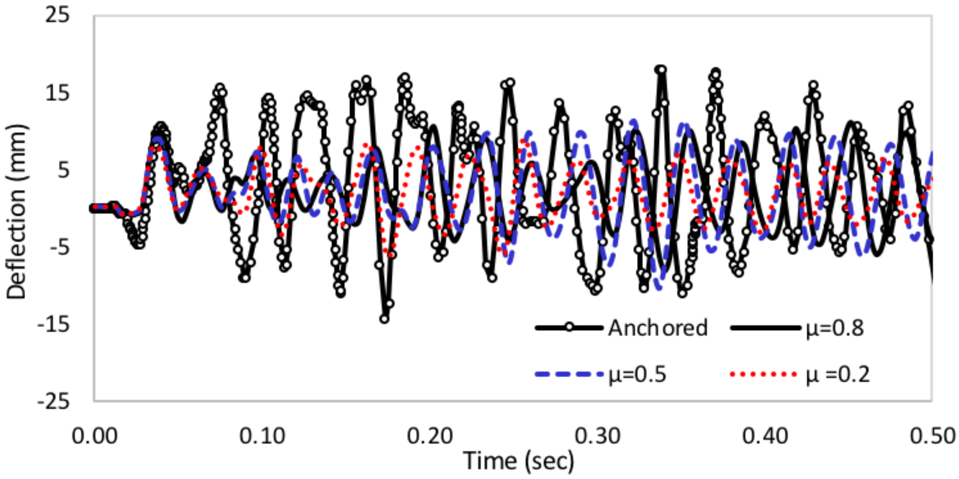


Figure 14: Roof Plate Deflection History

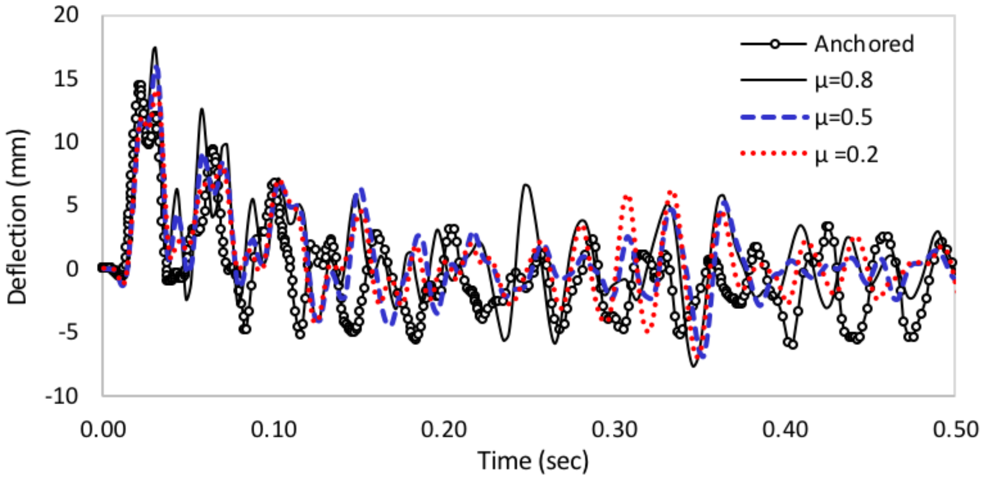


Figure 15: Roof Intermediate Transverse Beam Deflection History

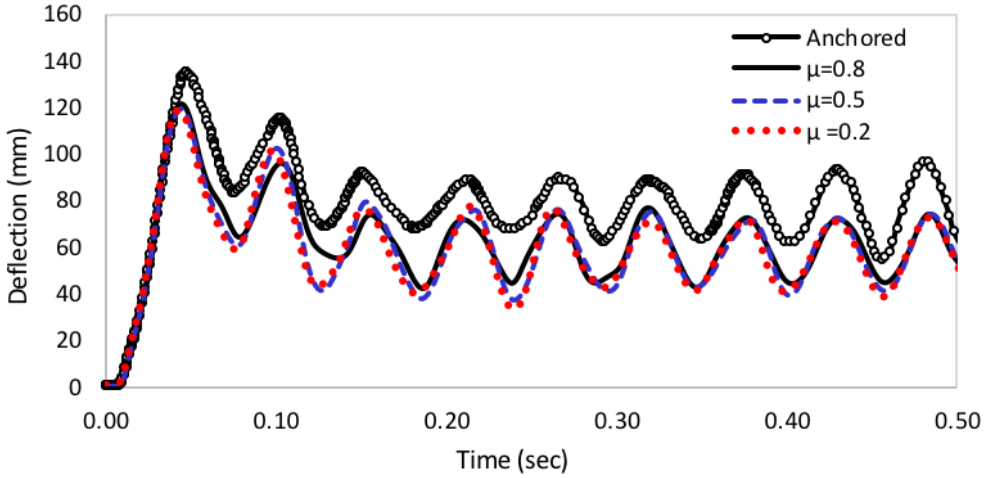


Figure 16: Roof Intermediate Longitudinal Beam Deflection History

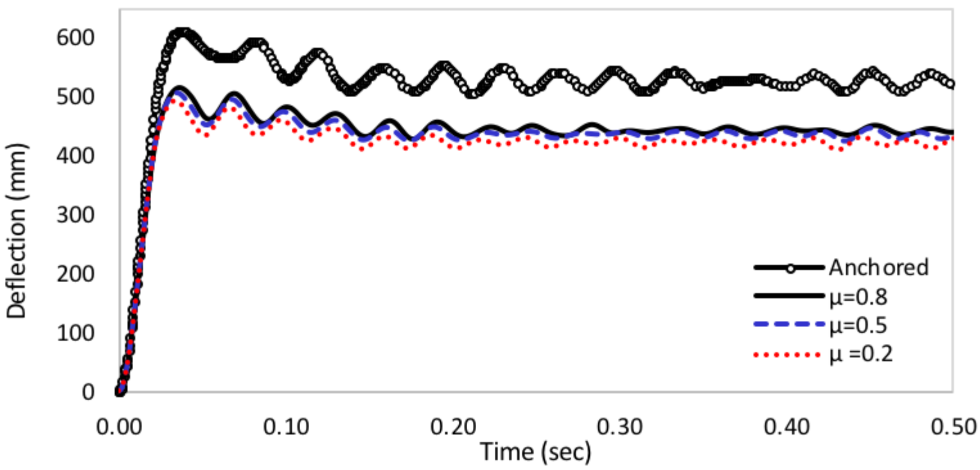


Figure 17: Crimp Wall Panel Deflection History

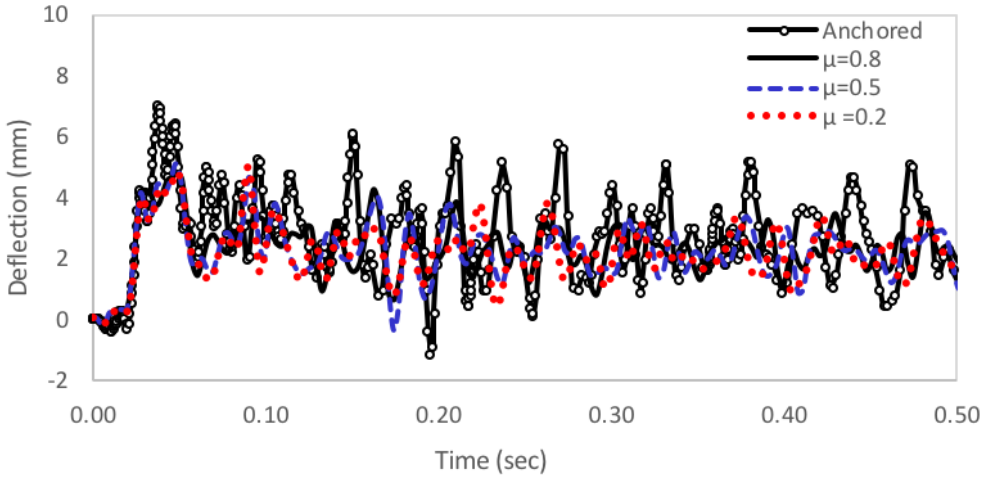


Figure 18: Roof Strut Deflection History

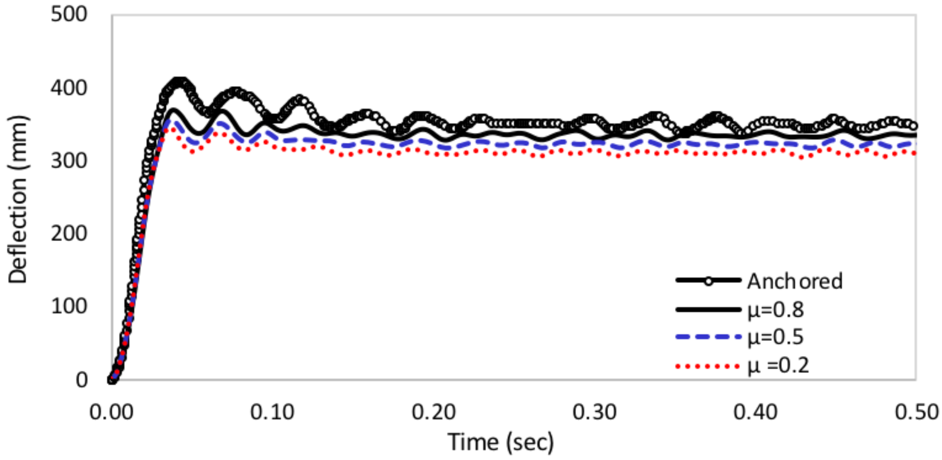


Figure 19: Broadside Center Column Deflection History

Figure 20 and 21 show total friction and kinetic energy histories for the whole structure. The friction energy histories show that the time it takes for the structure to dissipate kinetic energy through friction is very similar for the foundations with coefficient of friction (i.e., μ) 0.5 and 0.8. When friction energy of foundation with 0.5 coefficient of friction reached its maximum value, approximately 80% of maximum friction energy of that with coefficient of friction 0.5 was reached. This observation shows that a friction coefficient at the order of 0.5 may be sufficient for such structures. For structure with 0.2 coefficient of friction, the friction energy took much longer to reach to its maximum values (i.e., longer sliding). Figure 21 shows the kinetic energy histories for the building for each case. For all cases, maximum kinetic energy values were reached at the early stages of the loading (i.e., before 0.25 seconds). This further indicates that a mechanism dissipating energy at the early stages of loading will improve blast performance of the structure.

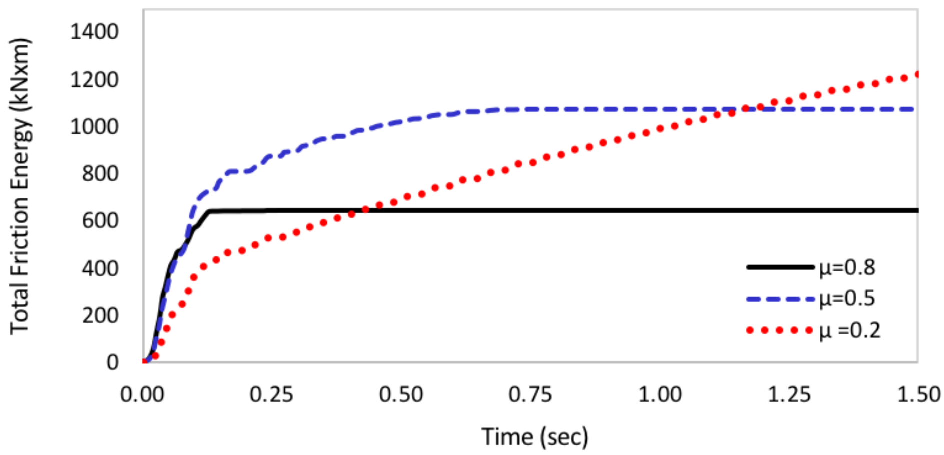


Figure 20: Total Friction Energy History

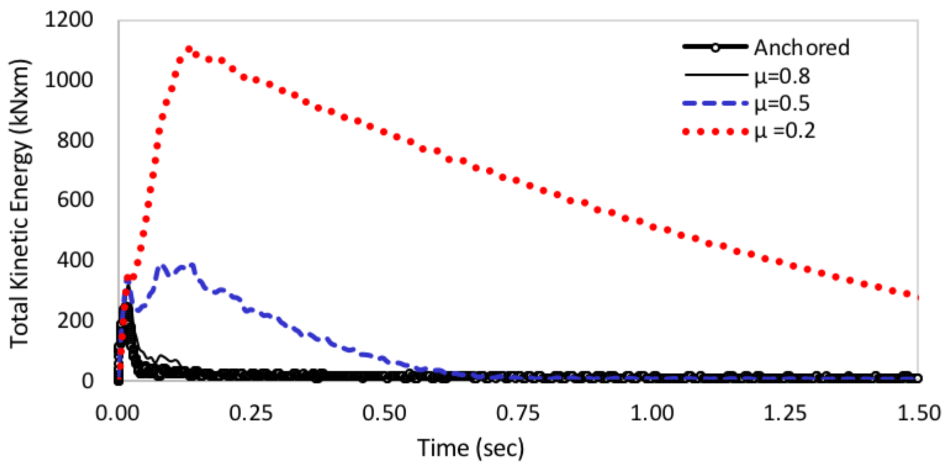


Figure 21: Total Kinetic Energy History

CHAPTER III

RIGID BODY SLIDING ANALYSIS FOR UNACHORED STRUCTURES SUBJECTED TO BLAST LOADING

Typically, Blast-resistant modular steel (BRMS) buildings are designed to be fixed (anchored) to their foundation. However, this design approach may result in very large dynamic anchorage and consequently foundations reactions, which in turn result in large and uneconomic foundation sizes. Another issue is related to excavation and foundation work that will be needed especially when BRMS structures are used for petrochemical facilities which are already running. Preforming earthwork or excavation in such facilities is typically not preferred if possible. Therefore, an alternative foundation approach, which minimize foundation work, loads, and known as “unanchored” or “free-to-slide” foundation, has recently attracted much attention in the industry. In this approach, the building and foundation connections are only designed for seismic and wind loads, but they are assumed to break under blast loads allowing the building to slide free on its foundation.

Performing such dynamic sliding analysis with very small-time steps using commercial finite element packages is computationally costly, requires advanced finite element knowledge including dynamic interaction and contact separation between model building and its foundation. Therefore, analysis approaches based on rigid-body motion of the structure are commonly used in industry to predict building velocity, sliding, and its interaction with the foundation.

This chapter discusses (1) development of a simple but computationally cheap and effective numerical integration method to predict overall blast performance of structures (i.e., velocity, acceleration, displacement, and foundation reactions) with free-to-slide foundation, (2) verification of developed numerical method using finite element results for rigid body sliding motion, and (3) accuracy evaluation of rigid body approximation for sliding allowed deformable structures subjected blast loading.

3.1 Numerical Integration Method (Rigid Body Approach)

A nonlinear numerical integration method was developed to predict velocity, acceleration, displacement, and foundation reactions based on assumption that the structure deformation does not affect the overall motion of the structure. In other words, the structure is assumed to be rigid under applied blast loads. This assumption is typically considered to be conservative since the energy dissipated through member deformation (plasticity) is conservatively neglected.

Figure 12 shows free-body diagram of a blast resistant building under blast loads. The blast loads considered are front wall, roof, and rear wall blast-pressure versus time data. The blast loads on other two walls are not considered since those loads do not affect motion of the structure in "X" direction considered. Other loads considered in addition to blast loads are structure self-weight and dynamic interaction between the structure and its foundations (i.e., normal load and friction load).

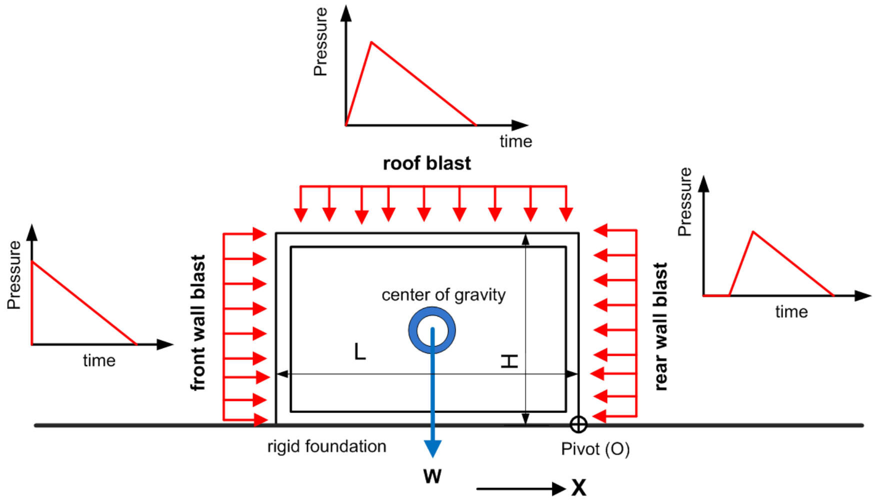


Figure 22: Free-body Diagram of Building Under Blast Loads

The system of forces on the building consists of front face (P_{Front} (t)), roof (P_{Roof} (t)) and rear wall (P_{rear} (t)) blast pressures, which vary with time, and self-weight of the building (W), which is a constant vertical load. The net vertical force $F_V(t)$ on the building is given as;

$$F_V(t) = W + P_{Roof}(t)A_{Roof} \tag{3. 1}$$

where A_{Roof} is the total roof surface area of the building. The net horizontal force $F_H(t)$ is calculated as the sum of the front and rear wall loads plus a friction force equivalent to the product of net vertical force and friction coefficient (μ). The net horizontal force is given as;

$$F_H(t) = P_{Front}(t)A_{Front} + P_{Rear}(t)A_{Rear} + \mu F_V(t) \tag{3. 2}$$

Typically, the front wall and rear wall blast surface areas (A_{Front} and A_{Rear}) are equal, and front and rear wall blast pressures act in opposite directions while the friction force ($\mu F_V(t)$) always opposes the direction of the motion. The instantaneous horizontal acceleration of the building is given as;

$$\ddot{x}(t) = F_H(t)/(W/g) \tag{3. 3}$$

where g is the gravitational constant. The instantaneous horizontal velocity ($\dot{x}(t)$) and displacement ($x(t)$) are calculated using central difference integration scheme as;

$$\dot{x}(t) = \dot{x}(t - \Delta t) + \ddot{x}(t)\Delta t \quad \& \quad x(t) = x(t - \Delta t) + \dot{x}(t)\Delta t \tag{3. 4}$$

where Δt is the chosen time step. Typically blast load durations are between 10 and 300 milliseconds, and time step selected is much smaller. The steps described above were placed in a loop that progress through time at a fixed time step until the desired analysis duration is reached.

The overturning and restoring moments are functions of building geometry, weight, and assumed position of building center of gravity. Both moments are independent of coefficient of friction between the building and its foundation. The overturning moment $M_0(t)$ is the net moment of the front and rear face loads relative to the base of building about the edge that rotation occur in clockwise direction, and it is given as;

$$M_0(t) = BH\left(P_{Front}(t) + P_{Rear}(t)\right)\frac{H}{2} \tag{3. 5}$$

where *H* and *B* are building height and width dimensions, respectively. The front and rear wall pressures directions have opposite sign, and pressure was assumed to be uniform along building height. On the other hand, the restoring moment is composed of moment due to building self-weight and roof blast pressure. For cases with roof pressure having negative pressure (i.e., suctions), the moment due to this negative pressure will not contribute to the restoring moment but rather to the overturning moment. The restoring moment for building with center of gravity at its geometric center is given as follows for roof blast loads with no negative phase;

$$M_R(t) = W\frac{L}{2} + BL \times P_{Roof}(t)\frac{L}{2} \tag{3. 6}$$

where L is the width of the building in the direction of blast load. It should be noted that if the overturning moment exceeds restoring moment, this does not imply instability, but implies that uplift may occur for a short period of time. A crude estimation on the level of uplift can done by assuming the net moment generates a rotational acceleration, which can be integrated in the same way as the translational motion to calculate a rotational velocity and displacement as;

$$
\begin{aligned}
\ddot{\theta}(t) &= \left(M_0(t) - M_R(t) \right)/I \\
\dot{\theta}(t) &= \dot{\theta}(t - \Delta t) + \ddot{\theta}(t)\Delta t \\
\theta(t) &= \theta(t - \Delta t) + \dot{\theta}(t)\Delta t
\end{aligned} \tag{3. 7}
$$

where I is building inertia about its center of mass. Verification of developed numerical approach was done by using finite element sliding results of a prototype building.

3.2 Verification of Numerical Method

The developed numerical model was verified using the finite element model developed for prototype BRMS building discussed in Chapter 2. However, the FE model material properties were modified such that the structure will behave as a rigid body under applied dynamic blast loads. For this purpose, steel material was assumed to be linear elastic (i.e., no yielding) and its modulus of elasticity was increased 100 times. Two FE analysis of the building were performed. The first analysis was a static analysis with the building being anchor to the rigid floor and all blast loads applied as static loads. This model was used to obtain support reaction and to accurately determine blast loads applied. The applied net horizontal and vertical blast loads are given in Figure 23. The vertical loads include the blast load and structure self-weight.

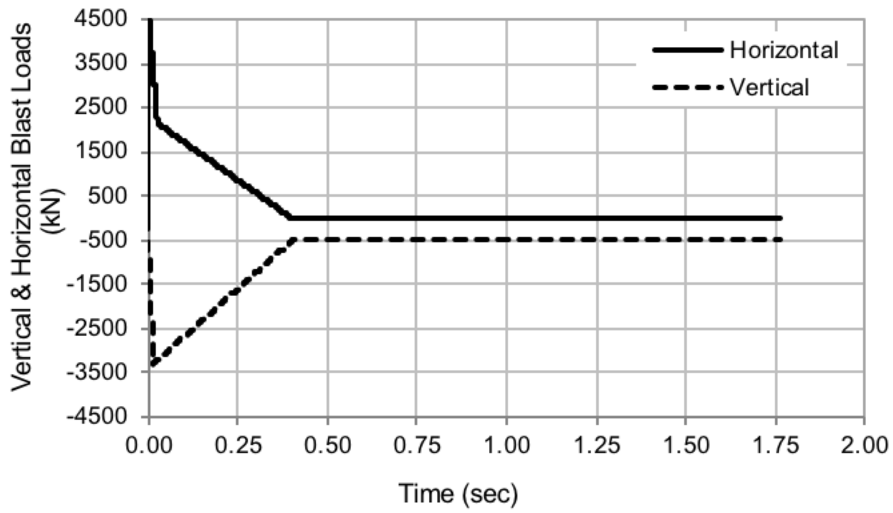


Figure 23: Total Vertical and Horizontal Blast Loads

The second analysis was done for dynamic blast analysis of the rigid structure on rigid base. Both static and dynamic friction coefficients were assumed to be equal to 0.55. The computed and calculated histories of friction force, horizontal acceleration, horizontal velocity, and horizontal displacement are given in Figure 24 through 26. The acceleration histories given in Figure 24 are in very good agreement. However, the FE results show that the structure has some horizontal vibration occurring while the numerical method is not capable of capturing this vibration behavior. This vibration behavior is considered to be due to structure and foundation being rigid, and it is not considered to be realistic.

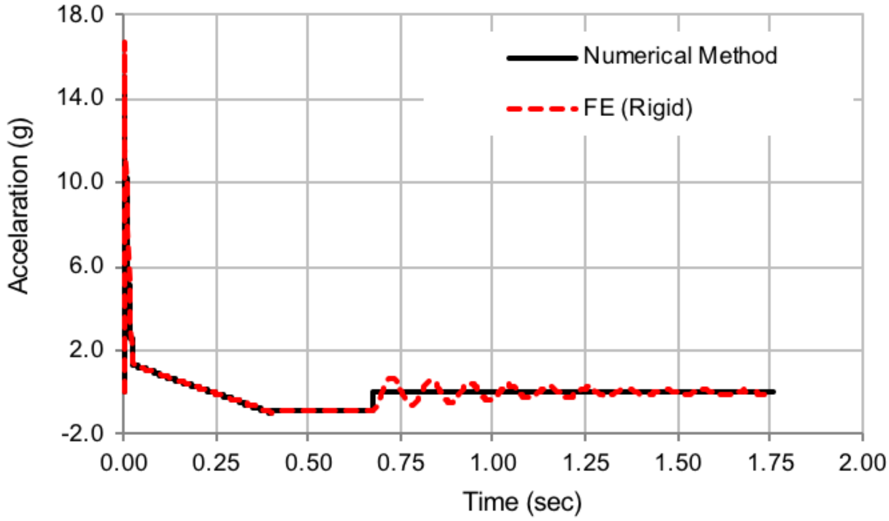


Figure 24: Horizontal Acceleration Histories

Figure 25 shows velocity histories, which precisely match. The FE velocity results show vibration of the structure after numerical method shows that it is not moving at all. Figure 26 shows that horizontal displacement histories precisely match throughout the analysis. Typically, maximum velocity, acceleration, and displacements values during sliding are the important performance parameters for blast resistant structures since they indicate the potential of impact/damage and personal injury due to possibility of interaction with the structure and fixed equipment. In conclusion, the results show that developed numerical method can be safely used to compute sliding behavior of structure under blast loading.

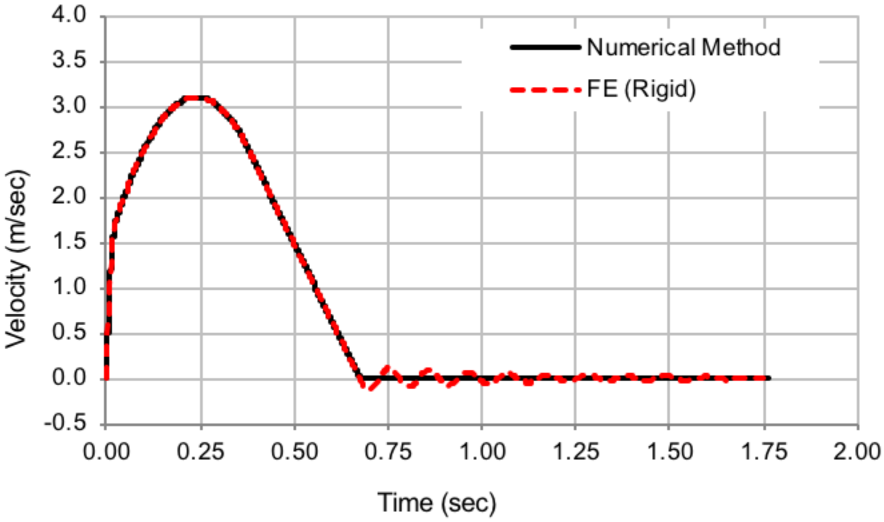


Figure 25: Horizontal Velocity Histories

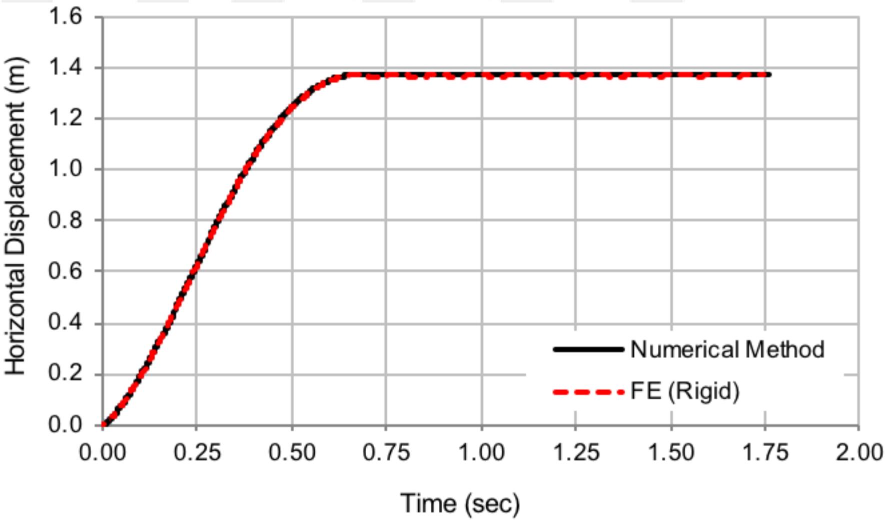


Figure 26: Horizontal Displacement Histories

3.3 Rigid Body and FE Blast Analyses of BRMS Building with Sliding Foundation for Sliding Motion and Foundation Reactions

The developed numerical method to predict foundation reactions and sliding acceleration, velocity, and distance of BRMS buildings is based on assumption that the structure behaves like a rigid body. In other words, the deformation of structure and its structural elements is neglected. However, typically these structures are designed to go under significant structural damage (plastic deformation) to minimize the associated construction cost [3]. Therefore, a part of structure's kinetic energy is dissipated by the structural elements while they go under cyclic plastic deformation. Based on this assumption, it is theoretically believed that the rigid body approximation is a conservative approach to predict building overall sliding motion under blast loads. However, to best of author knowledge there is no study documenting the magnitude of error associated with the rigid body approximation.

To determine the magnitude of error associated with the rigid body approximation, the prototype BRMS building discussed in Section 2.1 was used with the blast loads given in the same section. The blast performance of BRMS prototype building with several foundation conditions was determined using three-dimensional deformable FE models of the building. For all structural members elastic-perfectly plastic steel material properties were used to appropriately capture the effect of energy dissipation through plastic deformation. Both the dynamic and static coefficient of friction for interaction between the structure and its foundation were assumed to be equal. For friction between the structure and its rigid foundation, the coefficient of friction was assumed to be 0.2, 0.5 and 0.8. Typical value of static friction coefficient between concrete and steel is between 0.40 and 0.50 per various references [2], and a little less for dynamic friction coefficient. Therefore, friction value of 0.5 can be considered to be realistic while value of 0.2 and 0.8 were selected to further evaluate applicability of rigid-body analysis approach to predict gross structural behavior under blast loading.

The FE and rigid-body (numerical method) sliding blast analysis results of the prototype building for coefficient of friction 0.2, 0.5, and 0.8 are given in Figure 27 through 29 in terms of vertical reaction, horizontal acceleration, velocity, and acceleration.

The building vertical foundation reaction, which directly related to friction force through coefficient of friction, and horizontal acceleration histories predicted with the rigid-body analysis were smaller than those computed with the FE analysis. The main reason for FE results being much higher is the vibration of the deformable structure under dynamic blast loads, which is not captured with rigid body approach. The vibration of the structure can be clearly identified from the history of vertical force, which oscillates after blast load drops to zero. It should be noted that, determination of structure's horizontal acceleration from FE analysis is challenging due to deformation and vibration of the structural members. The reported FE horizontal structural accelerations are at those at the base of structure's perimeter column, which are laterally supported by structure's wall. The computed horizontal accelerations using FE and rigid body analyses are at the order of 10g to 20g, which are unacceptable for personal in the structure. However, these high accelerations occur over a very short period of time, and then they drop to more reasonable levels.

Despite the significant difference in horizontal accelerations predicted using both methods, the rigid-body analysis results for both building horizontal velocity and displacement histories are in good agreement with those computed using FE analysis especially for coefficient of friction values 0.2 and 0.5. However, as the value of coefficient of friction increases (i.e.,0.8), the FE computed maximum horizontal velocity was higher than that predicted with rigid body approach. Again, this difference is considered to be due to building vibration under dynamic blast loads. On the other hand, it should be noted that a value of 0.8 for coefficient of friction is not realistic.

The magnitude of horizontal sliding is one of the most important design consideration when a structure is designed to be free-to-slide since the amount of flexibility on the utility connections (power, water, wastewater, and gas) is limited and affects the down time. The results show that sliding distance of the building predicted with rigid body approximation in very good agreement with the FE results. In other words, predicted sliding histories and magnitudes with rigid body approximation are on the conservative side, which shows that rigid body approximating can be used to predict building sliding.

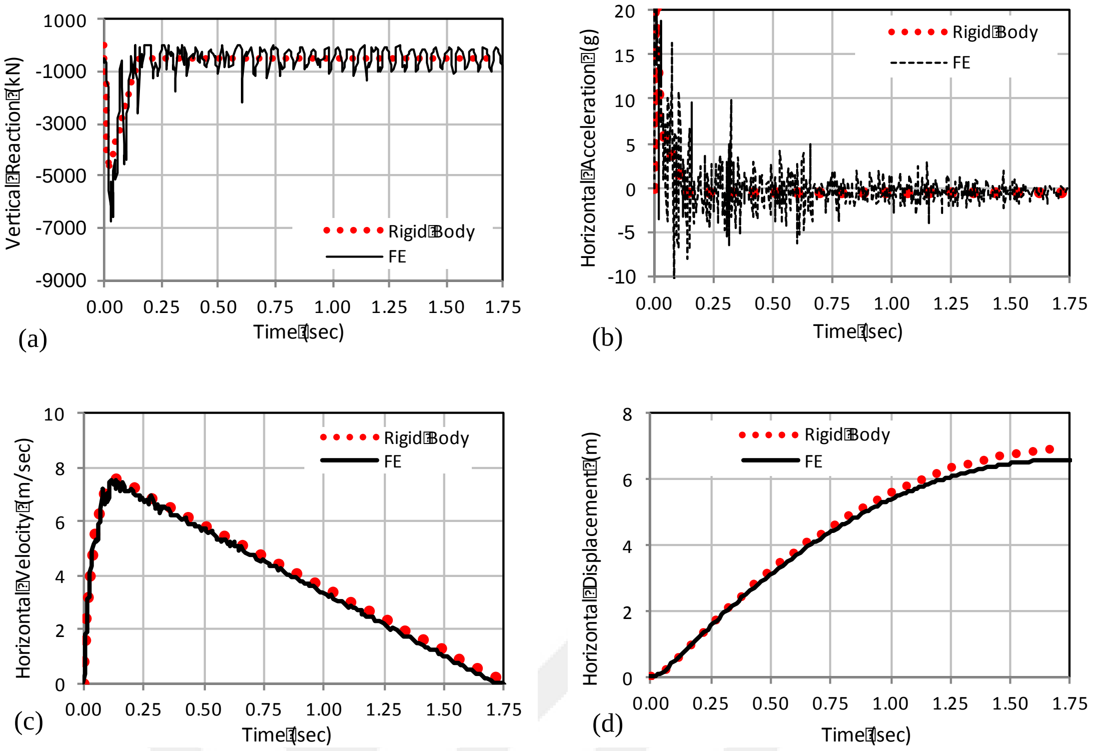


Figure 27: Building Sliding with μ=0.2 (a) Vertical Reactions, (b) Horizontal Acceleration, (c) Horizontal Velocity, and (d) Horizontal Displacement

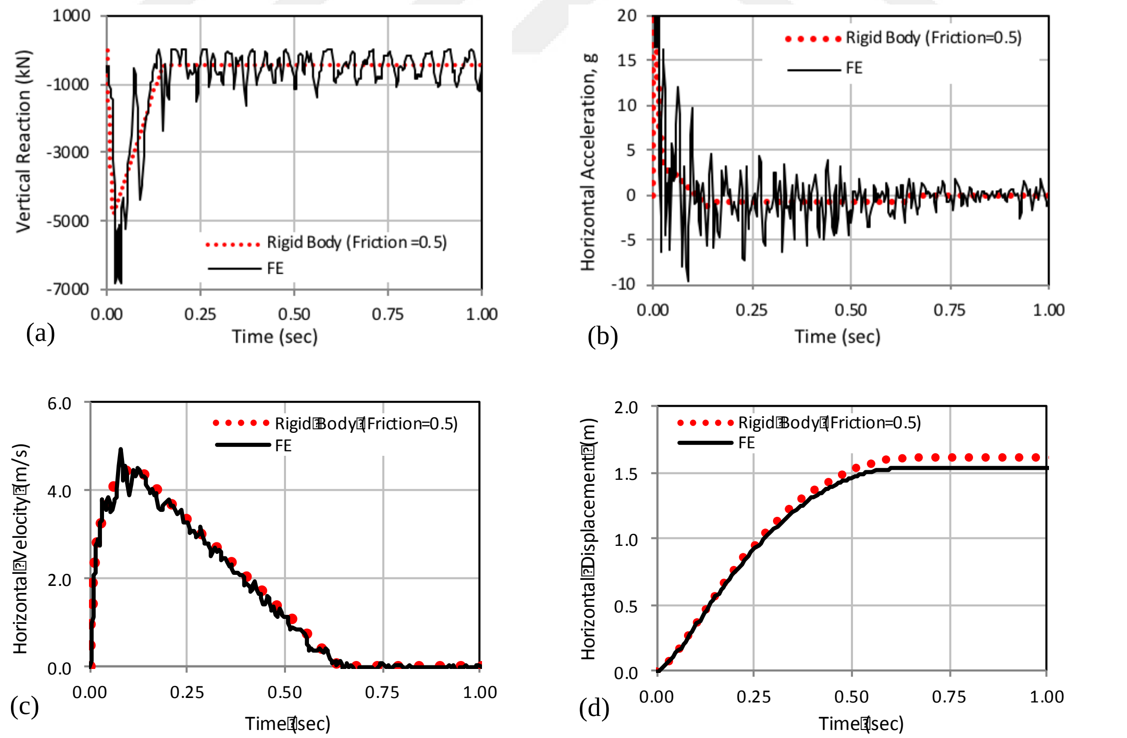


Figure 28: Building Sliding Motion with μ=0.5 (a) Vertical Reactions, (b) Horizontal Acceleration, (c) Horizontal Velocity, and (d) Horizontal Displacement

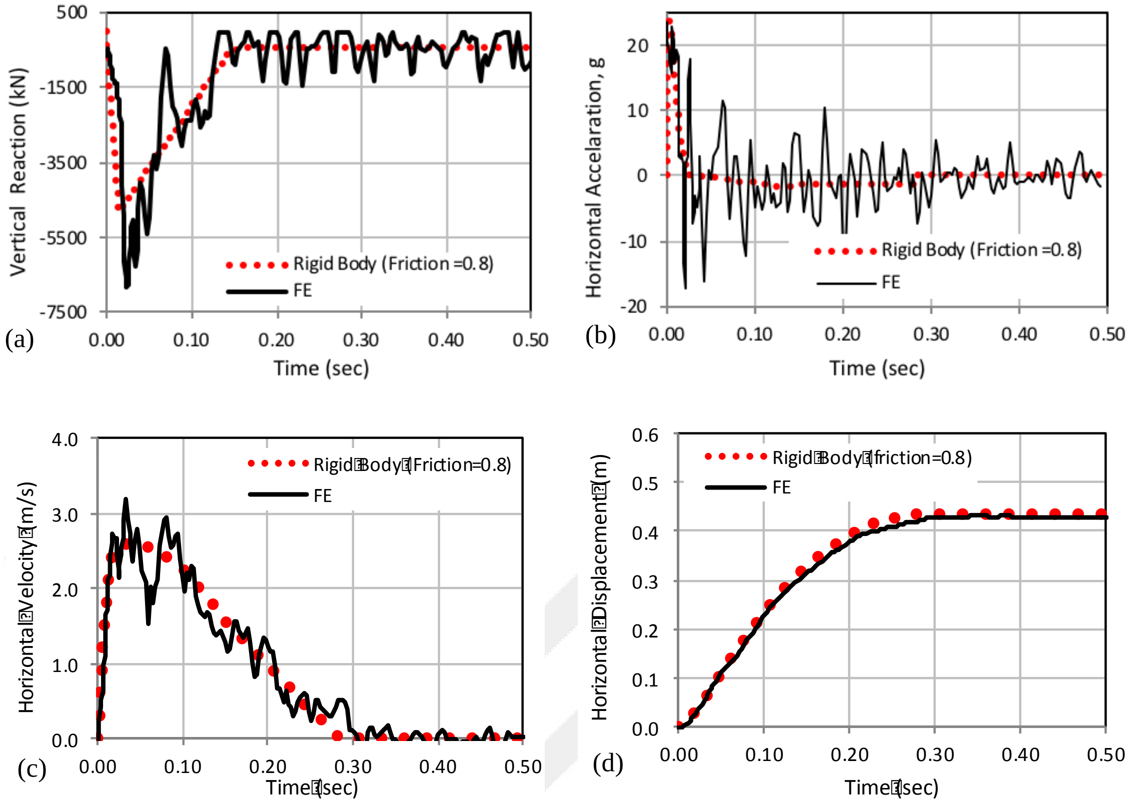


Figure 29: Building Sliding with μ=0.8 (a) Vertical Reactions, (b) Horizontal Acceleration, (c) Horizontal Velocity, and (d) Horizontal Displacement

CHAPTER IV

EFFECTS OF DYNAMIC INTERACTION OF STRUCTURAL MEMBERS ON ESDOF BLAST ANALYSIS

Another blast analysis method that is widely used is uncoupled Equivalent Single Degree of Freedom (USDOF) blast analysis method. This dynamic analysis method is simple, computationally cheap, and provided considerable advantages especially at the preliminary design stage, where cross sections of structural members are selected for a specific dynamic load. This dynamic analysis method is based on the assumption that there is no dynamic interaction (uncoupled) between structurally connected structural members. In other words, the deformation (rotation and displacement) at connections are ignored. Another crucial assumption is that the shape function of structural member deformation is needed. Another assumption is that shear deformations are neglected. Because of these inherent assumptions, the USDOF dynamic analysis method is only an approximate analysis method [2] [3] [5].

The effects of neglecting dynamic interaction of connected structural members and violating joint displacement compatibility has been studied by a limited number of researcher. Biggs (1964) showed that this dynamic interaction of structural members can be conservatively neglected for two degrees of freedom when the ratio of natural vibration frequency of interconnected elements is at least two. Baker et al. [7] showed that USDOF analysis method is an approximate but conservative approach based on work done on two-degrees of freedom systems.

This chapter discusses (1) Biggs' formulation of the USDOF for structural elements subjected to blast loading, (2) a parametric study, where blast performance of a beam-girder system is evaluated using FE and USDOF analysis methods for beam-to-girder period ratio ranging from 4.0 to 0.25 to evaluate accuracy of USDOF analysis method and determine relative errors associated.

4.1 Biggs's Formulation of Uncoupled Single Degree of Freedom System

Structural members are multi degree-of-freedom systems as shown in Figure 30 for a beam under vertical dynamic loads. The beam has distributed mass and different vertical displacement along its span. However, Biggs [5] has shown that deflection history of this multi degree-of-freedom beam system at midspan (i.e., location of maximum deflection) can be accurately predicted with appropriate modification for beam stiffness at the location of deflection and mass used for USDOF system.

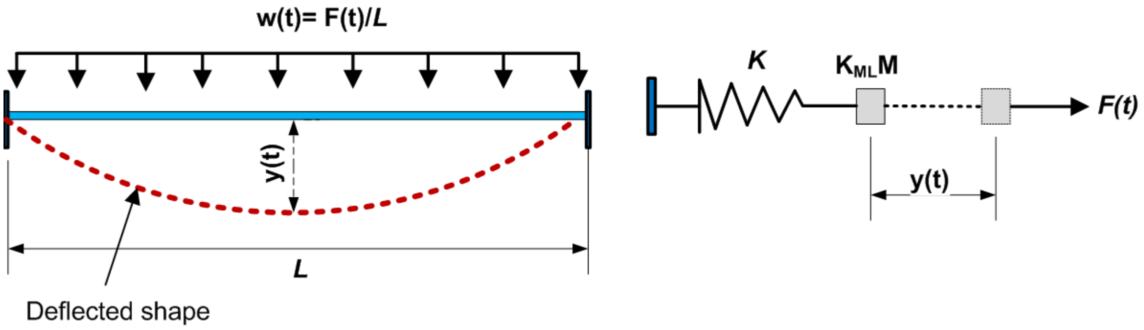


Figure 30: Multi Degrees-of-Freedom System (Beam) and Corresponding USDOF system

Equation of motion for a single degree of freedom system is given as;

$$M\ddot{y}(t) + c\dot{y}(t) + Ky(t) = F(t) \tag{4. 1}$$

Where M is the total mass attached to the spring, c is the viscous damping constant if it is not zero, K is stiffness of the spring, F(t) is the dynamic force, and $\ddot{y}(t)$, $\dot{y}\,(t)$ and $y(t)$ are acceleration, velocity, and displacement of mass, respectively. Typically damping is neglected for blast analysis since maximum deflection occurs before damping effects ae significant for deformation. Replacing these parameters of SDOF system with those of beam, for example using total mass of beam, stiffness of beam at midspan, or total dynamic load applied to the beam for SDOF system will not yield the same deflection history as that computed for the beam at midspan. Because, beam mass is distributed, acceleration and velocity of mass change along the beam, and dynamic load is not applied to a single point rather it is distributed. Biggs corrected the error due to distributed mass and dynamic load being distributed over the beam span by introducing stiffness transformation factor K_L and mass transformation factor K_M. These factors are commonly known as Biggs' factors. The values of these two factors are determined by equating total work and kinetic and strain energies of SDOF system and beam system. The general equations to predict Biggs' factor are given as;

$$K_M = \frac{\int_0^L m\emptyset^2(x)\,dx}{mL} \tag{4. 2}$$

$$K_L = \frac{\int_0^L w\emptyset(x)\,dx}{wL} \tag{4. 3}$$

Where m is mass density per length, L is beam span length, w is a static load having the same distribution as dynamic load along the beam, and ϕ is the assumed-shape function for the beam deflection with a magnitude of unity at the point of maximum deflection. Although the shape function remains constant for elastic systems (in terms of both geometry and material), it changes as the structural elements undergo plastic deformations. Therefore, the value of Biggs' factor depends on strain condition of structural elements and do not remain constant throughout the analysis. However, it should be noted that typically constant values are used in practice, but the selected values reflected the dominant deformation state of the beam (i.e., elastic or plastic). With Biggs' factors, the updated equation of motion for SDOF system to match beam multi degree-of-freedom system is given as for an elastic-perfectly plastic system;

$$(K_M M)\ddot{y}(t) + min[K_L K y(t), K_L R_u] = K_L F(t) \quad (4.\ 4)$$

$$(K_{ML} M)\ddot{y}(t) + min[K y(t), R_u] = F(t) \quad (4.\ 5)$$

Where R_u is the ultimate resistance (load capacity) of the beam or structural member under static load that has the same distribution as the dynamic load. A convenient way to express above equation is done by defining ratio of mass and load transformation factor as K_{ML}, which called mass-load transformation factor.

$$(K_{ML} M)\ddot{y}(t) + min[K y(t), R_u] = F(t) \quad (4.\ 6)$$

Values of Biggs' factor (K_L, K_M, K_{ML}) depends on structural element support conditions, dynamic load and mass distribution along member length, and shape of deformed shape (i.e., elastic, elastic-plastic, and plastic) corresponding to the time at which deflection is computed. A summary of Biggs' factors for different beam boundary conditions are given in Table 4 as well as spring stiffness, ultimate resistance for distributed uniform load condition.

Table 4: Biggs' Factors [5]

Beam with Both Ends Simply Supported						
Loading Diagram	Strain Range	K_L	K_M	Maximum Resistance R_u	Spring Constant K	K_{ML}
uniform	Elastic	0.64	0.5	$8M_p/L$	$384EI/5L^3$	0.781
	Plastic	0.5	0.33	$8M_p/L$	0	0.660
Point at midspan	Elastic	1	0.49	$4M_p/L$	$48EI/L^3$	0.490
	Plastic	1	0.33	$4M_p/L$	0	0.330
Beam with Both Ends Fixed						
Loading Diagram	Strain Range	K_L	K_M	Maximum Resistance R_u	Spring Constant K	K_{ML}
uniform	Elastic	0.53	0.41	$12M_{ps}/L$	$384EI/L^3$	0.773
	E-P	0.64	0.5	$8(M_{ps}+M_{pc})/L$	$384EI/5L^3$	0.781
	Plastic	0.5	0.33	$8(M_{ps}+M_{pc})/L$	0	0.660
Point at midspan	Elastic	1	0.37	$4(M_{ps}+M_{pc})/L$	$192EI/L^3$	0.370
	Plastic	1	0.33	$4(M_{ps}+M_{pc})/L$	0	0.330
Beam with Fixed-Pin Ends						
Loading Diagram	Strain Range	K_L	K_M	Maximum Resistance R_u	Spring Constant K	K_{ML}
uniform	Elastic	0.58	0.45	$8M_{ps}/L$	$185EI/L^3$	0.776
	E-P	0.64	0.5	$4(M_{ps}+2M_{pc})/L$	$384EI/5L^3$	0.781
	Plastic	0.5	0.33	$4(M_{ps}+2M_{pc})/L$	0	0.660
Point at midspan	Elastic	1	0.43	$16M_{ps}/3L$	$107EI/L^3$	0.430
	E-P	1	0.49	$2(M_{ps}+2M_{pc})/L$	$48EI/L^3$	0.490
	Plastic	1	0.33	$2(M_{ps}+2M_{pc})/L$	0	0.330

Note: M_{ps} and M_{pc} are plastic moment capacities at support and midspan, respectively

4.2 Dynamic Interaction of a Girder-Beam System

The girder-beam system used to investigate effects of dynamic interaction of structural members under blast loads is given Figure 31. The length, section properties, and material properties of both girders and beam were assumed to be equal for the reference case. The boundary condition of the supporting girders was assumed to be fixed at all four ends. The connection between the beam and girders was a ‘full connection” transferring all forces and moment.

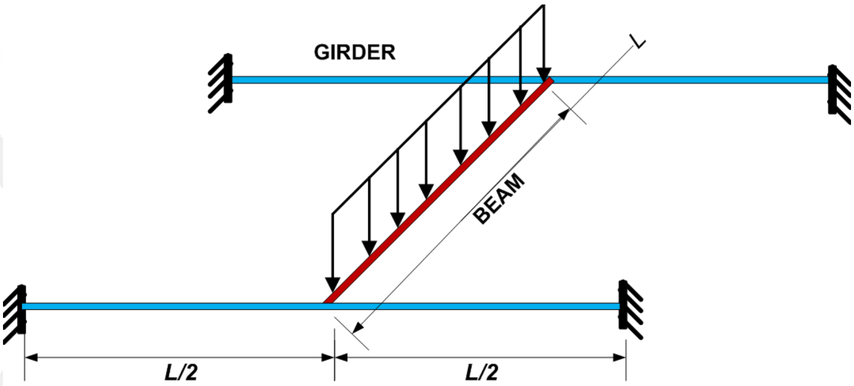


Figure 31: Girder-Beam System

A summary of girder-beam system dimensions, section properties, and material is given in Table 5. For parametric study, both elastic and elastic-perfectly plastic material properties were used. The table also include a summary of Biggs’ factor, computed period of vibration for corresponding USDOF systems. Note Biggs’ factors for the beam are based on a beam uniformly loaded and fixed at both ends while those for the girders are based on a concentrated load at the midspan of a girder with both ends fixed. The computed force versus midspan deflection relationships for both girders and beam are given in Figure 32.

Table 5: Properties of Girder-Beam System

Properties	Beam	Girder
Length, m	3	
Section Width, mm	100	
Section Height, mm	150	
Modulus of Elasticity, GPa	200	
Density, kg/m^3	7850 (*)	
Yield strength, MPa	414 (§)	
K_{LM}, Elastic	0.774	0.37
K_{LM}, Elasto-plastic	0.721	NA
K_{LM}, Plastic	0.66	0.33
T, Elastic (sec)	0.012	0.012
T, Elasto-Plastic (sec)	0.013	NA
T, Plastic (sec)	0.012	0.011

(*) Density of girder was adjusted for other cases
(§) Steel material was assumed to be elastic-perfectly plastic

For blast loading, two cases were considered as shown in Figure 32. Blast-1, which was considered for the elastic case, had a peak value of line load of 1750 kN/m. The blast load was reduced for elastic-perfectly plastic material case to limit maximum deflection to reasonable levels (ductility at the order of 5 to 10). Blast-2 load, which was used for elastic-perfectly plastic material case, had a peak value of line load of 525 kN/m. As mentioned above, the load levels were selected to achieve reasonable ductility levels corresponding to high damage of beam.

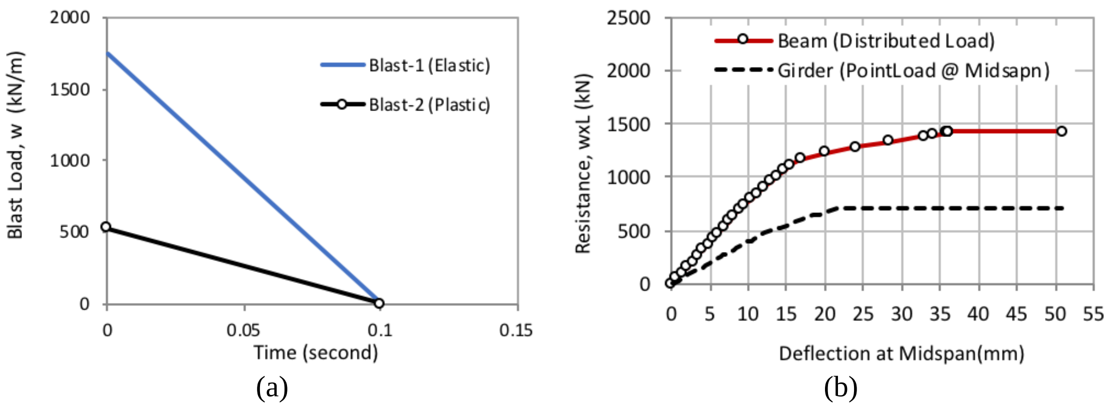


Figure 32: (a) Blast Loads and (b) Load versus Deflection Curves for Girder and Beam

4.2.1 Finite Element Model

Dynamic blast analyses of the girder-beam system were done using ABAQUS finite element software. The developed simple model is given in Figure 33. The girders and beam were modelled using B31 beam element, which is suitable for thick and slender beams. The beam element is a Timoshenko (shear flexible) beam allowing for transverse shear deformations. A nominal mesh size of 15 cm was used for the beam and girders. For materials, both elastic and elastic-perfectly plastic cases were considered. The blast load was applied as a uniform distributed line load on only the beam. No direct blast load was considered for the girder. In other words, end reactions of the beam were the only blast load considered for the girders.

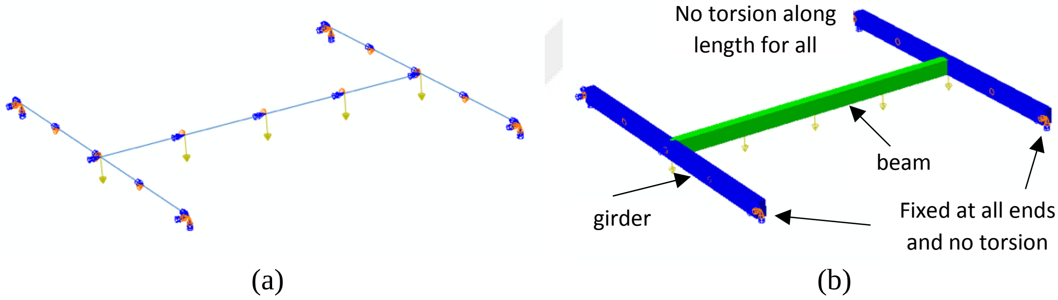


Figure 33: (a) Girder-Beam Model (b) Boundary Conditions

Defining proper boundary condition for the system was necessary. For this purpose, ends of girders were fixed for all degrees of freedom while their torsion was restrained by defining zero deflection in the direction of beam. In other words, the defined boundary conditions ensured that the girders deflect only in vertical direction with no torsion. A similar boundary condition was defined for the beam. The beam ends were connected to the girder (i.e., shared the same node) and beam torsion was also restrained by defining zero displacement boundary condition for the beam nodes in the direction of girder span.

4.2.2 Parametric Study

To determine effects of dynamic interaction between the girders and beam, ten cases were investigated for both elastic and elastic-perfectly plastic cases. A summary of these cases is given in Table 6. In addition to these ten cases, a “no-interaction-case”, which was a modified version of Case1, was also used. For no-interaction-case, the two ends of the beam were fixed for all degrees of freedom to prevent dynamic interaction of beam and girders. This no-interaction case was used as the reference case to determine effects of dynamic interaction. For the cases investigated, the only changing parameter was the single degree of freedom vibration period of the girders while that of the beam was not changed. The girder vibration period was simply changed by changing its material density. For each case, computed periods of corresponding single degrees of freedom systems and mass densities are given in Table 6.

Table 6: Summary of Parametric Cases Investigated

Properties	Case1	Case2	Case3	Case4	Case5	Case6	Case7	Case8	Case9	Case10
Beam Density	7850									
Beam Period, T_2	0.012									
Girder Density	7850	125600	96163	70650	49063	31400	17663	1963	491	123
Girder Period, T_1	0.011	0.044	0.039	0.033	0.028	0.022	0.017	0.006	0.003	0.001
T_1/T_2	1.0	4.0	3.5	3.0	2.5	2.0	1.5	0.5	0.25	0.125

Note: Density unit is kg/m^3

With the cases studied, a large range for girder-to-beam period ratio (i.e., T_1/T_2) was studied. The period ratio ranged from 0.125 to 4.0 as given in Table 6. It should be noted that a more appropriate way to achieve these period ratios would be by changing the girders length and cross-section dimensions. However, such a change will also affect ultimate load capacity (i.e., R_u) of girders and making difficult to distinguish effects of period only on dynamic interaction.

4.3 Results of Dynamic Interaction

Table 7 gives a summary of computed maximum deflections for the beam at its midspan for both elastic and elastic-perfectly plastic materials. Maximum deflections of the beam for the reference cases (uncoupled), where there is no dynamic interaction between the girders and beam are 111 mm and 338 mm for elastic and elastic-plastic materials, respectively. These are the deflections that would have been computed if USDOF analysis method was used for the beam under the given blast loads.

When elastic material was used for both beam and girders, the computed maximum deflections at the midspan of the beam were higher than that of the uncoupled beam for the whole range of girder's period considered. However, the difference in the maximum deflection due to dynamic interaction was only between -1% to 19% of uncoupled beam case. In other words, the maximum deflection of the beam at its midspan computed by ignoring beam-girder interaction was on the unconservative side by as much as 20%. This observation shows that for elastic systems using USDOF is unconservative, however it should be noted that typically structural members are designed to undergo significant plastic deformation under blast loads.

Table 7: Summary of Parametric Cases Investigated

T_1/T_2	Elastic Material		Elastic-Perfectly Plastic Material	
	Max. Deflection (mm)	Relative Deflection (%)	Max. Deflection (mm)	Relative Deflection (%)
4.00	122	110	259	77
3.50	121	109	247	73
3.00	123	111	235	70
2.50	123	111	218	64
2.00	120	108	198	59
1.50	110	99	176	52
1.00	111	100	177	52
0.50	131	118	294	87
0.25	132	119	358	106
0.125	132	119	377	112

(1) Max. deflection at beam midspan for no interaction cases is 111 mm and 338 mm for elastic and elastic-perfectly plastic materials.
(2) Relative deflection = (max. deflection/111 mm)100
(3) Relative deflection = (max. deflection/338 mm)100

With elastic-perfectly plastic material model, which allows members to go under plastic deformation, maximum deflections of beam at its midspan was larger than that of uncoupled beam for T_1/T_2 ratio larger than 0.5. For all other cases, where T_1/T_2 ratio larger and equal than 0.5, the computed maximum deflection of beam midspan was smaller than that of uncoupled beam. This observation clearly shows that USDOF analysis can be used for the beam for cases T_1/T_2 is larger than 0.5. However, typically the girder will be stiffer (smaller period of vibration) than beam, and T_1/T_2 ratio will be smaller than 1.0. Therefore, in a realistic girder-beam system where girder is stiffer than the beam, using USDOF system may yield unconservative results.

Figure 34 shows beam's computed maximum midspan deflections as a function of periods ratios. The results show that dynamic interaction (coupling) is becoming important for elastic girder-beam system when the periods of girder and beam differ by two. This observation contradicts with Biggs' statement that "*it may be said that two* *such elements may be treated separately if the periods differ by a factor of 2 or more*". However, it should be noted that Biggs's statement is based on closed form solution for a girder-beam system similar to the one studied here but with a concentrated load on the beam as blast load, and simply support end condition for all beam and girder ends.

Results for elastic-perfectly plastic material show that coupling is important when girder period is less than half of the beam period. In other words, when T_1/T_2 ratio is less than 0.5, the USDOF analysis of the beam yields unconservative results. Typically, girders will have a smaller period than the beams its supporting. Therefore, using USDOF for beams supported on stiffer girders will yield unconservative deflection results. On the other hand, beam maximum deflection decreased due to coupling when girder period was more than quarter of beam period. In other words, using USDOF for the beam yields conservative deflection results when girder-to-beam period ratio T_1/T_2 is more than 0.25.

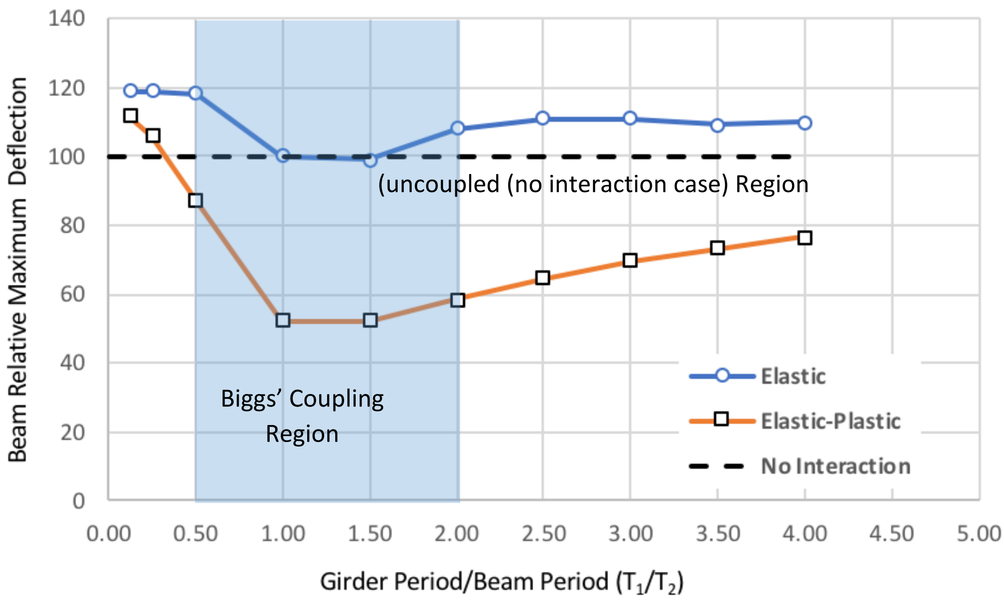


Figure 34: Beam Maximum Relative Deflection and Biggs' Coupling Region

Figure 35 shows beam midspan deflection histories for the girder-beam system for all cases studied. The results show that beam goes under plastic deformation and vibrates at plastic deformations. The deflection histories further verify above discussions.

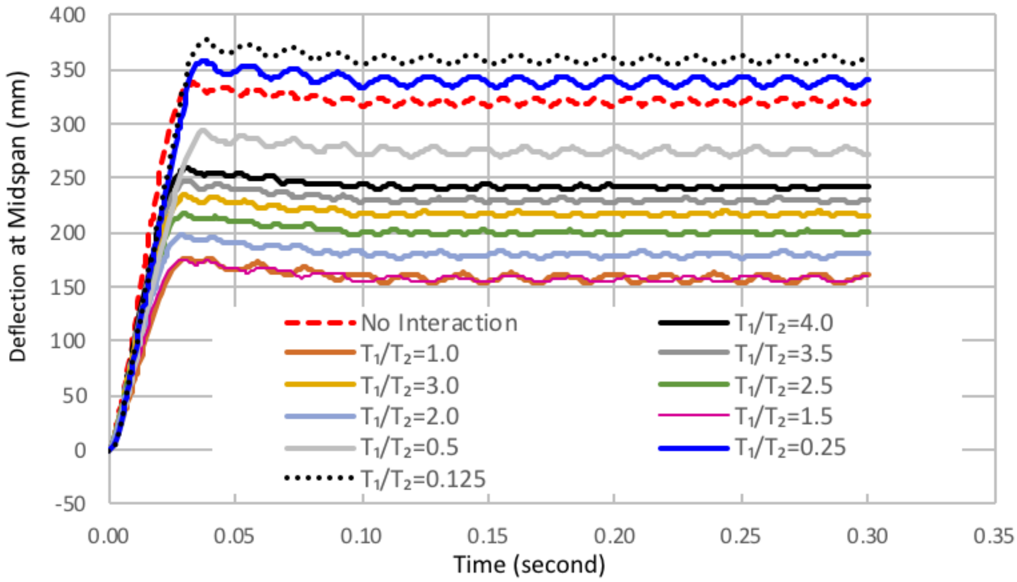


Figure 35: Beam Midspan Deflection Histories for Girder-Beam System

Figure 36 through 40 shows deflection histories of beam midspan for elastic material cases (no yielding, elastic material) relative to the uncoupled case, where there is no dynamic interaction between the beam and supporting girders.

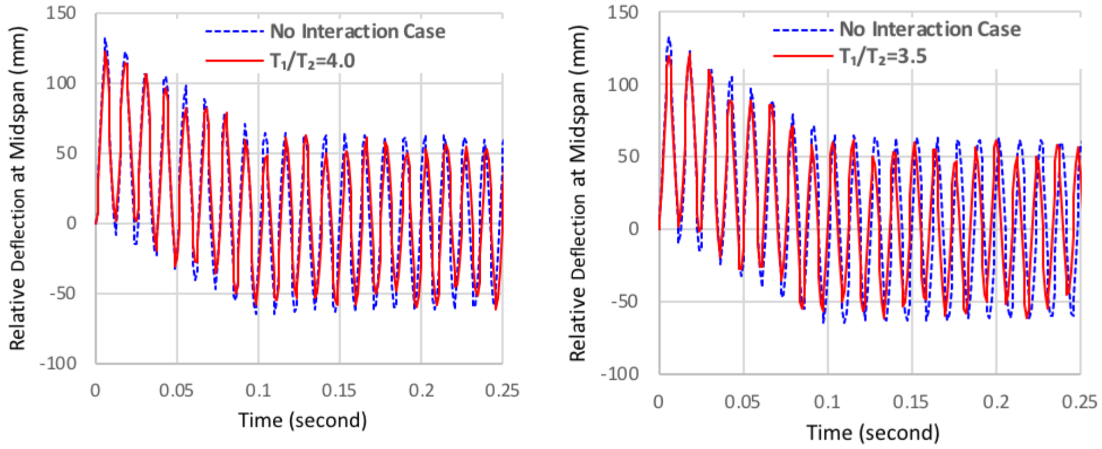


Figure 36: Beam Midspan Deflection Histories for Period Ratios 4.0 and 3.5

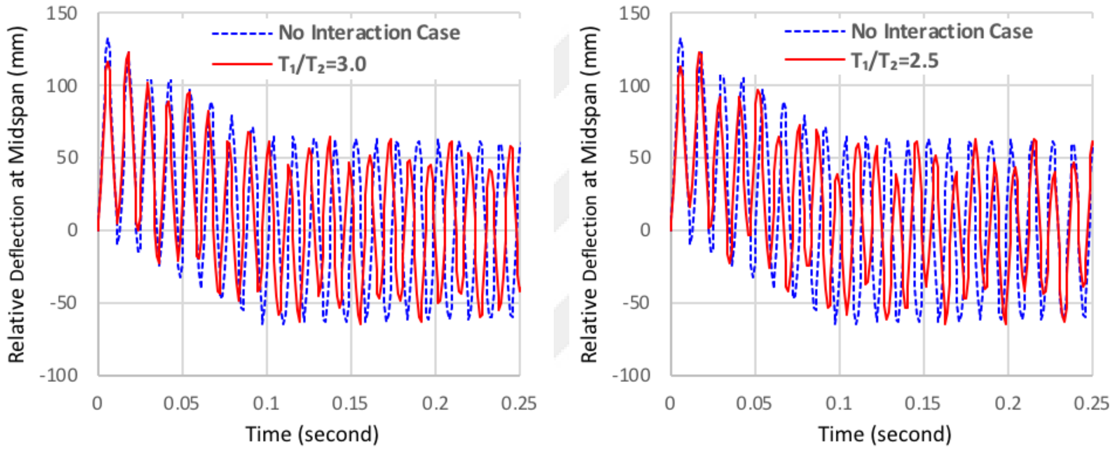


Figure 37: Beam Midspan Deflection Histories for Period Ratios 3.0 and 2.5

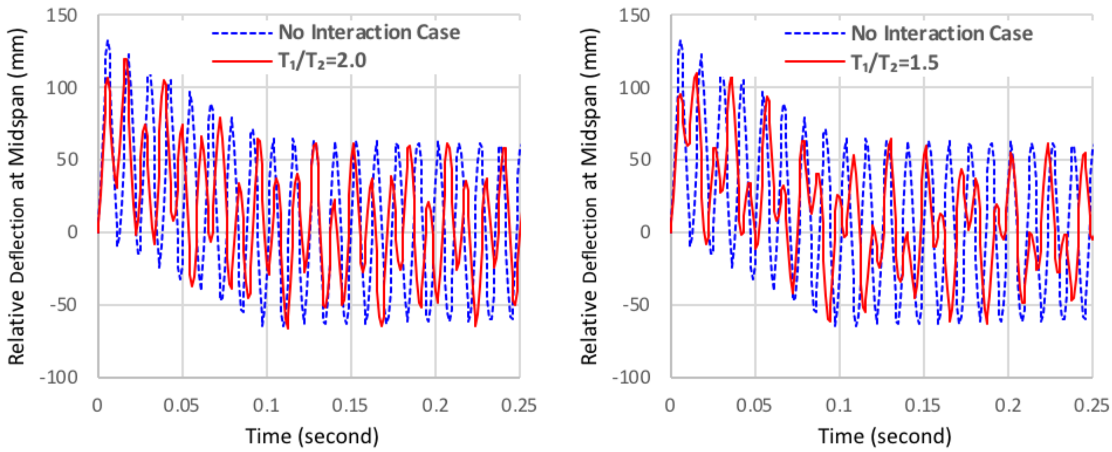


Figure 38: Beam Midspan Deflection Histories for Period Ratios 2.0 and 1.5

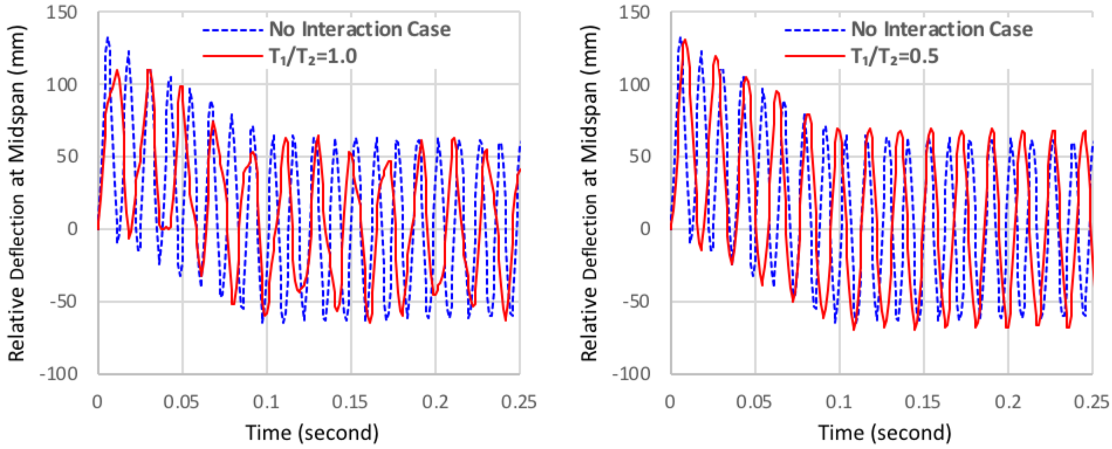


Figure 39: Beam Midspan Deflection Histories for Period Ratios 1.0 and 0.5

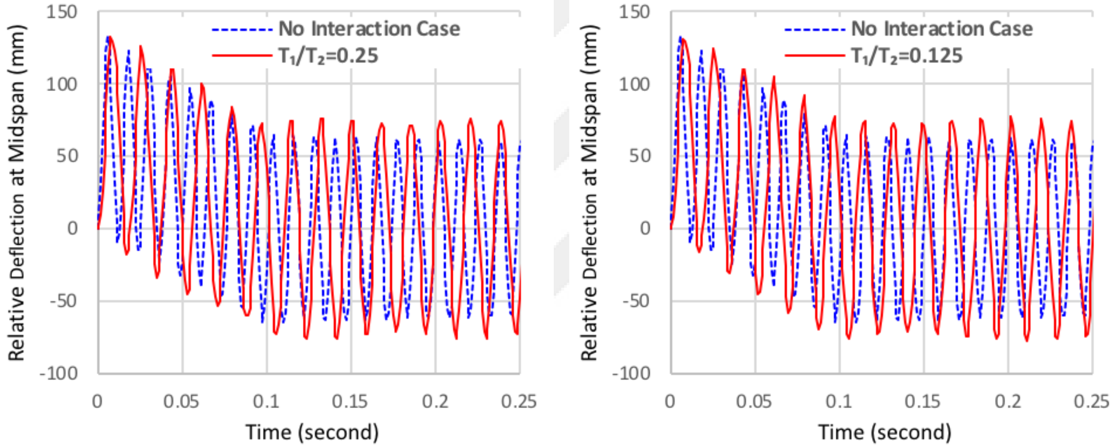


Figure 40: Beam Midspan Deflection Histories for Period Ratios 0.25 and 0.125

CHAPTER V

SUMMARY, CONCLUSIONS AND FUTURE WORK

5.1 Summary

This thesis includes three main blast related subjects studied. These are effects of unanchored foundation on blast performance of blast-resistant modular steel (BRMS) buildings, Rigid body sliding analysis of unanchored structures subjected to blast loading, and Effects of dynamic interaction of structural members on uncoupled single degree of freedom analysis.

In Chapter 2, blast performance of a prototype BRMS building was determined using nonlinear dynamic analysis for several foundation conditions including anchored foundation, where structure was fixed to the foundation and with free-to-slide foundation where the structure was allowed to slide over its foundation. For free-to-slide foundation case, three values (0.2, 0.5, and 0.8) of coefficient of friction between the structure and its foundation were considered. The FE blast analysis results were used to determine effects of sliding on blast performance of individual structural members as well as foundation horizontal and vertical reactions.

In Chapter 3, rigid body analysis of unanchored structures subjected to blast loading was investigated. A numerical integration method, which was verified using finite element analysis, was developed. The applicability of rigid body approximation to computed sliding acceleration, velocity, and distance and foundation reactions were evaluated by comparing rigid body results with those corresponding to the same structure when its deformation under blast loading is included through full finite element analysis of structure foundation and their dynamic interaction.

In Chapter 4, effects of dynamic interaction of inter connected structural elements on results of uncoupled single degree of freedom (USDOF) blast analysis method were investigated. For this purpose, blast performance of a girder-beam system was determined in terms of maximum deflections at the midspan of the beam for values of girder to beam period ratio of 0.125 to 4.0. For the interaction, effects of both elastic and elastic-perfectly plastic materials (deflections) were also investigated to provide guidance for using USDOF analysis for determining blast performance of structural members.

5.2 Conclusions

The following conclusions can be derived from the present study on each subject studied.

Effects of Unanchored Foundation on Blast Performance of BRMS Buildings

- The analysis results show that allowing structure to slide even a limited distance on its foundation improves blast performance of structural members by as much as 25%.

- Allowing structures to slide over their foundation can significantly reduce horizontal foundation reaction while its effect on vertical foundation reaction is negligible. The decrease in horizontal foundation reaction for the porotype building studied under blast load was 40%, 60% and 85% for coefficient of friction 0.8, 0.5 and 0.2, respectively. These results show that providing even limited sliding can significantly decrease foundation horizontal reaction.

Rigid Body Analysis of Unanchored Structures Subjected to Blast Loading

- A simple and cost effective numerical method developed based on rigid body motion can be successfully used to predict horizontal acceleration, velocity, and sliding distance of blast resistant modular structures under blast loading.

- For deformable structures subjected to blast loading, the numerical method under predicted vertical reaction force and horizontal maximum acceleration. The main reason for this is the fact that deformable structure vibrates under blast load, and this vibration behavior is not included in the developed numerical method. However, still predicted horizontal velocity and horizontal sliding displacement were reasonably in good agreement with those predicted using FE analysis.

- Accuracy of developed numerical method decreased for vertical reaction, horizontal acceleration, and horizontal velocity as the coefficient of friction between the foundation and structure increased (i.e., 0.8). However, the predicted sliding displacement were still reasonably accurate.

Effects of Dynamic Interaction of Structural Members on USDOF Blast Analysis:

- For elastic girder-beam system (coupled system), computed maximum midspan deflection of the beam was higher than that predicted with uncoupled single degree of freedom (USDOF) analysis for the whole ratio of girder-to-beam period ratio range of 0.125 to 4.0. This finding shows that using the USDOF analysis approach for computing blast performance of members, which are interconnected to other structural members, yields unconservative deflection (blast evaluation) for elastic systems.

- For elastic-perfectly plastic girder-beam system studied, maximum deflection of the beam at its midspan was larger than that of uncoupled system (i.e., USDOF) when vibration period of the girder was less than half of beam vibration period. For all other cases investigated, maximum deflection of beam on the girder-beam system was only between 50% and 80% of maximum deflection computed using USDOF analysis. In other words, blast performance of the beam determined using USDOF was on the conservative side when girder period was larger than ¼ of the beam supported.

- Findings on dynamic interaction of girder-beam system studied contradict with Biggs' statement that coupling between structural elements can be neglected when their periods differ by a factor of 2 or more. One possible reason for this contradiction might be the difference of beam and girder support conditions for the girder-beam system studied by Biggs being simply supported while the system studied here is fixed.

5.3 Future Work

Design procedures based on the results of uncoupled single degrees of freedom analysis are the ones that are most common used for blast resistant design of structures. However, the effects of coupling due to inter connected elements is still not fully discovered. Therefore, future work on coupling of structural elements for different blast loading, boundary condition, damage level should be investigated.

In addition, the USDOF blast analysis is based on predicted displaced shape (Biggs' shape function) of structural elements. The fact that structural elements are interconnected and deformation of one element will affect other connected may have some effects on the assumed deflected shape or shape function, which can affect results and accuracy of uncoupled single degree of freedom analysis. The effect of structural element interaction of shape function can also be investigated.

Finally, the value of Biggs' factors (K_L and K_M) changes depending on whether deflection is in elastic, elastic-plastic, or plastic region. However, in SDOF blast analysis typically a constant value is assumed for Biggs' factor depending on expected deformation levels. Effects of using a single value for Biggs' factor on SDOF results may also be investigated.

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VITA

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