

**DYNAMIC ANALYSIS OF MULTISTOREYED BUILDING
SUBJECTED TO BLAST LOAD**

PROJECT REPORT



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DECLARATION

I undersigned hereby declare that the project report “Dynamic analysis of multi-storeyed building subjected to blast load”, submitted for partial fulfilment of the requirements for the award of the degree of Master of Technology of the APJ Abdul Kalam Technological University, Kerala, is a bonafide work done by me under supervision of Prof. Shefeena N., Assistant Professor, Department of civil engineering, TKM College of Engineering. This submission represents my ideas in my own words and where ideas or words of others have been included, I have adequately and accurately cited and referenced the original sources. I also declare that I have adhered to ethics of academic honesty and integrity and have not misrepresented or fabricated any data or idea or fact or source in my submission. I understand that any violation of the above will be a cause for disciplinary action by the institute and/or the University and can also evoke penal action from the sources which have thus not been properly cited or from whom proper permission has not been obtained. This report has not previously formed the basis for the award of any degree, diploma or similar title of any other University.

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CERTIFICATE

Certified that this report entitled 'DYNAMIC ANALYSIS OF MULTISTOREYED STRUCTURES SUBJECTED TO BLAST LOAD' is the report of the project presented by **ABDUL BASITH**, Roll No: **TKM20CESC01** during **2021-2022** in partial fulfilment of the requirements for the award of the Degree of Master of Technology in Structural Engineering & Construction Management of the A P J AbdulKalam Technological University.

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ABSTRACT

Terrorists have targeted iconic and public buildings in recent years. Terrorists are developing high-intensity bombs as technology advances. Bomb blasts and threats are a growing problem all over the world. The protection of human life against such attacks includes the prediction, avoidance, and modification of such events. In recent years, the design and analysis of such impulsive loads applied to structures have been thoroughly investigated to determine the performance of structural elements subjected to the sudden type of loading. Because blast loads are highly unpredictable and dynamic in nature, it is extremely unlikely that a fully blast resistant structure can be designed. As a result, it is critical to comprehend the effect of blast on the structure and the behaviour of structural elements as a result of the load. Depending on the location of the blast, the structure suffers a partial or complete collapse of structural members, resulting in the loss of structural integrity.

In this study, static and dynamic analyses are performed on Extended Three-Dimensional Analysis of Building System (ETABS) 2019 to analyse the response of a G+5 storey building subjected to blast effect due to the blasting of 100kg TNT explosive at various locations. The blast parameters, such as peak reflected overpressure and positive phase duration, are calculated using the codes IS 4991-1968 and UFC 3-340-02, and pressure time history analysis is performed. Various structural systems, such as shear walls and steel bracings, are used to make the building more resistant to blast loads. Structural modifications such as increasing column size and changing plan configuration are carried out to determine their impact on the structure's blast response. The two elements taken into consideration when assessing the building's safety are the storey displacement and the storey drift. The building's behaviour under a blast load is expressed in terms of safe standoff distance. For the study, two different blasting locations—namely, blasting at the front face and blasting at the corner side—are taken into consideration. A bare frame model's blast response and safe standoff distance are compared to other models with structural modifications. This study's major goal is to shed light on the idea of blast-resistant buildings and to determine how a structure will react to blast loads using ETABS software, with special emphasis placed on various standoff distances from the blast.

Keywords: Blast loading, dynamic analysis, storey displacement, storey drift, bracing, safe standoff distance.

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NOTATIONS AND ABBREVIATIONS

a_0	:	Speed of sound in air
b	:	Decay coefficient of the waveform
E	:	Energy released
H	:	Total height of the building
P_0	:	Ambient atmospheric pressure
P_r	:	Reflected pressure
P_{so}	:	peak overpressure
q_s	:	Maximum dynamic pressure.
R	:	Distance from the detonation source to the point of interest
RC	:	Reinforced Concrete
SSD	:	Safe standoff distance
t	:	Time elapsed
t_A	:	Time of arrival of shock front
TNT	:	Trinitrotoluene
t_o	:	positive phase duration
t_s	:	Positive phase duration
U_s	:	Blast wave front velocity
W	:	Weight of the explosive (charge, TNT)
Z	:	Scaled distance
ρ_0	:	Density of air ahead of blast wave
ρ_s	:	Air density behind wave front

CHAPTER 1

INTRODUCTION

1.1 GENERAL

Terrorist attacks on public buildings are a big contributor to the global problem because they not only result in fatalities but also severely harm a nation's economy. In recent years, issues with spontaneous loading, such as blast loads, impact loads, and earthquake loads, have received the most attention. Recent terrorist attacks, like those on the Jewish Community Center in Argentina in 1994, the Manchester Arena in 2017, the Khobar Towers in Saudi Arabia in 1996, the World Trade Center - USA 9/11 in 2001, etc., shows that explosions can cause significant harm to people and property, depending on the type of charge utilised. Terrorists are also employing new chemicals and technologies to improve the impact or effect of charge detonation on the affected area. Although it is impossible to pinpoint the exact risks brought on by an explosion, loads can be computed to see how they will affect a structure and different safety precautions can be considered. The explosion of explosives within and surrounding structures can have a significant impact on the structural elements, as well as the interior and exterior of the building.

The behaviour of structural elements like beams, columns, and slabs, which are frequently subjected to this type of pressure from various accidental or planned incidents, particularly blast loading, has been the focus of intensive research in recent years. It is crucial to understand how a building responds to blast loads. It is also required to lessen the impact of blast loads on structures, which not only increases their safety but also lowers the likelihood of fatalities and other severe consequences from structural collapse. Due to the increased demand for improved structural safety brought on by terrorist attacks, it is more important than ever to understand how structural components like beams, columns, and slabs react under blast load.

There are many distinct sorts of explosions, including gas chemical explosions, accidental explosions, aerial blasts, surface blasts, etc. The building may even collapse or fail under this kind of attack. Therefore, it is crucial to implement safety measures and assess the damage to structures that have been exposed to blast loads. The interdependence of various components makes it difficult to obtain data on the blast

effect. Finding the impact of the blast on the structure and its behaviour thus becomes quite difficult.

The U.S. Department of the Army has released several technical documents for blast-resistant design. The most widely used design guide is UFC 3-340-02. IS 4991 - 1968: Criteria for blast Resistant design of structures for explosions above ground is the Indian manual. Due to the large cost, it is almost economically unfeasible to obtain the experimental data for multistorey reinforced concrete (RC) structures. As a result, numerical methods like FEM have taken over as the primary tool for blast analysis. Therefore, thorough knowledge of blast parameters and dynamic reactions, such as displacements of different structural elements, is necessary for the analysis of RC structures subjected to blast loads.

1.2 BLAST PHENOMENON

Energy and hot gases are released into the atmosphere during blasting in a matter of milliseconds. This explosion was brought on by chemicals (TNT) combined at a high temperature. The hot gases produced during a blast fill the immediate area. Blast load is a uniformly distributed load which decreases with the increase of standoff distance. Blasts can be exemplified as physical blast, chemical blast, and a nuclear blast. Above the ground surface blast are mainly three different types air blast, surface blast and high-altitude blast. Shock fronts are created by the rapid release of energy brought on by pressure waves in the immediate environment. Hot gas accumulations occur as a result of the blast. The absolute highest pressure above atmospheric pressure that occurs during a shock wave is referred to as the maximum or peak value of overpressure.

1.2.1 BLAST LOAD AND EFFECT ON BUILDING

Energy and hot gases are released rapidly into the atmosphere during blasting. During the blast the hot gases that are generated occupy the space surroundings. The hot gases produced during a blast fill the immediate area. Figure 1.1 shows the vehicle weapon blast with stand-off distance and blast pressure acting on the building.

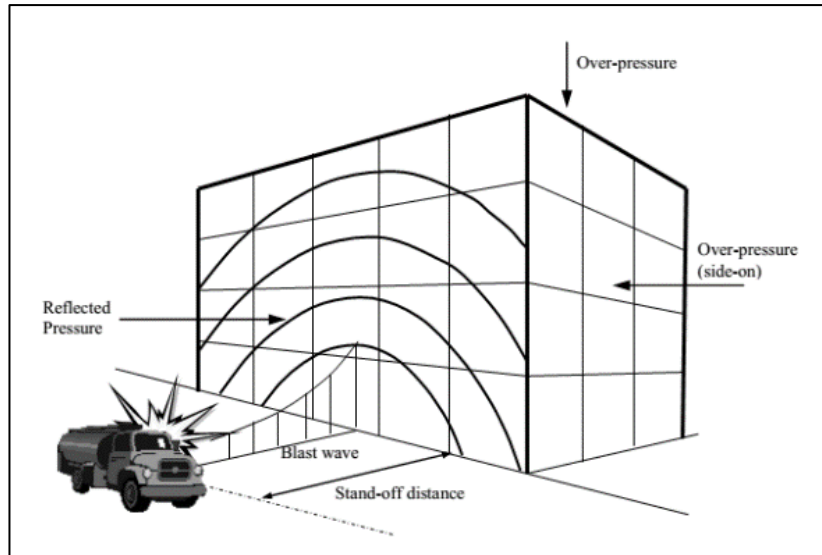


Figure 1.1 Blast load effect on building Charan and Deveraju (2018)

It is not possible to claim that the blast will only occur in one direction. Therefore, safety measures need to be taken in all directions. The beams and columns in the basement area are those that are most severely damaged by a blast. They must therefore be strengthened. Adding barrier compounds all around the structure is a feasible method of protecting it.

1.2.2. BLAST LOAD PROFILE

An explosion is characterised as an extremely quick chemical reaction involving solid, dust, or gas that results in a sudden release of hot gases and energy. The phenomena produce incredibly high temperatures and pressures even though it only lasts for a few milliseconds.

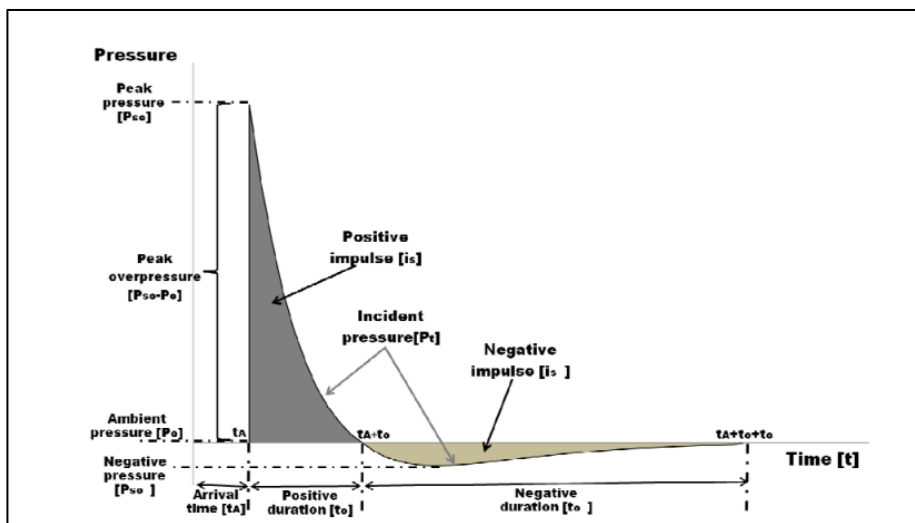


Figure 1.2 Ideal blast wave's pressure time history Charan and Deveraju (2018)

Figure 1.2 depicts the idealised pressure versus time profile for a free air blast wave that travels to a location a specific distance from the detonation. When the shock front arrives at the element at time t_A , the pressure surrounding it immediately increases to a peak pressure P_{so} from its original value of ambient pressure P_o . Since it takes a very brief amount of time for the pressure to reach its maximum value, zero is assumed for design purposes. The peak pressure P_{so} is also referred to as side-on overpressure or peak overpressure. With increasing distance from the detonation center, the value of the peak overpressure and the velocity of propagation of the shock wave decrease. The pressure declines exponentially after reaching its peak value until it reaches the ambient pressure at t_A+t_o , t_o being known as the positive phase duration. Following the positive phase of the pressure-time diagram, the pressure decreases (is referred to as negative) and eventually returns to the ambient value.

The minimum pressure value of the negative phase is $P_{so} -$ and its duration is t_o . It lasts longer than the positive phase. Since it has been established that the main structural damage is related to the positive phase of the explosive wave, the negative phase is typically ignored during design. Furthermore, since the pressure created by the blast wave's negative phase is smaller than its positive phase and moves in the opposite direction, it is generally reasonable to presume that they will not significantly affect the structural integrity of buildings exposed to blast loads.

The energy that a reflected shock front of an explosion imparts to building energy that both the positive and negative pressure-time history phases contribute to determines how much damage is done to the structure. The location of the detonation about the building determines how the pressure and, consequently, forces on the exposed surface of the building change over time and space. The point of explosion, that has the worst effects on the building should therefore be found when examining how a structure responds to a particular blast. Patel et al. (2020).

1.2.3 BLAST WAVE SCALING LAWS

The distance from the explosion and the quantity of energy produced by a detonation in the form of a blast wave are the two factors that affect all blast characteristics the most. Scaling distance about $(E/P_o)^{1/3}$ and scaling pressure in is the energy relation to P_o , where E released (kJ) and P_o is the ambient pressure (usually 100 kN/m^2), can provide a universal standardised description of the blast effects.

However, it is standard practice to describe the fundamental explosive input or charge weight W as an equivalent mass of TNT for ease. After that, results are presented about the dimensional distance parameter,

$$\text{Scaled Distance } (Z) = R/W^{1/3}$$

Where R is the actual effective distance from the explosion also called standoff distance shown in figure 1.3, W is generally expressed in kilograms. Scaling laws offer parametric relationships between a specific explosion and an average charge of the same substance.

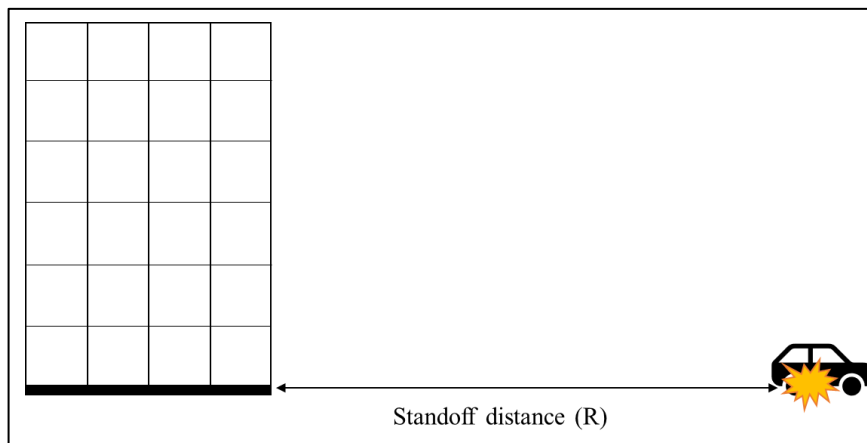


Figure 1.3 Standoff distance

1.3 SIGNIFICANCE OF THE STUDY

In the current situation, the necessity for effective blast-resistant buildings is crucial. The impact of the blast load on the structure as a result of the rise in terrorist activity is a severe problem that leads to the collapse of structures and the loss of lives. Depending on where the explosion occurs within or near other buildings the structure suffers catastrophic failure as a result of the explosion. It is crucial to comprehend how blasts affect structures and to priorities taking the impact of blast load into account when designing a structure because the overall reaction of the structure to the blast load will depend on several criteria. The goal of this study was to perform a nonlinear time history analysis using ETABS 2019, and the codes IS 4991-1968 and UFC 3-340-02, and to determine the degree to which a building may be made safe against blast loads.

1.4 OBJECTIVES OF THE WORK

The objectives of the work are as shown below:

- To determine the storey drift and storey displacement of a G+5 storied reinforced concrete building subjected to surface blasting of 100kg TNT explosive using time history analysis in ETABS software.
- To determine the safe standoff distance of the structure with and without bracing subjected to blast loading.
- To determine the safe standoff distance of the building with different column sizes subjected to blast loading.
- To determine the safe standoff distance of the structure with and without shear wall subjected to blast loading.
- To determine the safe standoff distance of the building with different plan configurations subjected to blast loading.
- To identify the effect of the location of blast on safe standoff distance for all the modelled buildings

1.5 SCOPE OF WORK

The current study has a limited scope and focuses on structure explosion interaction. It takes some model study to ascertain the precise behaviour or performance of the building. Identifying which building is safe and which is not is tough. The study only uses the ETABS programme to conduct an analytical analysis of a typical six-story building. The scope of the project includes

- Only the surface blast condition without taking into account the effect of ground motion is considered.
- The study only focuses on some main parameters, such as story displacement and story drift.
- To know the better performance of any building, the development of a prototype model is required.
- IS code provides the data only for one-tonne explosive weight, so it is very difficult to find the blast parameters for anything above one-tonne explosive weight.

- Here a reinforced concrete structure is analysed, but the vulnerability of steel buildings may also need to be evaluated under the prescribed conditions of blast loading.
- The analytical study of the building is carried out in the ETABS 2019 software.
- A G+5-storey reinforced concrete building with 4 bays in both X and Y directions is considered for the analysis.
- The study is based only on the explosion of 100kg of TNT explosive at various standoff distances.
- The building is analysed only using pressure time history analysis.

1.6 METHODOLOGY IN BRIEF

For satisfying the objectives stated above, the step by step methodology was formulated as follows. The first step in the methodology was to conduct an extensive literature survey and collect the data required for conducting this particular study. It included the data regarding material properties of steel and concrete under consideration, specifications of the building, type of explosion and explosive used. The next step is a validation of ETABS software on the basis of previously conducted research. For validating the software ETABS a numerical analysis of a multi-story building is conducted according to existing literature. After satisfactory completion of validation, the load data of the blasting due to the explosion of 100kg TNT at various locations are calculated by using empirical formulae and standard codes. The next step is modelling; five different building models are made by using ETABS software including the bare frame model. Then the bare frame model is analysed statically and dynamically for finding the range of the safe standoff distance and the exact safe standoff distance respectively. Then all the models are analysed dynamically by taking the location of a blast as front face for finding the safe standoff distance and thereby understanding the extent up to which it can be reduced with respect to the bare frame model. All the models are then dynamically analysed by changing the location of blast from front face to the corner side. The final step is detailed discussion and comparison of the results obtained to find out the dynamic response and influence of blast load on the structure and understanding how the blast phenomenon is related to the load resisting capacity of the structure.

1.7 ORGANIZATION OF THE REPORT

The thesis is structured into six main chapters. Chapter one describes the background of the study, problem statement, significance of the research, objectives of the study, a brief description of the adopted methodology and the scope and limitations of the study.

Chapter two reviews the studies conducted based on blast load on structures. It focuses on the literatures about static and dynamic analysis of multi-storeyed structures subjected to blast load due to the explosion of various kinds of explosives at various locations.

Detailed reporting of the methodology adopted for the study is done in chapter three. A description on different models made in ETABS and the parameters used for the study is discussed in this chapter.

Chapter four describes software validation of ETABS. It details about the comparison of results obtained through reference journal and the results obtained through the dynamic analysis of the model.

Chapter five presents the results of static and dynamic analysis and the inferences obtained from them. It mainly includes the discussion about the response of a structure subjected to blast load in terms of storey displacement and storey drift. It also includes the study about the influence of the location of blast i.e., the change in the response of the structure when the location of the blast is changed from the front face to the corner side. The chapter also discusses the comparison of results of all the cases.

Chapter six reports the major findings and conclusions derived from the study.

CHAPTER 2

LITERATURE REVIEW

2.1 GENERAL

Jayatilake et al. (2004) investigated the three-dimensional nonlinear dynamic responses of typical tall buildings under blast loading with and without setbacks. The impact of the setbacks on the lateral load response caused by blasts was examined about critical site peak deflections, accelerations, inter-storey drift, and bending moments (including hinge formation). Buildings with setbacks shield the tower portion above the setback level from blast loading, according to comparisons. When compared to buildings without setbacks, the building with setbacks exhibits a noticeably better reaction in terms of peak displacement and inter-storey drift. Comparatively speaking, frames with shear walls closer to the explosion experience less damage in terms of the number of hinges created than equivalent frames with shear walls farther from the explosion. When subjected to a blast equivalent to 500 kg of TNT at a standoff distance of 10 m, twenty storey tall structures with shear walls and frames that are built for simply normal loads perform reasonably well, without catastrophic collapse.

Luccioni et al. (2004) explained the impact of blast loads on reinforced concrete buildings' structural failure was investigated. According to research, every step of the process, from the explosive's explosion to the building's total destruction, including the explosion sign's spread and its connection with the structure, was rebuilt. With the aid of a hydro code, the analysis was permitted. The issue being studied corresponds to an actual structure that has been attacked by terrorists. The destruction of the minor columns created a gravitational system that caused the disintegration. The destruction of the face mass of the structure under examination may have been produced with a smaller accusation, but the blast accusation was firmly based on further data in this investigation.

Nguyen and Tran (2011) described the dynamic behaviour of vertical wall structures subjected to blast loads. It has practical applications in the construction of protective buildings in both the civil and defense sectors. Blast loading is simulated by the term of dynamic response in time based on specific assumptions to ensure the physical character of dynamic problems.

Fu (2013) investigated the durability and dynamic responsiveness of tall buildings when subjected to blast stress. This work proposes a 3-D numerical model with direct blast load simulation to examine the actual behaviour of a 20-storey building under blast loading. On the 12th floor, a typical package bomb charge weighing 15 kg exploded, and the related dynamic response of the structure was investigated. It is discovered that the column force under the direct blast simulation approach is less than that of the alternate path method because of the uplift and downward pressure acting on the slab. The shear capacity and ductility of the column need to be improved in top of the building under blast pressure from gradually collapsing. As long as an alternate path can be provided for the load to be dispersed, a small size detonation like a package bomb hardly ever causes the collapse of the entire building for buildings designed utilising the current design guidelines.

Kulkarni (2014) explained the dynamic response of a High-Rise Structure subjected to blast load. In this study, it was discovered that a regular infill frame is an ideal model since it exhibits the least amount of story drift and has excellent lateral stability under blast loads.

Cai et al. (2014) conducted research on building structures' inter-story drifts. One of the extremely helpful engineering reaction numbers and structural performance indicators, particularly for high-rise buildings, is inter-story drift. However, a lot of scientists and engineers fail to distinguish between harmless and hazardous inter-story drift. This essay defines inter-story drift and its components, as well as the difference between detrimental and benign inter-story drift.

Ekström (2015) conducted two studies on Concrete Structures Subjected to Blast Loading. In the first investigation, the reaction of a concrete wall to a shock wave explosion that caused the spalling failure was examined. The importance of this issue stems from the possibility that splintered pieces within protective structures could seriously harm the people or property they are meant to safeguard. His research demonstrates that spalling actually happens when the inelastic strains in the concrete gradually grow as a result of the cyclic response to a blast wave using a straightforward uni-axial computational model. The study further demonstrates that the position and size of the ensuing spalling crack are significantly influenced by the cyclic response in the material model employed for numerical simulation. The second study looked into

how reinforced concrete structures responded to blast loads. The numerical response of a one-way supported slab and a simply supported reinforced concrete beam is assessed using numerical models. In contrast to whether or not the strain-rate dependency of the material properties was taken into account, he discovered that the fracture energy during tensile fracture and how this value is chosen have a greater impact on the deformations of the structure. Additionally, it is established that mesh size and modelling approaches may have a significant influence on the structure's response to the numerical analysis.

Vinothini and Elavenil (2016) illustrate, using SAP2000, the dynamic response of a high-rise building subjected to blast loading. With three distinct standoff distances of 10 meters, 12.5 meters, and 15 meters, respectively, a building with ten stories is exposed to 30 kg and 60 kg of TNT. For examining a structure's dynamic response, a non-linear three-dimensional model is employed. In the current situation, as described in the section of TM-5 1300, structures under blast loading (i.e., bomb explosion) are acting in short duration with high pressure intensity of shock wave. It has been discovered that the best way to minimise the structural damage caused by an explosion is to reduce the facades of the nearby buildings.

Madonna et al. (2016) described the analysis of high rise RCC buildings subjected to blast load. According to the results, a decrease in standoff distance and a rise in charge weight have a substantial impact on the system. Standoff length is the primary factor that affects blast pressure while safeguarding a building.

Bhatt et al. (2016) used ETABS to conduct a comparative performance study of a G+3 storey structure subjected to blast and earthquake loads. The blast pressure is calculated for a four-story building utilizing a variety of input parameters, including the explosive charge, stand-off distance, and building architecture. Different factors, such as the maximum storey displacement, storey drift, and amount of materials, are compared under blast and earthquake loading. With the sections of structural elements identical to the requirements for earthquake resistance, safe charge explosive and safe stand-off distance is obtained for the RCC construction. When compared to earthquake loading, displacement is larger for blast loading and particularly high for the storey at which blast load is applied. The storey drift is found to be within the acceptable limit set forth by the code for the seismic load. In contrast, when there is a blast load, the storey drift goes over the allowed limit, particularly on the storey where the blast is applied. For

blast-resistant buildings versus earthquake-resistant buildings, the amount of concrete is 40 percentage higher.

Syed et al. (2017) investigated the structural behaviour and effectiveness of blast-loaded, earthquake-resistant reinforced concrete (RCC) frame structures. To assess the structural reaction, the vulnerability of the structures was examined under several realistic blast scenarios that were created by adjusting scaled distances and explosive charge weight. The parametric analysis results reveal that standoff distances of 6 to 12 m are necessary to avoid column failure due to a typical explosion on an earthquake-resistant structure. Additionally, it was discovered that structural components of an earthquake-resistant building were better able to withstand blasts. In comparison to an earthquake-resistant construction, a non-earthquake-resistant structure required a larger standoff distance to withstand the same blast load. It was also shown that if the blast load is acting on the longer span, the structure is slightly more susceptible than the case if the blast load is acting on the shorter span. Using high strength concrete improves the shear and moment capacities of the columns and beams, but they have very little impact on the minimum safe standoff distances. Also, if the number of stories is increased, the minimum safe standoff distance also increases slightly because of the additional gravity loads that are acting on the critical column.

Rajeeva (2017) conducted study on the dynamic response of a multi-story building subjected to blast load. The focus is on analysing the dynamic behaviour of a SAP2000-modeled structure. A six-story building is subjected to various TNT stand-off distances totaling 500kg. The blast loads are taken into account utilising the methods outlined in Section 5 of TM5-1300 (UFC 3-340-02), and a nonlinear modal analysis is utilised to analyse the dynamic load of the blast. The two main factors that affect the structure's performance are the total drift and the inter-story drift. It has been found that as standoff distances increase, reflected positive pressure decreases.

Deshmukh et al. (2017) conducted study nonlinear dynamic analysis of high rise building subjected to air and ground blast. They found that the displacement variance is not uniform throughout the height of the building and is distinct from earthquake and wind (Building does not behave as cantilever structure under blast load). Non-linear dynamic response decreases with increasing standoff distance. Building performance reaches the point of collapse with the shortest standoff distance. When a building's

middle height is struck by a bomb, the response is dramatically increased. Building performance is crucial if a blast occurs in the top half of the structure.

Charan and Deveraju (2018) study of effect of blast load on multi-storey building by using time history method. They discovered that when the point of detonation is far from the building, the pressure is low; but, at a distance of 30 meters, the pressure is high. As the standoff distance grows, the pressure falls down exponentially. The detonation point's relationship to pressure is inverse. When the standoff distance is 50 meters, the pressure beyond 30 meters is lowered to 54%.

Liu et al. (2018) conducted research on the behaviour of reinforced concrete columns and beams under blast loading. The experiment investigates the effects of scaled distance and charge mass on RC beams at various scaled distances and charge masses. According to the experiment's findings, RC beams only experience flexural deformation. Damage mode shifts from a few surface cracks to spallation on the back surface with a reduction in scaled distance. Additionally, there was crushing on the upper surface. In addition, RC beams suffer more when charge mass increases at the same scaled distance. Through dimensional analysis, it was also suggested a computational formula for estimating the mid-span maximum deformation that took scaled distance and charge mass into account. The formula is capable of accurately predicting the mid-span maximum displacement of RC beams subjected to blast loading, as evidenced by the reasonable agreement between the analytical and experimental data. The RC beams employed in this experiment are the only ones to which this empirical formula is applicable. A nonlinear single degree of freedom model was created to combine material constitutive models, strain rate effects, progressive impacts of formation, and development of plastic behaviour.

Guha and Mukherjee (2020) carried out research on the subject of RC Structure behavior under Blast Loading. He conducted the study in such a way that he first created an RCC-framed model and its comprehensive plan, after which the verified model was used, examined, and tested to see how well buildings resisted explosions. Additionally, he performed a zone V seismic load analysis on the RCC Structure. Time history analysis is used to determine the blast loading, and the values for force and time are derived from the code UFC 3-340-02. He discovered that the magnitude of the blast pressure dramatically decreases as the stand-off distance from the building grows.

Prashanthi and Elavenil (2021) conducted study on Analytical Investigation on non-linear dynamic analysis of reinforced concrete building subjected to blast loading. They discovered that the incident over pressure is observed with maximal pressure at the bottom and relatively low pressure at the top. In terms of standoff distance, the pressure was stronger at a distance of 10 meters from the construction site and was more prevalent at a range of 12.5 to 15 meters.

Mahaladkar (2019) did a study on the effects of blast loads at standoff distances of 20, 40, and 60 meters on a G+5 storeyed building. They discovered that, when compared to bare frame buildings, the implementation of shear walls at the corner periphery of the building reduced storey displacement and drift by 53.51 percent and 30.04 percent, respectively. In comparison to bare frame buildings, the implementation of X steel bracings at the corner perimeter of the building reduces storey displacement and drift by 23.6 percent and 13 percent, respectively.

Tolani et al. (2020) conducted study on effect of surface blast on multistory building. The SAP 2000 software cannot detect a local failure in members over short standoff distances ($R = 5$ m). Additionally, for blast loads, strain rates are quite high, increasing the concrete's compressive strength and the reinforcement bars' tensile strength.

2.2 SUMMARY OF THE LITERATURE REVIEW

According to the research review, blast load significantly affects structural stability. Therefore, when constructing a structure, it is imperative to take the blast load and its parameters into account. The storey displacement and storey drift are greatly reduced when additional structural supports are introduced to the structure. The research also notes that the safe standoff distance depends on the type and amount of blast load utilised for blasting. From the literature review, it was found that there are many detailed static analysis on the structure subjected to blast load. The of structure due to blast load. In most of the studies story displacement and the storey drift are the main two parameters taken into account for the analysis.

CHAPTER 3

METHODOLOGY

3.1 INTRODUCTION

Blasting causes release of huge quantity of energy within a short duration. Thus, we cannot exactly predict the response of the structure which is subjected to blasting. With the use of some standard codes and analysis software the behaviour of a building under blast load can be approximately found before the design stage itself. In this study, a multistoried reinforced concrete structure subjected to blast load is analyzed by pressure time history analysis using ETABS 2019. Detailed analysis procedure of the study is given as follows.

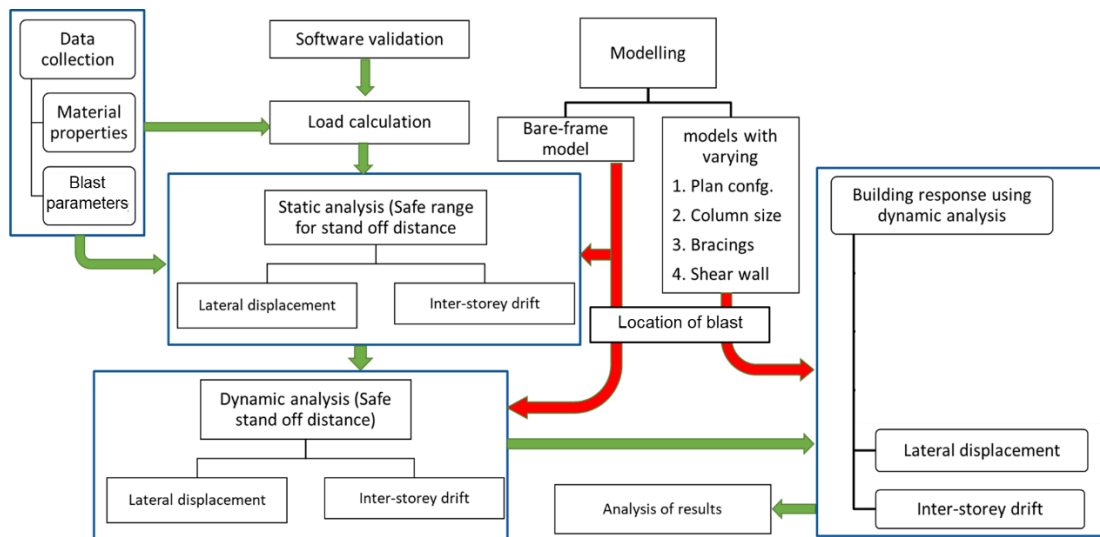


Figure 3.1 Schematic representation of methodology

3.2 DATA COLLECTION

The first step of the project work is collection of data. The parameters required for the study including material properties, specifications of the building and the type of explosion and explosive are fixed. A reinforced concrete building model of plan dimension 20mx20m is considered for the study. The properties of the material used are listed in table 3.1. The general loading of the building is computed as per IS 875 part I & II-1987. A dead load of 4 kN/m² is applied as floor loads as per IS 875-part I. A live load of 3 kN/m² is applied on the general floor area and 1.5 kN/m² on the roof as per IS 875 part II.

The load due to blasting depends on both type and charge weight of the explosive used. TNT is one of the most widely used military high explosives, partly because of its insensitivity to shock and friction. IS:4991-1968 and UFC 3-340-02 are the two codes which consider TNT as standard explosive. So, this study deals with the various kind of response of the building exposed to the surface blasting of a 100kg TNT explosive.

Table 3.1 Properties of material considered

Grade of concrete	M30
Grade of rebar	Fe415 & Fe250
Density of concrete	25kN/m ³
Density of steel	78.5kN/m ³
Poisson's Ratio	0.2

3.3 LOAD CALCULATION

The exact calculation of load due to blasting is very difficult because during blasting, a large amount of energy is releasing within a short period of time. There are many analytical and experimental studies are conducted to determine the load due to different kind of blasting. Such kind of studies results various standard codes used for the design of blast resistance structure. Also, some empirical formulas were formulated for the calculation of the blast load. IS:4991-1968 and UFC 3-340-02 are the two codes used for the design of blast resisting structure and calculation of accidental load. The Most commonly accepted relations are those proposed by Kinney and Graham due to their close proximity with the experimental results. The empirical formulas by Kinney and Graham are shown below:

$$\frac{P_s}{P_0} = \frac{808 \left(1 + \left(\frac{Z}{4.5}\right)^2\right)}{\sqrt{\left(1 + \left(\frac{Z}{0.048}\right)^2\right)} \times \sqrt{\left(1 + \left(\frac{Z}{0.32}\right)^2\right)} \times \sqrt{\left(1 + \left(\frac{Z}{1.35}\right)^2\right)}}$$

$$\frac{t_s}{W^{1/3}} = \frac{980 \left[1 + \left(\frac{Z}{0.54}\right)^{10}\right]}{\left[1 + \left(\frac{Z}{0.02}\right)^3\right] \times \left[1 + \left(\frac{Z}{0.74}\right)^6\right] \times \sqrt{\left[1 + \left(\frac{Z}{6.9}\right)^2\right]}}$$

$$U_s = \sqrt{\frac{6P_s + 7P_0}{7P_0}} * a_0$$

$$P_0 = \frac{6P_s + 7P_0}{P_s + 7P_0} * \rho_0$$

$$q_s = \frac{5P_s^2}{2(P_s + 7P_0)}$$

$$P_r = 2P_s \left\{ \frac{7P_0 + 4P_s}{7P_0 + P} \right\}$$

Where,

Z - Standoff distance

P₀ - Ambient atmospheric pressure

P_s - Peak incident overpressure

t_s - Positive phase duration

a₀ - Speed of sound in air

ρ₀ - Density of air ahead of blast wave

ρ_s - Air density behind wave front

U_s - Blast wave front velocity

q_s - Maximum dynamic pressure.

P_r - Reflected pressure

Blast parameters depend upon, the distance of source of explosion and the energy released by that explosion. Blast wave scaling law is the distance of the detonation point from the structure of interest. It is considered one of the most critical parameters for blast loading computations, because of the peak pressure and wave strength decrease by the distance from the explosion resource. The expression for scaling law is given below.

$$Z = \frac{R}{W^{1/3}}$$

Where,

Z - Scaled distance

R - Distance from the detonation source to the point of interest

W - Weight of the explosive (charge, TNT) (kg)

Scaling law provide the peak pressure and time durations. They may be found from the peak values given in IS: 4991-1968.

An explosion can be defined as a huge quantity of energy released within few milliseconds. Figure 3.3 shows pressure and time relations of structural loadings produced from charge. The blast wave characteristics are truly dependent on distance of structure (standoff distance) from the centre of charge (w) with time (t). The peak positive pressure (P_{so}) is known as peak pressure or maximum pressure, ambient air pressure (P_o), as we can see from the figure 3.2 P_o is zero at arrival time (t_a), there is a sudden increase in peak positive pressure (P_{so}) at positive time (t_d) this is called peak positive over pressure. As the duration increases negative pressure occurs at a negative time (t_n) this is called under pressure here pressure is lesser than ambient pressure (P_o). For the simplicity in the analysis process, only positive peak pressure (P_{so}) is considered by neglecting under pressure. A triangular blast load profile is used for peak positive pressure (P_{so}) for a positive duration (t_d) as shown in figure 3.3.

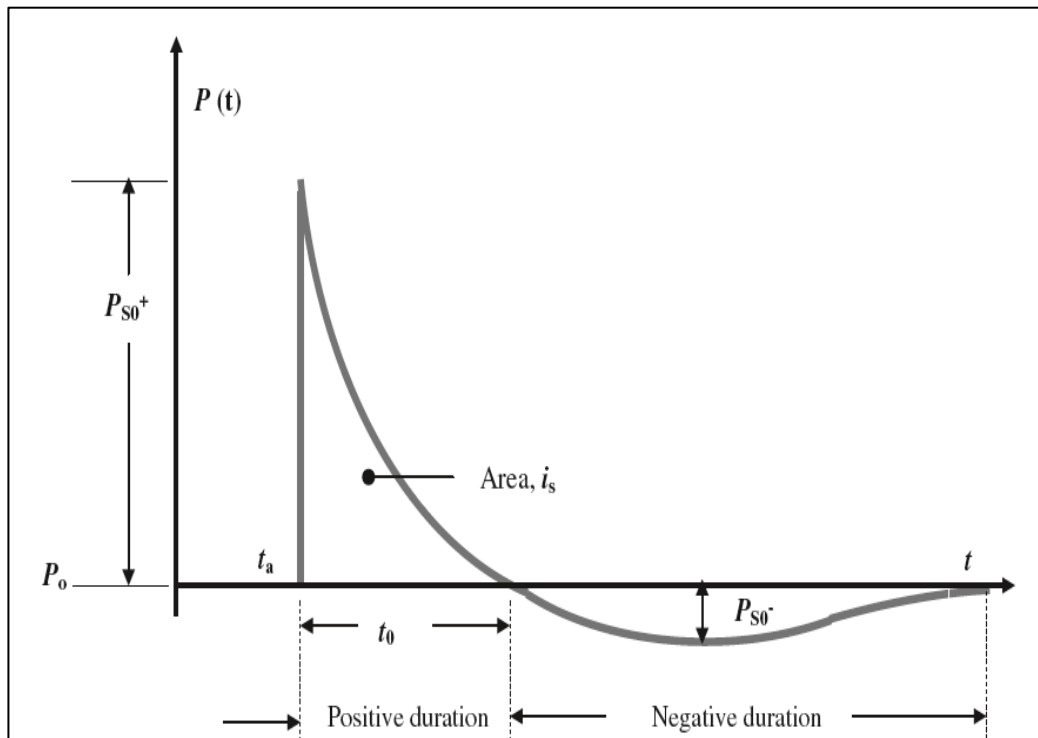


Figure 3.2 Pressure time history of blast wave (Goel,2015)

In this study, the load calculations are done on the basis of IS:4991-1968. The code provides a detailed tabulated data of the explosion of 1 tonne of TNT explosive shown in figure 3.3. It can be used for finding out the peak reflected over pressure ratio. The variation of peak reflected over pressure ratio with the standoff distance is plotted by using the values from figure 3.3. IS code provides the peak reflected overpressure ratio up to a standoff distance of 99m for the explosion of 1 tonne TNT explosive. But the

thesis work requires the peak reflected overpressure ratio for standoff distance up to 150m. So, the data provided in the IS:4991-1968 is extrapolated with the help of MATLAB software. The approximately extended and modified plot between peak reflected overpressure versus standoff distance is shown in figure 3.4. This plot can be used to find out the peak reflected overpressure at various standoff distance up to 150m. By taking the values of standoff distance, positive phase duration and peak reflected overpressure ratio, the data required for defining the time history functions are also formulated.

DISTANCE, m x	PEAK SIDE ON OVER- PRESSURE RATIO P_{so}/P_a	MACH NO. M	POSITIVE PHASE DURATION t_p , milli- secs	DURATION OF EQUIVALENT TRIANGULAR PULSE t_d , milli-secs	DYNAMIC PRESSURE RATIO q_0/P_a	PEAK RE- FLECTED OVERPRES- SURE RATIO P_{ro}/P_a
(1)	(2)	(3)	(4)	(5)	(6)	(7)
15	8.00	2.80	9.50	5.39	10.667	41.60
18	5.00	2.30	11.00	7.18	5.208	22.50
21	3.30	1.96	16.38	9.33	2.643	12.94
24	2.40	1.75	18.65	11.22	1.532	8.48
27	1.80	1.60	20.92	13.30	0.920	5.81
30	1.40	1.48	22.93	15.39	0.583	4.20
33	1.20	1.42	24.95	16.31	0.439	3.45
36	1.00	1.36	26.71	17.94	0.312	2.75
39	0.86	1.32	28.22	19.20	0.235	2.28
42	0.76	1.28	29.74	20.20	0.186	1.97
45	0.66	1.25	31.25	21.60	0.142	1.66
48	0.59	1.23	32.26	22.70	0.115	1.46
51	0.53	1.20	33.52	23.70	0.093	1.28
54	0.48	1.19	34.52	24.70	0.077	1.14
57	0.43	1.17	35.53	26.40	0.062	1.01
60	0.40	1.16	36.29	26.60	0.054	0.93
63	0.37	1.15	37.30	27.80	0.046	0.85
66	0.34	1.14	38.05	28.76	0.039	0.77
69	0.32	1.13	38.81	29.25	0.035	0.72
72	0.30	1.12	39.56	29.87	0.031	0.67
75	0.28	1.11	40.32	30.71	0.027	0.62
78	0.26	1.104	40.82	31.85	0.023	0.58
81	0.25	1.100	41.58	31.92	0.022	0.55
84	0.24	1.098	42.34	32.00	0.020	0.53
87	0.23	1.095	42.84	32.26	0.018	0.50
90	0.22	1.086	43.60	33.39	0.016	0.47
93	0.20	1.082	44.35	34.70	0.014	0.43
96	0.19	1.077	45.46	35.37	0.013	0.41
99	0.18	1.072	45.61	36.22	0.012	0.40

Figure 3.3 Blast parameters from ground burst of one tonne explosive (IS:4991-1968)

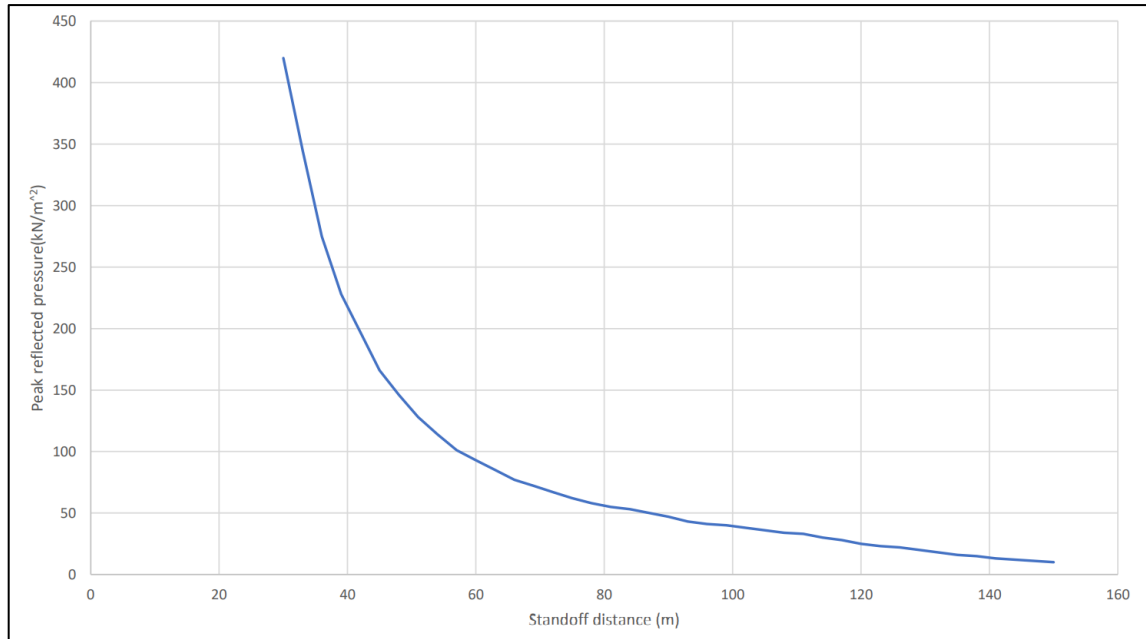


Figure 3.4 Peak reflected pressure vs. standoff distance

3.3.1 JOINT LOAD CALCULATION

The actual effect of blast load on structure is very difficult to predict. For the calculation of the building deformations due to the blast load requires lots of assumptions and historical data related to the blast load. For the easiness of computation of the load due to blasting, here the load is considered as joint loads. Here two different location of blasting point is considered. These are blasting at the front face of the building and blasting at the corner face of the building. A schematic representation of the cases is shown in figure 3.5. The influence area of the blast load at each joint is calculated as shown in figure 3.6. The areas at center bottom joints are shaded as red color, as that of top corner joints is blue and center joints is green. This influence area multiplied with peak reflected pressure gives the load on each joint.

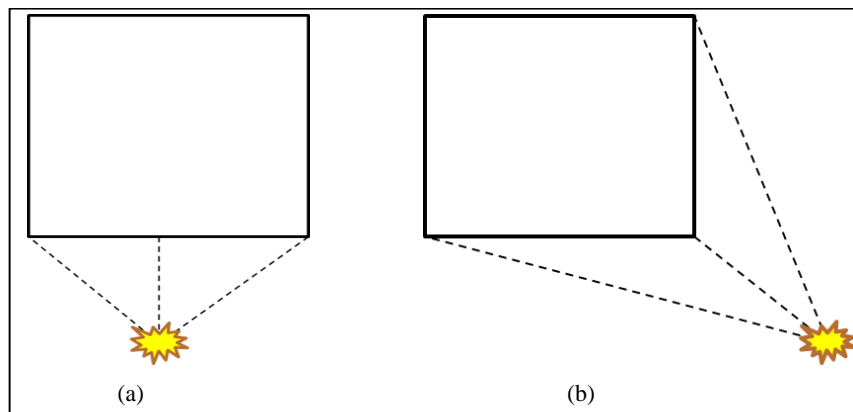


Figure 3.5 Location of the blast load (a) front face (b) corner side

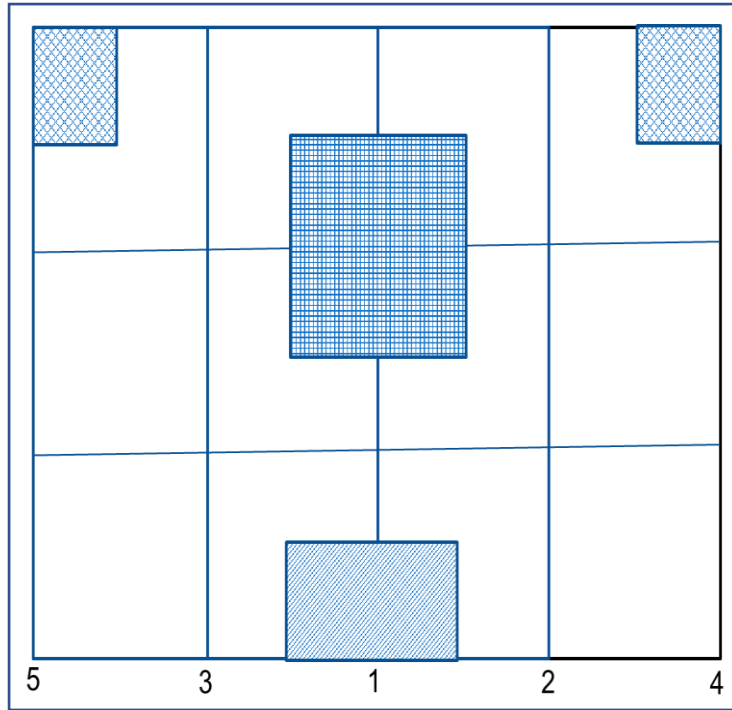


Figure 3.6 Load influence area at joints (Elevation view)

3.3.1.1 JOINT LOAD WHEN BLAST AT FRONT FACE

In the first case, the location of blast is at front face of the building as shown in figure 3.7. For the study, it is assumed that the blasting is happening at the exact center portion of the front face of the building. The shortest distance between the blast point and the front face of the building is called as standoff distance. The energy generated due to the blast will be applied on the building face in different type of pressure like overpressure on the top, overpressure on the side and reflected pressure on the front face of the building. In this study for analysis of the building response, only the reflected pressure on the front face is considered. For making the load data, the blast load at every joint on each floor is calculated as shown in table 3.2 to table 3.6. For this, five different standoff distance 20m, 30m, 40m, 50m and 60m are considered for making the load data table. Here the standoff distance is represented as R . By using the equation mentioned in 3.3, the scaled distance is calculated which is represented as Z . Then the blast pressure is obtained from the plot between peak reflected pressure versus standoff distance. The load at each joint is then found out by multiplying the peak reflected pressure with the corresponding influence area at each joint.

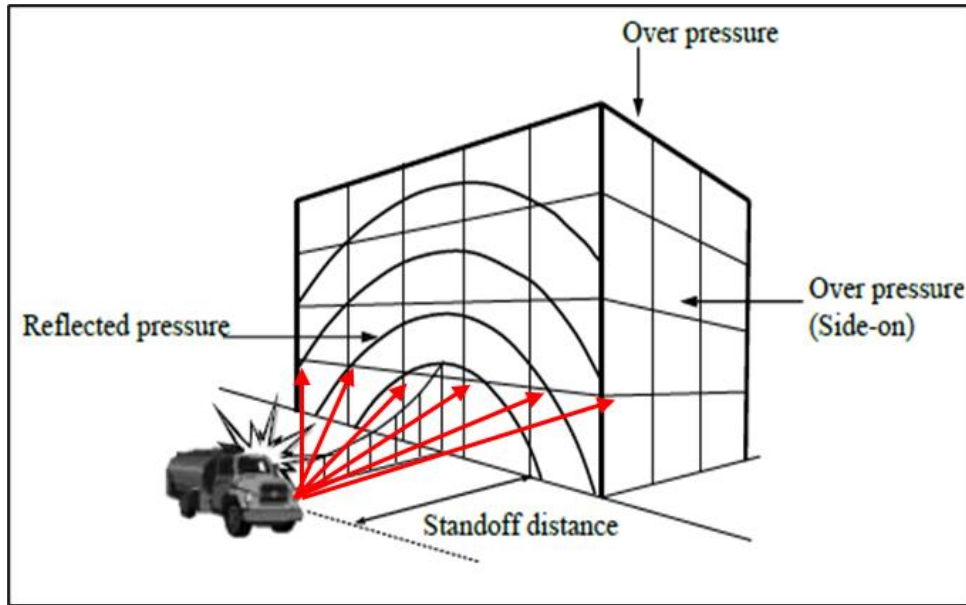


Figure 3.7 Blast load at the front face of the building

Table 3.2 Joint load at R=20m

FL	JOINT	R (m)	Z (m)	P (kN/m ²)	A (m ²)	F (kN)
GL	1	20.00	43.08	185.60	8.75	1624.00
	2&3	20.62	44.42	171.40	8.75	1499.75
	4&5	23.36	50.32	131.80	4.38	577.28
1	1	20.30	43.74	178.50	17.50	3123.75
	2&3	20.92	45.07	165.60	17.50	2898.00
	4&5	23.62	50.89	128.80	8.75	1127.00
2	1	21.19	45.65	161.00	17.50	2817.50
	2&3	21.78	46.92	152.30	17.50	2665.25
	4&5	24.39	52.55	120.50	8.75	1054.38
3	1	22.59	48.67	141.80	17.50	2481.50
	2&3	23.14	49.85	134.50	17.50	2353.75
	4&5	25.61	55.18	108.50	8.75	949.38
4	1	24.41	52.59	120.30	17.50	2105.25
	2&3	24.92	53.69	115.30	17.50	2017.75
	4&5	27.23	58.67	96.40	8.75	843.50
5	1	26.58	57.26	100.20	17.50	1753.50
	2&3	27.04	58.25	97.40	17.50	1704.50
	4&5	29.19	62.89	85.30	8.75	746.38
6	1	29.00	62.48	86.30	8.75	755.13
	2&3	29.43	63.40	83.90	8.75	734.13
	4&5	31.41	67.67	74.10	4.38	324.56

Table 3.3 Joint load at R=30m

FL	JOINT	R (m)	Z (m)	P (kN/m ²)	A (m ²)	F (kN)
GL	1	30.00	64.63	80.30	8.75	702.63
	2&3	30.41	65.51	78.10	8.75	683.38
	4&5	31.62	68.12	73.40	4.38	321.49
1	1	30.20	65.06	79.30	17.50	1387.75
	2&3	30.61	65.94	77.10	17.50	1349.25
	4&5	31.81	68.53	72.70	8.75	636.13
2	1	30.80	66.35	76.30	17.50	1335.25
	2&3	31.20	67.22	74.80	17.50	1309.00
	4&5	32.39	69.82	70.50	8.75	616.88
3	1	31.78	68.47	72.80	17.50	1274.00
	2&3	32.17	69.31	71.50	17.50	1251.25
	4&5	33.32	69.63	70.90	8.75	620.38
4	1	33.10	71.31	68.10	17.50	1191.75
	2&3	33.48	72.13	66.60	17.50	1165.50
	4&5	34.58	74.50	62.70	8.75	548.63
5	1	34.73	74.82	62.30	17.50	1090.25
	2&3	35.09	75.60	61.20	17.50	1071.00
	4&5	36.17	77.86	58.20	8.75	509.25
6	1	36.62	72.52	66.10	8.75	578.38
	2&3	36.96	79.62	56.20	8.75	491.75
	4&5	37.96	81.78	53.40	4.38	233.89

Table 3.4 Joint load at R=40m

FL	JOINT	R (m)	Z (m)	P (kN/m ²)	A (m ²)	F (kN)
GL	1	40	86.18	48.26	8.75	422.28
	2&3	40.31	86.85	47.54	8.75	415.98
	4&5	41.23	88.83	45.49	4.38	199.25
1	1	40.15	86.50	47.92	17.5	838.60
	2&3	40.91	88.14	46.17	17.5	807.98
	4&5	41.39	89.17	45.15	8.75	395.06
2	1	40.61	87.49	46.85	17.5	819.88
	2&3	41.66	89.75	44.6	17.5	780.50
	4&5	41.82	90.10	44.26	8.75	387.28
3	1	41.36	89.11	45.22	17.5	791.35
	2&3	41.66	89.75	44.6	17.5	780.50
	4&5	42.57	91.71	42.76	8.75	374.15
4	1	42.38	91.30	43.13	17.5	754.78
	2&3	42.67	91.93	42.56	17.5	744.80
	4&5	43.71	94.17	40.61	8.75	355.34
5	1	43.66	94.06	40.7	17.5	712.25
	2&3	43.94	94.67	40.19	17.5	703.33
	4&5	44.79	96.50	30.72	8.75	268.80
6	1	45.18	97.34	38.07	8.75	333.11
	2&3	45.45	97.92	37.63	8.75	329.26
	4&5	46.27	99.69	36.35	4.38	159.21

Table 3.5 Joint load at R=50m

FL	JOINT	R (m)	Z (m)	P (kN/m ²)	A (m ²)	F (kN)
GL	1	50.00	107.72	31.26	8.75	273.53
	2&3	50.25	108.26	30.96	8.75	270.90
	4&5	51.00	109.88	30.06	4.38	131.66
1	1	50.12	107.98	31.11	17.50	544.43
	2&3	50.37	108.52	30.80	17.50	539.00
	4&5	51.12	110.13	29.94	8.75	261.98
2	1	50.49	108.78	30.67	17.50	536.73
	2&3	50.74	109.32	30.37	17.50	531.48
	4&5	51.48	110.91	29.54	8.75	258.48
3	1	51.09	110.07	29.97	17.50	524.48
	2&3	51.34	110.61	29.69	17.50	519.58
	4&5	52.07	112.18	28.89	8.75	252.79
4	1	51.92	111.86	29.05	17.50	508.38
	2&3	52.16	112.38	28.79	17.50	503.83
	4&5	52.89	113.95	28.02	8.75	245.18
5	1	52.97	114.12	27.93	17.50	488.78
	2&3	53.21	114.64	27.69	17.50	484.58
	4&5	53.92	116.17	26.98	8.75	236.08
6	1	54.23	116.83	26.69	8.75	233.54
	2&3	54.46	117.33	26.47	8.75	231.61
	4&5	55.15	118.82	25.82	4.38	113.09

Table 3.6 Joint load at R=60m

FL	JOINT	R (m)	Z (m)	P (kN/m ²)	A (m ²)	F (kN)
GL	1	60.00	129.27	21.92	8.75	191.80
	2&3	60.21	129.72	21.75	8.75	190.31
	4&5	60.83	131.05	21.34	4.38	93.47
1	1	60.10	129.48	21.84	17.50	382.20
	2&3	60.31	129.93	21.70	17.50	379.75
	4&5	60.93	131.27	21.27	8.75	186.11
2	1	60.41	130.15	21.63	17.50	378.53
	2&3	60.62	130.60	21.48	17.50	375.90
	4&5	61.23	131.92	21.07	8.75	184.36
3	1	60.91	131.23	21.28	17.50	372.40
	2&3	61.12	131.68	21.14	17.50	369.95
	4&5	61.73	132.99	20.74	8.75	181.48
4	1	61.61	132.73	20.81	17.50	364.18
	2&3	61.82	133.19	20.67	17.50	361.73
	4&5	62.42	134.48	20.29	8.75	177.54
5	1	62.50	134.65	20.24	17.50	354.20
	2&3	62.70	135.08	20.11	17.50	351.93
	4&5	63.30	136.38	19.74	8.75	172.73
6	1	63.57	136.96	19.59	8.75	171.41
	2&3	63.77	137.39	19.46	8.75	170.28
	4&5	64.35	138.64	19.12	4.38	83.75

3.3.1.2 JOINT LOAD WHEN BLAST AT CORNER SIDE

In the second case, the location of blast is at corner side of the building as shown in figure 3.8. Figure (a) shows the plan view of the building subjected to blasting at corner face and the figure (b) represents the 3D view of the building. Here it is assumed that the blasting is happening at the exact corner side of the building. Here also the shortest distance between the blast point and corner edge of the building is taken as standoff distance. For making the load data, the blast load at every joint on each floor is calculated. The load data corresponding to standoff distance 30 meter is shown in table 3.7. In this case also five different standoff distance 20m, 30m, 40m, 50m and 60m are considered for the calculation of load data table.

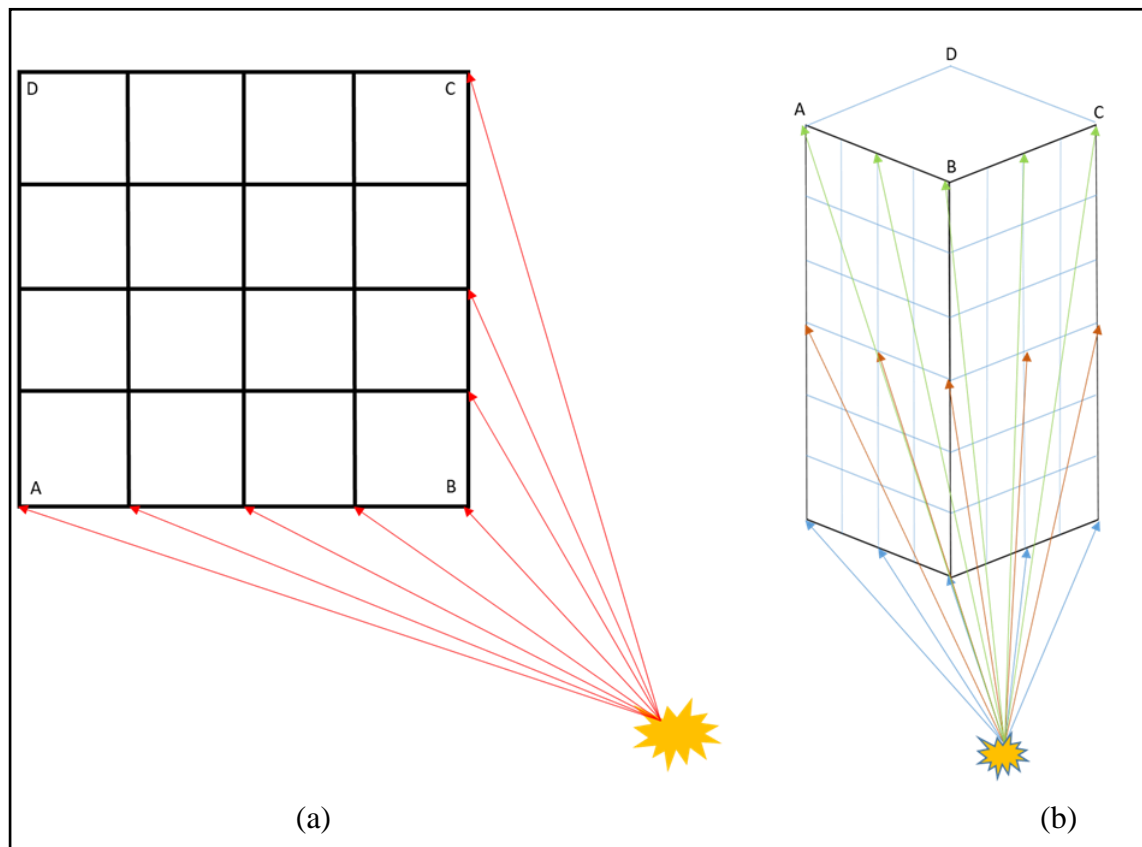


Figure 3.8 Blast at corner side of the building (a) plan view (b) 3D view

Table 3.7 Joint load due to blasting at corner for R=30m

FL	JOINT	R (m)	Z (m)	P (kN/m ²)	A (m ²)	F (kN)
GL	1	30.00	64.63	80.30	15.00	1204.50
	2&3	37.07	79.86	56.03	15.00	840.45
	4&5	44.14	95.10	39.84	7.50	298.80
1	1	30.15	64.96	79.53	30.00	2385.90
	2&3	37.19	80.13	55.67	30.00	1670.10
	4&5	44.24	95.32	39.68	30.00	1190.40
2	1	30.59	65.91	77.30	30.00	2319.00
	2&3	37.55	80.90	54.58	30.00	1637.40
	4&5	44.55	95.97	39.15	30.00	1174.50
3	1	31.32	67.48	74.53	30.00	2235.90
	2&3	38.15	82.18	52.99	30.00	1589.70
	4&5	45.05	97.05	38.25	30.00	1147.50
4	1	32.31	69.61	71.10	30.00	2133.00
	2&3	38.96	83.95	50.78	30.00	1523.40
	4&5	45.74	98.55	37.17	30.00	1115.10
5	1	33.54	72.26	66.58	30.00	1997.40
	2&3	39.99	86.16	48.23	30.00	1446.90
	4&5	46.62	100.44	35.81	30.00	1074.30
6	1	34.99	75.37	61.52	15.00	922.80
	2&3	41.21	88.78	45.59	15.00	683.85
	4&5	47.67	102.70	34.30	7.50	257.25

3.4 MODELLING

Response of the building subjected to blast load is analysed in ETABS 2019 by modeling a G+5 storied RC structure with varying conditions. A plan area of 20 m x 20 m is maintained for all the modelled buildings. For the analysis of blast response, the building is modelled as follows.

Model 1: Bare frame model

The following are specifications of the model:

Beam size: 300mm x 450mm

Column size: 600mm x 600mm

Plan dimension: 4 bays of 5m span in both X and Y direction

The plan and 3D view of the bare frame modeled in ETABS is shown in figure 3.9. For the modelling the material properties are provided as given in table 3.1.

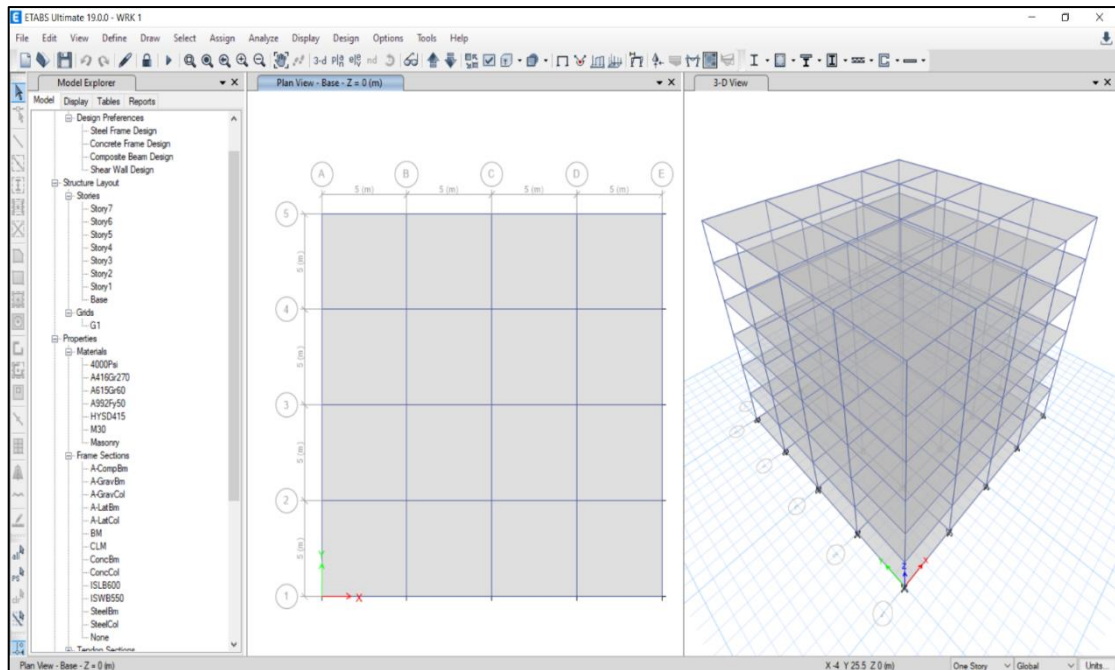


Figure 3.9 Bare frame model

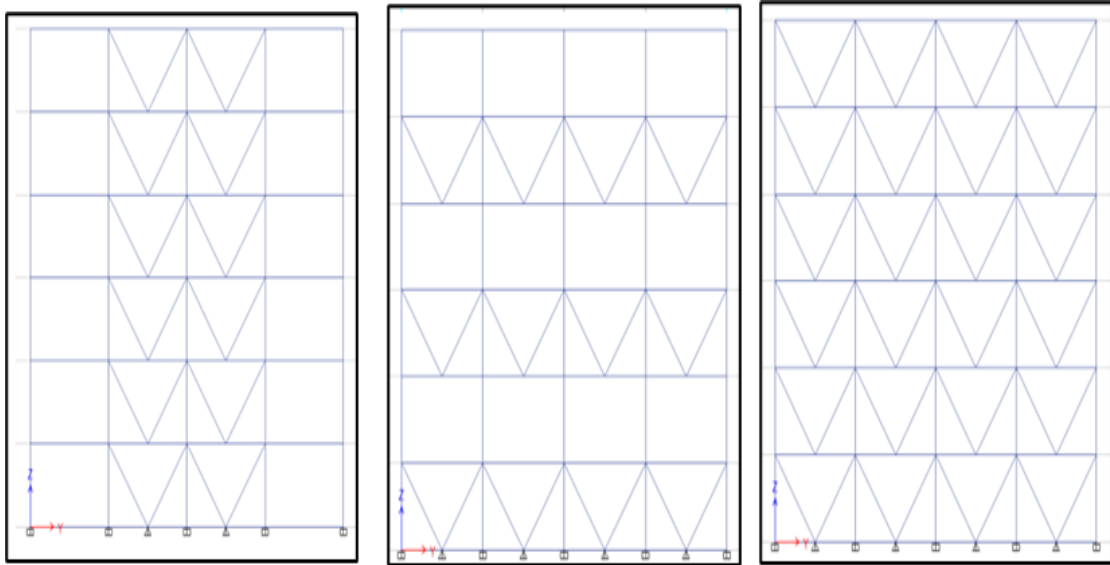
Model 2: Building with bracings

Generally, bracing helps to resist wind and seismic force in a more excellent way. It has the flexibility to design to achieve the demanded strength and stiffness. A significant advantage of providing bracing is the lowering in lateral displacement. The bracing can be of many types. Depending upon the user requirements and building specifications the type and arrangement of bracings in structures will change accordingly.

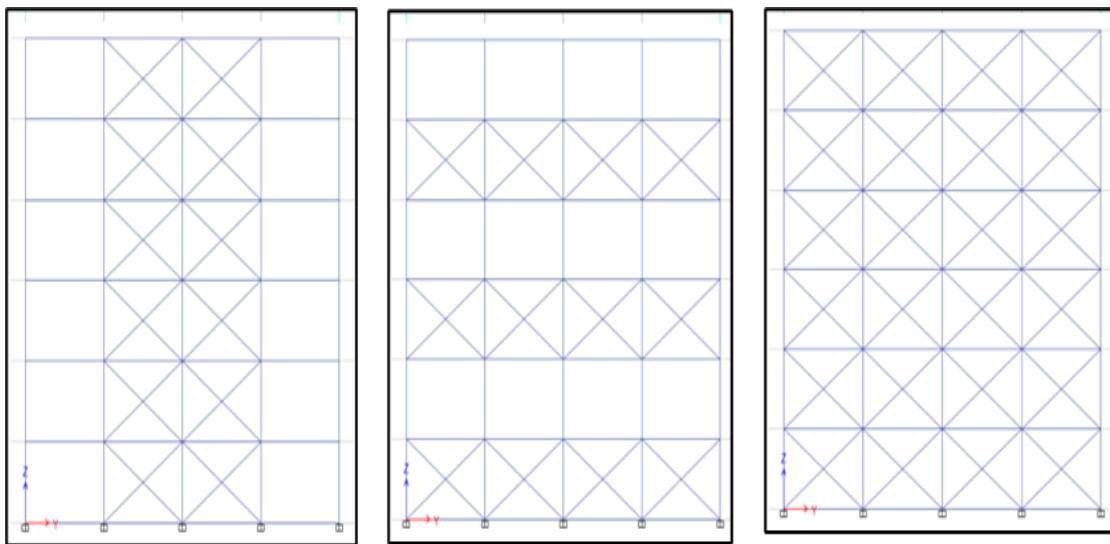
The model 2 has the same specifications of model 1 except bracings are added. Here two type of bracings is considered. These are V Bracings (ISA 150X150X10) and X Bracings (ISA 150X150X10). For the analysis purpose three different arrangement of both V and X bracings are considered. The three different cases are listed below

- Case 1: Bracings provided at bay 2 & 3 at all floor levels
- Case 2: Bracings provided at alternate floor levels
- Case 3: Bracings provided at all floor level

The elevations of the three different cases of V bracings are shown in figure 3.10. The elevations of the three different cases of X bracings are shown in figure 3.11. In all the cases the bracings are provided at the outer periphery of the building.



(a) (b) (c)
 Figure 3.10 Building with V bracings (a) case 1(b) case 2 (c) case 3



(a) (b) (c)
 Figure 3.11 Building with X bracings (a) case 1(b) case 2 (c) case 3

Model 3: Building with different column size

The size of the column has some significant effect on the overall strength of the structure. For determining the influence of column size on the response of structure under blast loading, models having different column sizes are modelled. Here four models are made each having different column sizes are analyzed. The size of the external and internal columns model is kept as same in all the four cases shown in table 3.8.

Table 3.8 Models and column sizes

Cases	Size of Column (mm x mm)	
	External	Internal
Case 1	600 x 600	600 x 600
Case 2	650 x 650	650 x 650
Case 3	700 x 700	700 x 700
Case 4	750 x 750	750 x 750

Model 4: Building with shear wall

Shear wall is a rigid vertical diaphragm capable of transferring lateral forces from exterior walls, floors, and roofs to the ground foundation in a direction parallel to their planes. Shear walls provide large strength and stiffness to buildings in the direction of their orientation, which significantly reduces lateral sway of the building and thereby reduces damage to structure and its contents. Regarding high stiffness and ductility, shear wall has a high energy dissipation capacity, making it a good choice for blast resisting systems. Generally, the strength of shear wall depends on the action of load on it. When shear wall subjected to in plane action of load it shows maximum strength. And when subjected to out plane action of load it shows minimum strength. By taking this behaviour of shear wall into consideration, four different arrangements of shear walls are considered for the analysis. The plan and 3D view of the four different arrangement of shear wall is shown in figure 3.12. The four different arrangements of shear wall are listed below.

- Case 1: Out plane loading in shear wall

The shear wall is subjected to out of plane action of the load due to blast. Here the shear wall of span 5 metre is provided in all the corner in such a way that it is parallel to the front face of the building as shown in figure 3.12 (a).

- Case 2: In plane loading in shear wall

The shear wall is subjected to in plane action of the load due to blast. Here the shear wall of span 5 metre is provided in all the corner in such a way that it is perpendicular to the front face of the building as shown in figure 3.12 (b).

- Case 3: Shear wall at corner

The shear wall is subjected to both in plane and out of plane action of the load due

to blast. Here two shear walls of span 3 metre is provided in all the corners perpendicular to each other as shown in figure 3.12 (c).

- Case 4: Shear wall at the core

The shear wall is subjected to both in plane and out of plane action of the load due to blast. Here a rectangular shear walls of span 10 metre is provided at the inner core of the structure as shown in figure 3.12 (d).

In all the cases, a reinforced concrete shear wall of wall thickness 300 mm is considered. The material specifications are same as that provided in table 3.1.

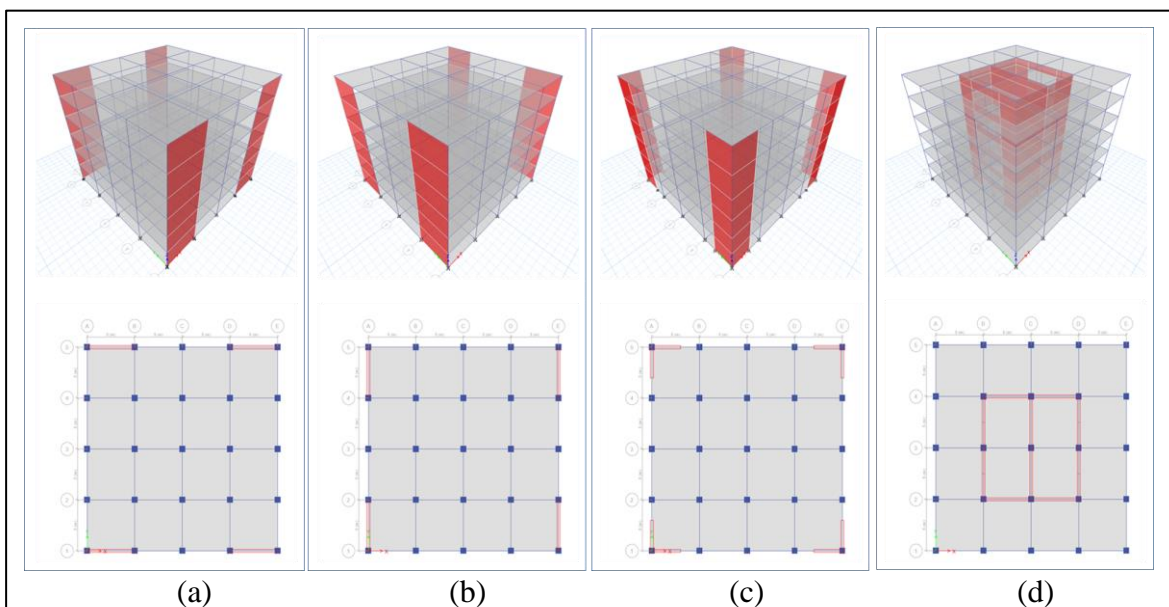


Figure 3.12 Arrangement of shear wall (a) Case 1 (b) Case 2 (c) Case 3 (d) Case 4

Model 5: Building with varying plan configuration

The arrangement and span of bays in both X and Y direction may have significant effect on the response of the structure. So here the study focusses to determine the best plan configuration for resisting the load due to blasting on the structure. For that building having four different plan configurations are considered. The plan view of the 4 cases are shown in figure 3.13. In all the cases the plan area is kept 400 m². With respect to the change in plan configuration the influence area of all the joints in the front face of the building will change. Thus, for finding the building response the load is calculated as multiplying the peak reflected pressure with the corresponding influence area of each joint in all the cases. The details of the four cases are as follows

- Case 1: Four bays of 6.25m in X direction and 4m in Y direction
- Case 2: Four bays of 4m in X direction and 6.25m in Y direction
- Case 3: Two central bays of span 6m and two corner bays of span 4m in both X and Y direction
- Case 4: Two central bays of span 4m and two corner bays of span 6m in both X and Y direction

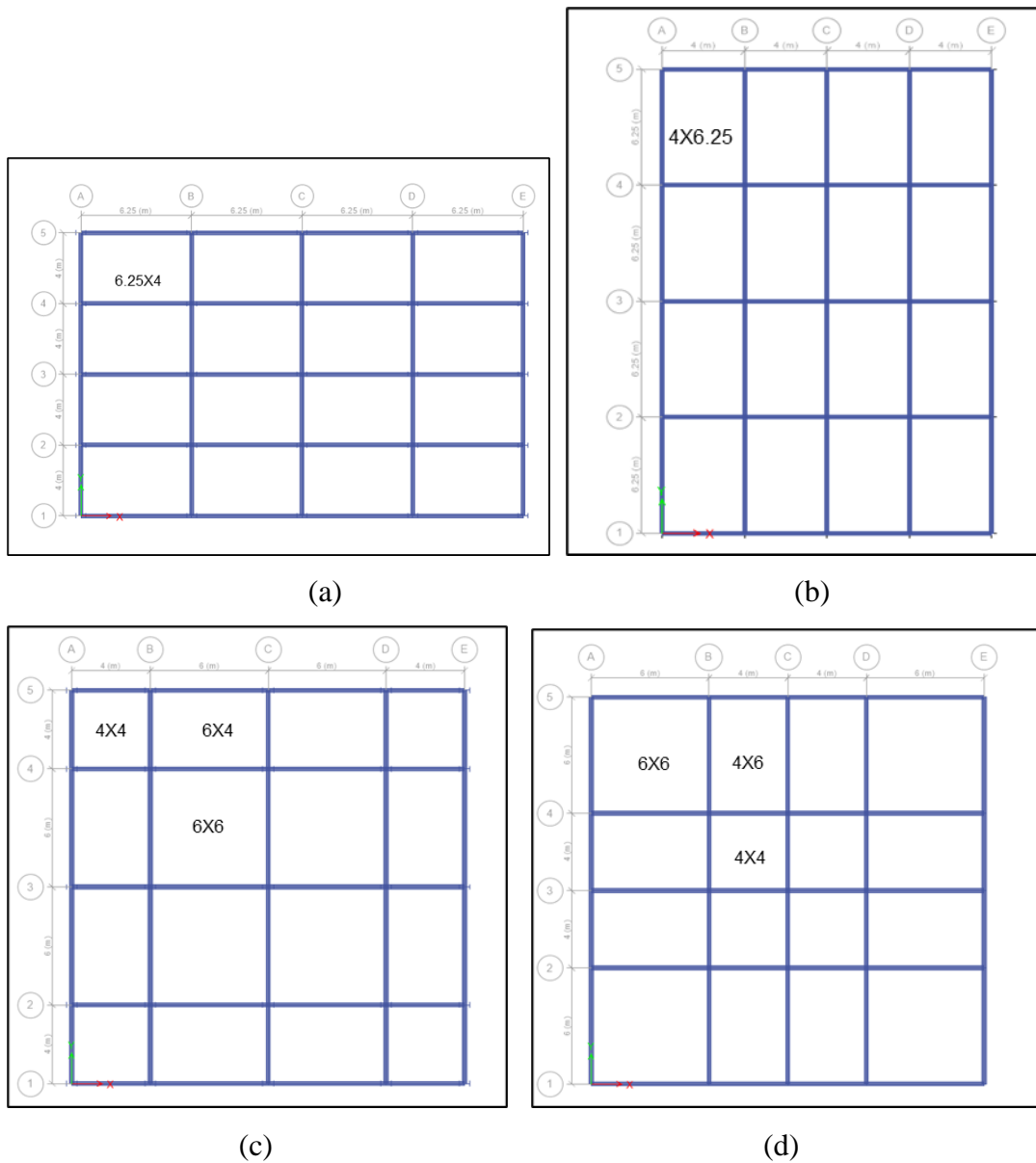


Figure 3.13 Different plan configuration (a) Case 1 (b) Case 2 (c) Case 3 (d) Case 4

3.5 STATIC ANALYSIS

The forces developed under a blast event are dynamic in nature, so the design of buildings for such forces must consider the dynamic nature of forces. However, for simple and regular structures with small to medium heights static analysis is considered sufficient and allowed by Universal building Code. Linear static analysis involves the calculation of joint load and its distribution along the height by formulae given in the IS:4991-1968. In ETABS the first step was to model the building. After modelling, load patterns and load combinations were defined. The calculated blast loads are applied to the corresponding joints.

The static analysis is carried out only on the bare frame model to find the range of safe standoff distance. The safe standoff distance is found out on the basis of two parameters namely lateral displacement and storey drift. Both lateral displacement and storey drift has to be within the acceptable limit for the building to be safe under blast load. As per IS: 456-2000 the lateral sway at the top of the building shall not exceed $H/500$, where H is total height of the building. The storey drift in any storey due to the minimum specified design lateral force, shall not exceed 0.004 times storey height. The response of the bare frame model subjected to blast load is found out for standoff distance of range 20m to 80m. Here by comparing the storey displacement and storey drift of the bare frame model with the permissible values, the range of the safe standoff distance is fixed.

3.6 DYNAMIC ANALYSIS

When blasting happens, a huge quantity of energy is released within a short period of time. The nature of forces thus produced are dynamic in nature. It is very difficult to accurately find the building response due to the dynamic action of the blast load. But on the basis of IS:4991-1968 and UFC 3-340-02 it is possible to determine the dynamic load due to blast load at various standoff distance. With the help of empirical formulae and code data, the time history functions are defined for each joint. The typical blast wave time history is defined using Friedlander's equation,

$$P_s(t) = P_{so} \left(1 - \frac{t}{t_o} \right) e^{-b \frac{t}{t_o}}$$

Where,

P_{so} is the peak overpressure

t_0 is the positive phase duration

t is the time elapsed

b is a decay coefficient of the waveform. Borgers and Vantomme proposed a formulation for the blast decay coefficient.

$$b = 1.5 \cdot Z^{-0.38}$$

Time history analysis is a step-by- step analysis of the dynamic response of a structure to a specified loading that may vary with time. For doing the dynamic analysis it is necessary to find the time history functions of blast load at different standoff distance for all the joints. The blast load parameters are computed as per IS: 4991-1968 and the blast load is multiplied with its tributary area and these pressures are applied as a joint load on the front face of the building i.e. in the direction of ‘x’ and pressure time history method is carried out. The time history curve defined for the joint 1 of story 3 of bare frame model is shown in figure 3.14.

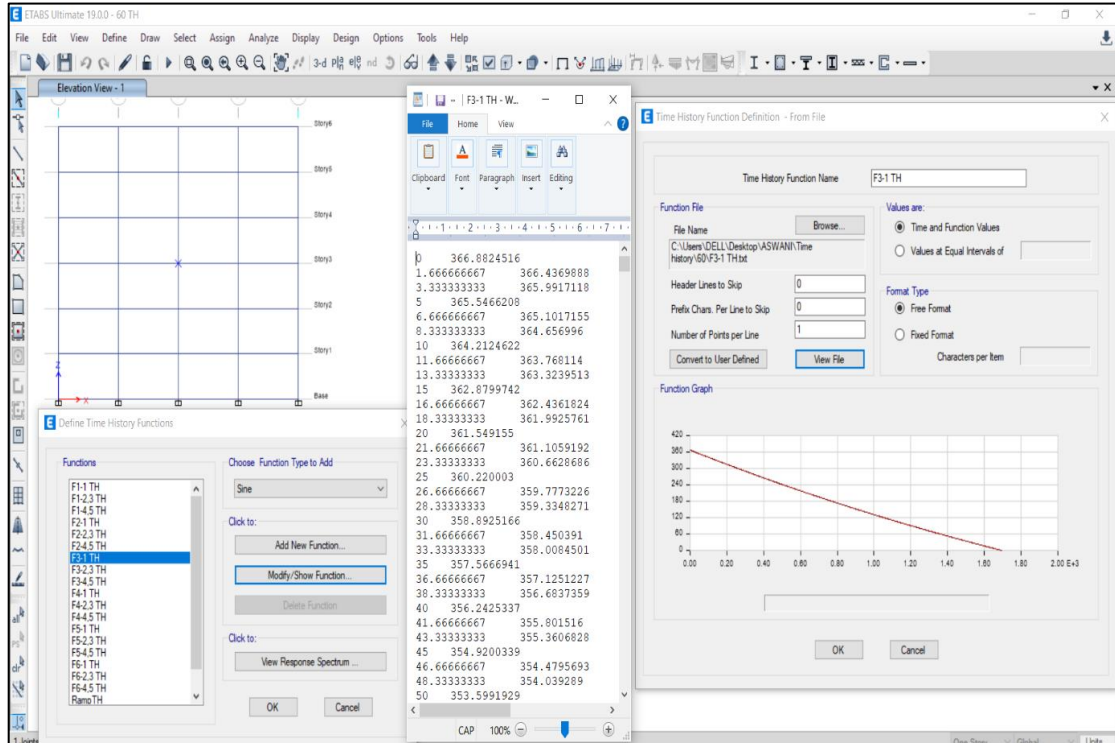


Figure 3.14 Time history function at joint 1 of story 3

For doing the dynamic analysis also, the safe standoff distance is found out on the basis of two parameters namely lateral displacement and storey drift. Both lateral displacement and storey drift has to be within the acceptable limit for the building to be safe under blast load. Firstly, the dynamic analysis is carried out to find the exact safe standoff distance of the bare frame model (model 1) within the range obtained from static analysis. After that, dynamic analysis is carried out on all the other modelled buildings and the variation of safe standoff distance is found out. The ultimate aim of the analysis is to find out the extent to which the safe standoff distance can be reduced for model 2, model 3, model 4 and model 5 in comparison to model 1 (bare frame model). The schematic representation of the safe standoff distance for different case are given in figure 3.15. where x is the safe standoff distance of bare frame model and y is the safe standoff distance of modified building.

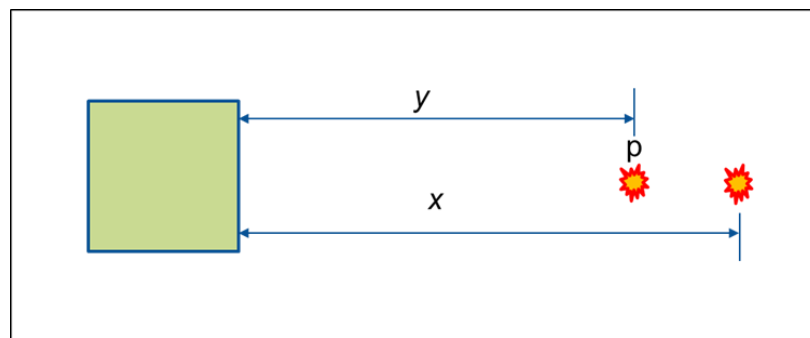


Figure 3.15 Safe standoff distance for different cases

3.7 LOCATION OF BLAST

The blast load acting on each joint is computed on the basis of the standoff distance and the influence area which vary with the location of the blasting. In this study two different blasting locations are considered.

- Location of blast at the front face
- Location of blast at the corner side

All the building models are analysed considering both the case of blast location and the responses are compared. The effect of blast location on the safe standoff distance is studied. In the first case the location of blast is at the front face of the building as shown in figure 3.7. Here the blast load is directly applied in the joints on the front face of the building and the corresponding building responses are analysed. For finding the safe standoff distance range the building the bare frame model is analysed statically. Then

for finding the exact safe standoff distance the building is analysed dynamically by defining time history function for all the joints. After that the time history functions are defined for all the joints in model 2 model 3 model 4 and model 5, then the model responses are found out dynamical. By comparing the lateral displacement and story drift the exact safe standoff distance is find out for all the models.

In the second case the location of blast is at the corner side of the building as shown in figure 3.8. Here the blast load is directly applied in the joints on the two faces adjacent to the corner of the building and the corresponding building responses are analysed. For finding the safe standoff distance range for the second case the bare frame model is analysed statically. Then for finding the exact safe standoff distance the building is analysed dynamically by defining time history function for all the joints. After that the time history functions are defined for all the joints in model 2 model 3 model 4 and model 5, then the model responses are found out dynamically. By comparing the lateral displacement and story drift the exact safe standoff distance is find out for all the models.

3.8 BUILDING RESPONSES

The response of the building when subjected to blast load need to be studied to properly understand the behaviour of the structure under blast load. The final step of the work is comparing the building response of all the models. Here the building responses like lateral displacement and story drift are taken into consideration. Those mentioned responses of the building subjected to blast at safe standoff distance is found out. Finally, a comparative analysis of all responses of the various models are done.

3.8.1 RESULT PARAMETERS

- Lateral displacement
As per IS: 456-2000 the lateral sway at the top of the building shall not exceed $H/500$ for lateral loads, where H is total height of the building. Here the height of the building is 18m. Thus, the permissible limit of lateral displacement is 36mm for the building.
- Storey drift
As per IS: 456-2000 the storey drift in any storey due to the minimum specified design lateral force, shall not exceed 0.004 per metre height of the building.

So, the building having lateral displacement less than or equal to 36mm and storey drift less than or equal to 0.004 per metre height of the building may be considered safe under the load due to blasting.

CHAPTER 4

SOFTWARE VALIDATION

The software is validated by using analytical study carried out by Charan and Deveraju (2018) was used. In this study, a G+5 storey RCC building is subjected to 100, 300 and 500kg charge weight of blast load with a standoff distance 30, 40 and 50m. The nonlinear time history analysis is carried out by using ETABS 2016. The response of the structure is determined in terms of displacement, velocity and acceleration, storey drift, beam forces, column forces and storey displacement. Table 4.1 shows the pressure and the joint load acting on the front face of the building acting at 30m at a charge weight of 100kg. Time history analysis of the building is carried out and the response of the building in terms of lateral displacement is observed.

Table 4.1: Pressure and the joint load acting on the front face of the building acting at 30m at a charge weight of 100kg Charan and Deveraju (2018)

Joint	FL	R in m	Z in m	P in kN/m ²	A in m ²	F in kN
1	GL	30.00	64.6	80.6	8.8	706
2 & 4		30.41	65.5	78.3	7.9	616
3 & 5		31.32	67.5	74.5	3.5	261
1	1	30.20	65.1	79.5	16.3	1291
2 & 4		30.61	66.0	77.1	14.6	1128
3 & 5		31.52	67.9	73.8	6.5	480
1	2	30.70	66.1	76.8	15.0	1152
2 & 4		31.10	67.0	75.3	13.5	1017
3 & 5		31.99	68.9	72.1	6.0	433
1	3	31.47	67.8	74.0	15.0	1110
2 & 4		31.86	68.6	72.6	13.5	980
3 & 5		32.73	70.5	69.5	6.0	417
1	4	32.50	70.0	70.3	15.0	1055
2 & 4		32.88	70.8	68.9	13.5	931
3 & 5		33.72	72.7	65.9	6.0	395
1	5	33.77	72.8	65.7	7.5	493
2 & 4		34.14	73.5	64.4	6.8	435
3 & 5		34.95	75.3	61.6	3.0	185

For the software validation a G+5 storey building is subjected to 100 kg charge weight at 30m standoff distance. The response of the building in terms of storey displacement analysed in ETABS 2019 is shown in figure 4.1. The results given in the journal is compared with that obtained after analysis.

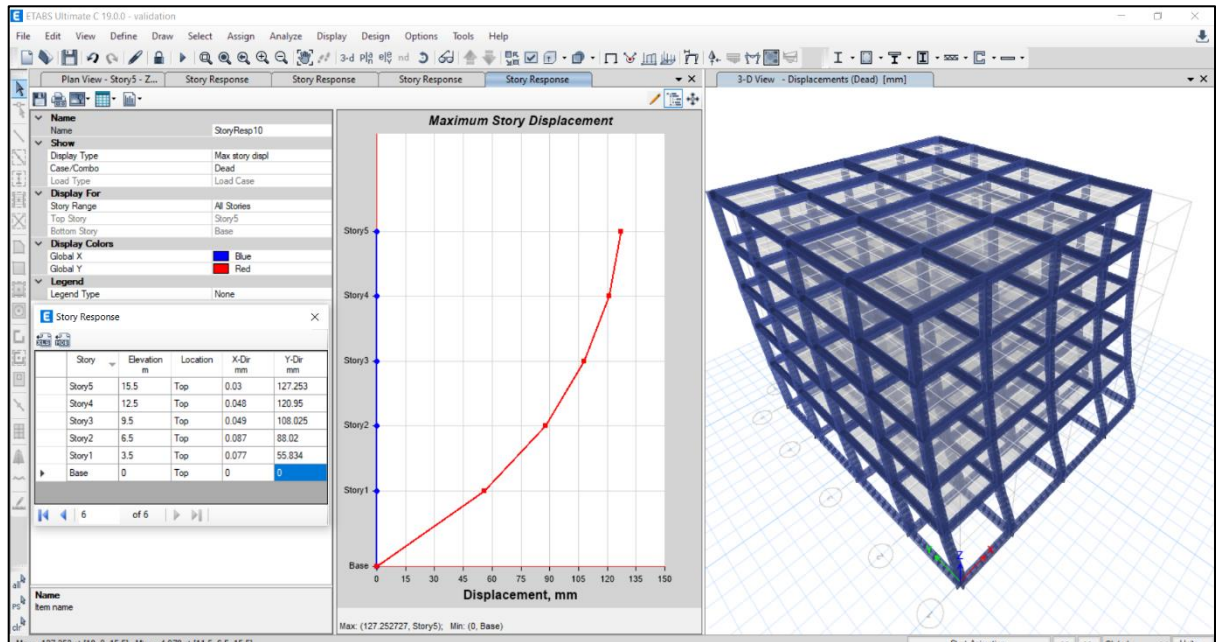


Figure 4.1 Storey displacement of G+5 storey building is subjected to 100 kg charge weight at 30m standoff distance

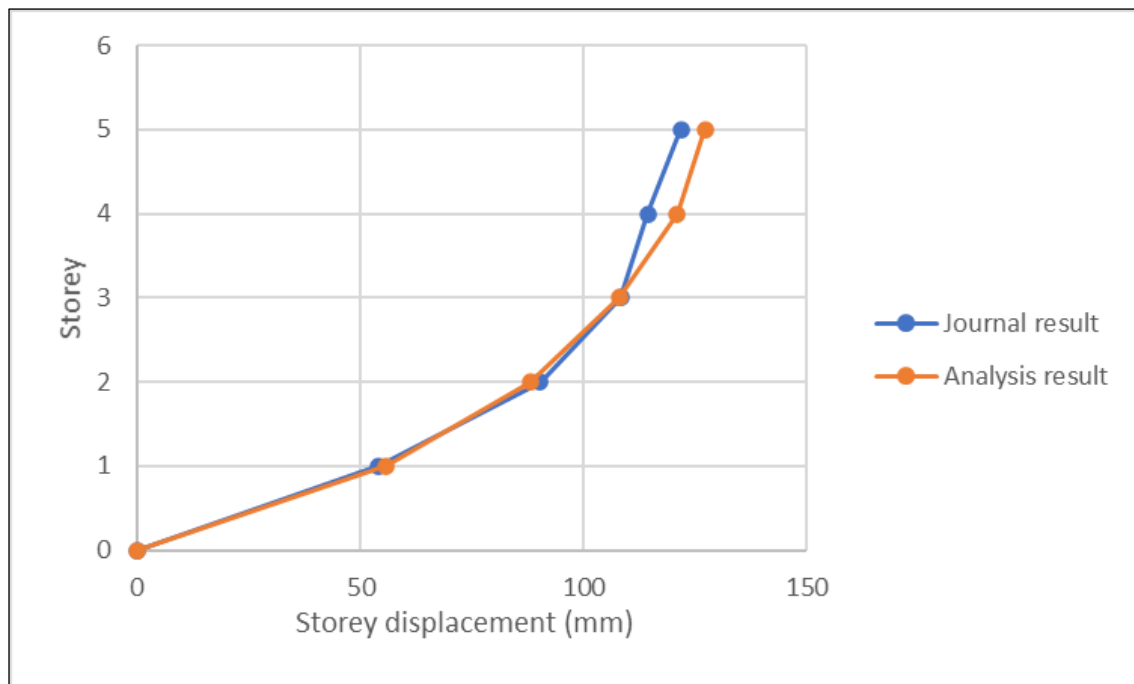


Figure 4.2 Comparison of storey displacement of G+5 storey building

Figure 4.2 shows the comparison of storey displacement obtained from journal and the after analysis in the ETABS software. The difference in the values obtained from journal and ETABS are compared by considering the area under the curves.

Area under the curve obtained from journal result= 428.17 units

Area under the curve obtained after analysis in software= 436.45 units

Percentage difference= 2.4%

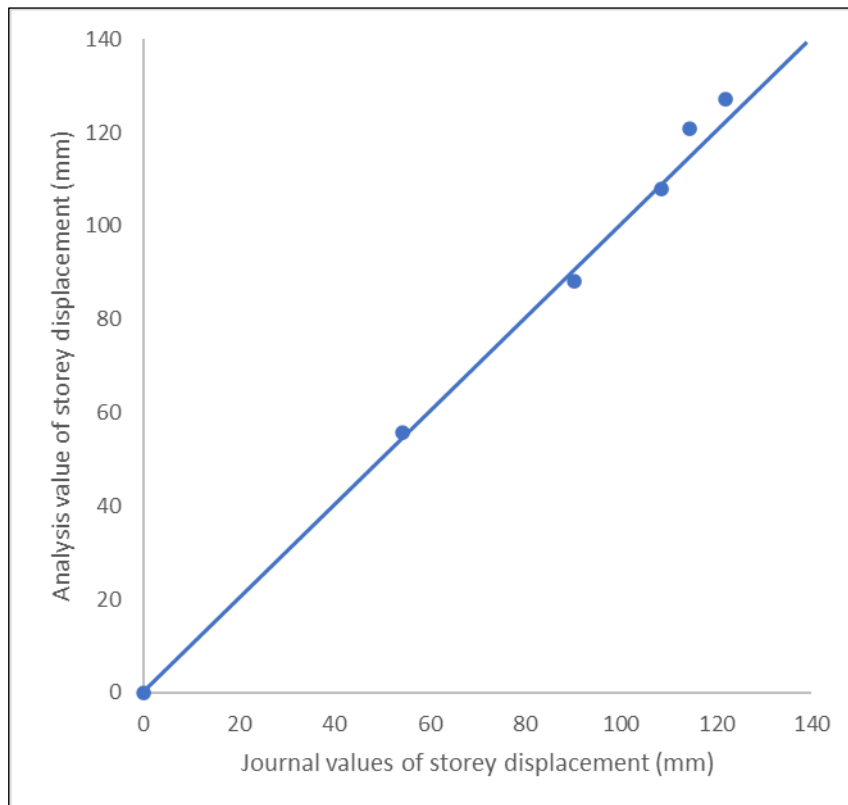


Figure 4.3 Parity curve for storey displacement

Comparison of the results from journal and that obtained after analysis is represented through parity curve as shown in figure 4.3. Comparing the results, it was found to be within acceptable limits. Hence the software is considered to be validated.

CHAPTER 5

RESULTS AND DISCUSSION

5.1 INTRODUCTION

The analysis of response of the G+5 storey building subjected to blast effect due to the blasting of 100kg TNT explosive at various locations are carried out. Building models with varying structural specifications are considered for the analysis. The structure is analysed by both statically and dynamically for finding the building responses. The ultimate aim of the thesis work is to find the safe standoff distance of the building subjected to blast load and to find the extent up to which it can be reduced when modifications are made in the building. The results obtained through the study and their subsequent inferences are discussed below.

5.2 SAFE STANDOFF DISTANCE OF BARE FRAME MODEL

The first step of the thesis work is to find the exact safe standoff distance of the bare frame model when the blasting is at the front face and corner side of the building. For that the building is analysed statically and dynamically. The static analysis is carried out for finding the range in which the safe standoff distance lies. Then, the building is analysed dynamically in that range for finding the exact value of the safe standoff distance. The static and dynamic analysis results of the bare frame model are discussed below.

5.2.1 STATIC ANALYSIS OF MODEL 1

The linear static analysis is carried out on the bare frame model to find the range of safe standoff distance. Model 1 is a bare frame model as mentioned in section 3.4. The safe standoff distance is found out by considering two parameters namely lateral displacement and storey drift. Both lateral displacement and storey drift has to be within the acceptable limit i.e., 36mm and 0.004/m respectively for the building to be safe under blast load. The calculated load for each joint in the bare frame model at different standoff distance are applied. The load data required for each case are calculated as mentioned in section 3.3.1. The static analysis of bare frame model with blasting at two different locations are discussed in the following sections:

❖ BLASTING AT THE FRONT FACE

The model subjected to the blast load due to the blasting at the front face is considered. The model is analysed statically in a standoff distance ranged between 20m to 60m for finding the safe standoff distance range. Firstly, the lateral displacement is taken into account. The lateral displacement of the structure due to blast at a standoff distance of 40m is shown in figure 5.1. The variation of lateral displacement with the standoff distance is found out and plotted as shown in figure 5.2. The lateral displacement shows a non-linear variation with the standoff distance. As the standoff distance increases the lateral displacement decreases. The curve shows a sudden decrease in lateral displacement when the standoff distance changes from 20m to 40m. Thereafter the lateral displacement varies gradually with standoff distance. The permissible limit of lateral displacement is less than or equal to 36mm for the building considered. So from the graph, it was found that safe standoff distance lies beyond 50m.

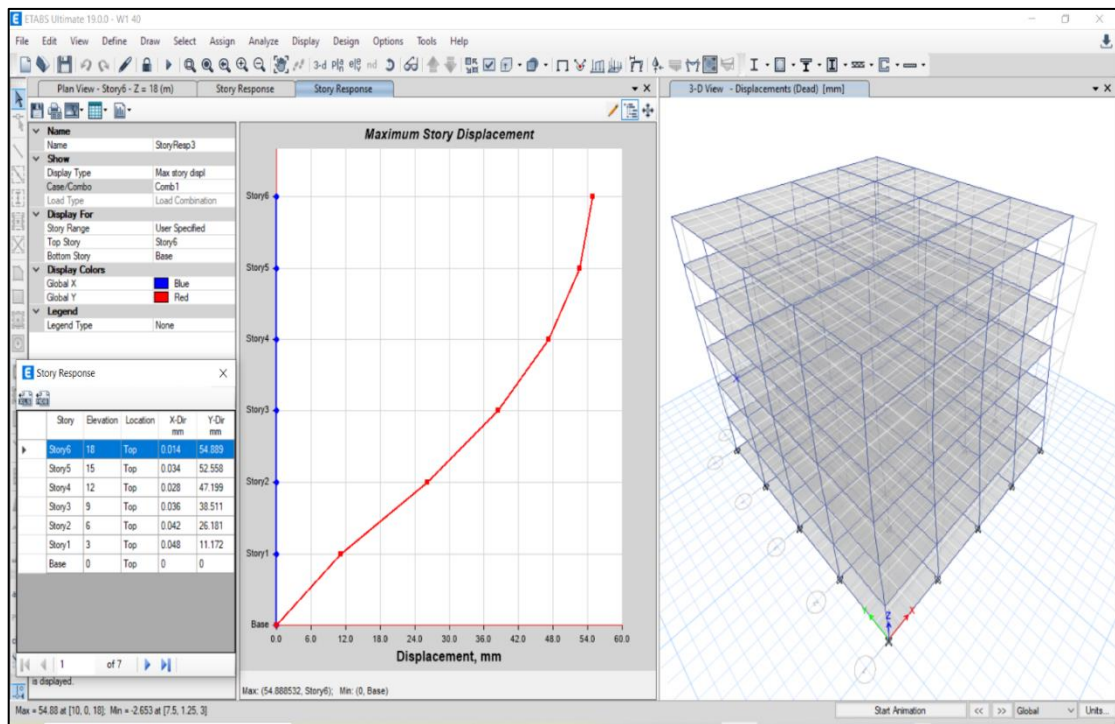


Fig 5.1 Storey displacement of model 1 at 40m standoff distance

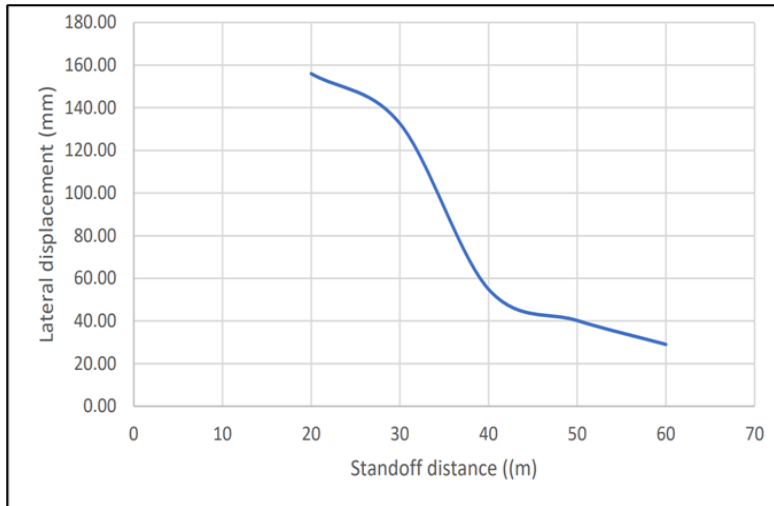


Figure 5.2 Lateral displacement vs standoff distance

The next parameter taken into account is the storey drift of the building. The storey drift of the structure due to blast at a standoff distance of 40m is shown in figure 5.3. The variation of storey drift with the standoff distance is found out and plotted as shown in figure 5.4. The storey drift shows a non-linear variation with the standoff distance. As the standoff distance increases the storey drift decreases. The curve shows a sudden decrease in storey drift when the standoff distance changes from 20m to 40m. Thereafter the storey drift varies gradually with standoff distance. The storey drift was found to be maximum at second storey of the model The permissible limit of storey drift is less than or equal to 0.004/m for the building considered. So from the graph, it was found that safe standoff distance lies beyond 46m.

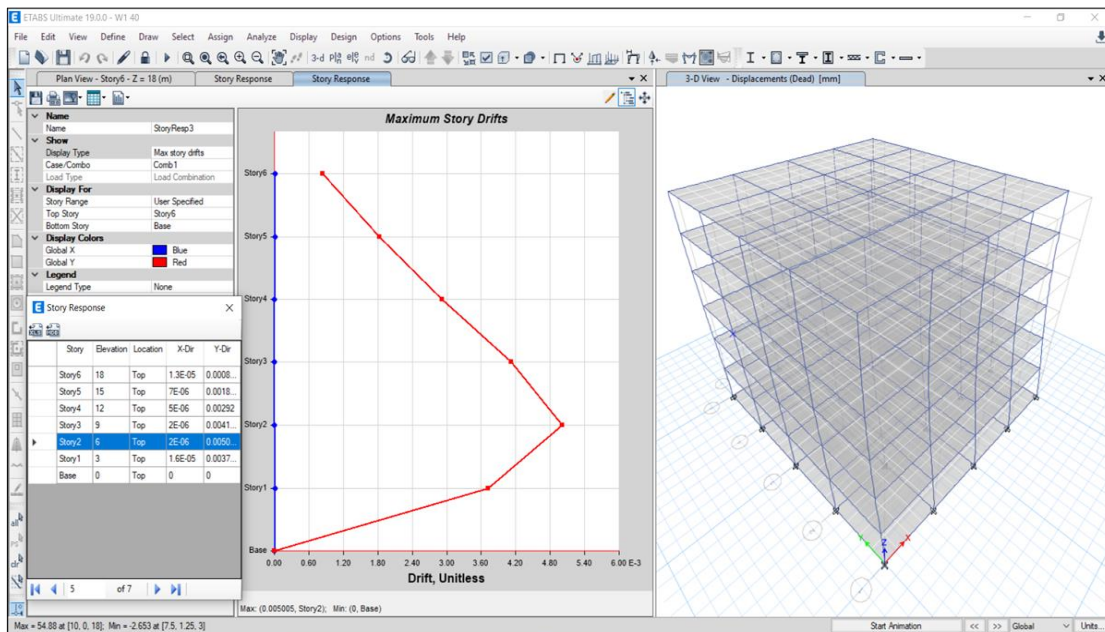


Figure 5.3 Storey drift of model 1 at 40m standoff distance

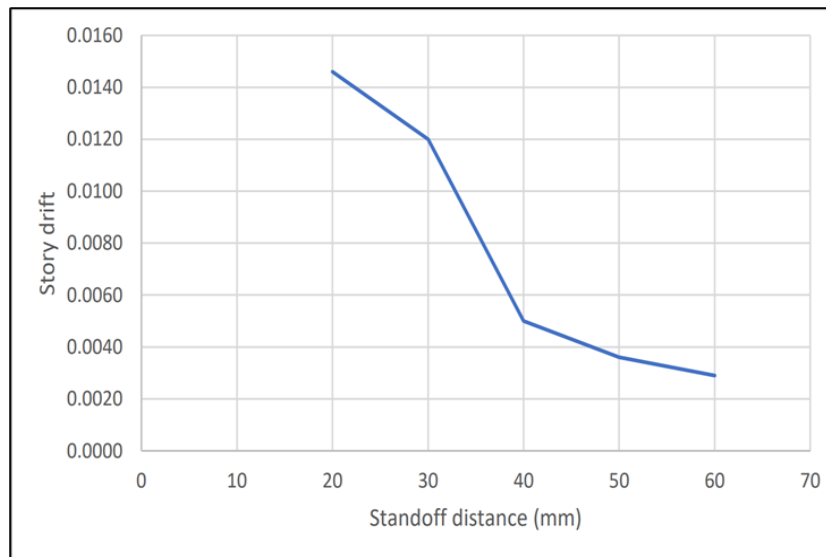


Figure 5.4 Storey drift versus standoff distance

By analysing the results of static analysis of the bare frame model, it can be concluded that the exact safe standoff distance lies between 50m to 60m, where the buildings has permissible values of lateral displacement and storey drift. So, the range of safe standoff distance for the dynamic analysis is fixed as 50m to 60m.

❖ BLASTING AT THE CORNER SIDE

The model subjected to the blast load due to the blasting at the corner side is considered. The model is analysed statically in a standoff distance ranged between 20m to 60m for finding the safe standoff distance range. Firstly, the lateral displacement is taken into account. The lateral displacement of the structure due to blast at a standoff distance of 20m is shown in figure 5.5. The variation of lateral displacement with the standoff distance is found out and plotted as shown in figure 5.6. The lateral displacement shows a non-linear variation with the standoff distance. As the standoff distance increases the lateral displacement decreases. The curve shows a sudden decrease in lateral displacement when the standoff distance changes from 20m to 40m. Thereafter the lateral displacement varies gradually with standoff distance. The permissible limit of lateral displacement is less than or equal to 36mm for the building considered. So, from the graph it was found that safe standoff distance lies beyond 40m.

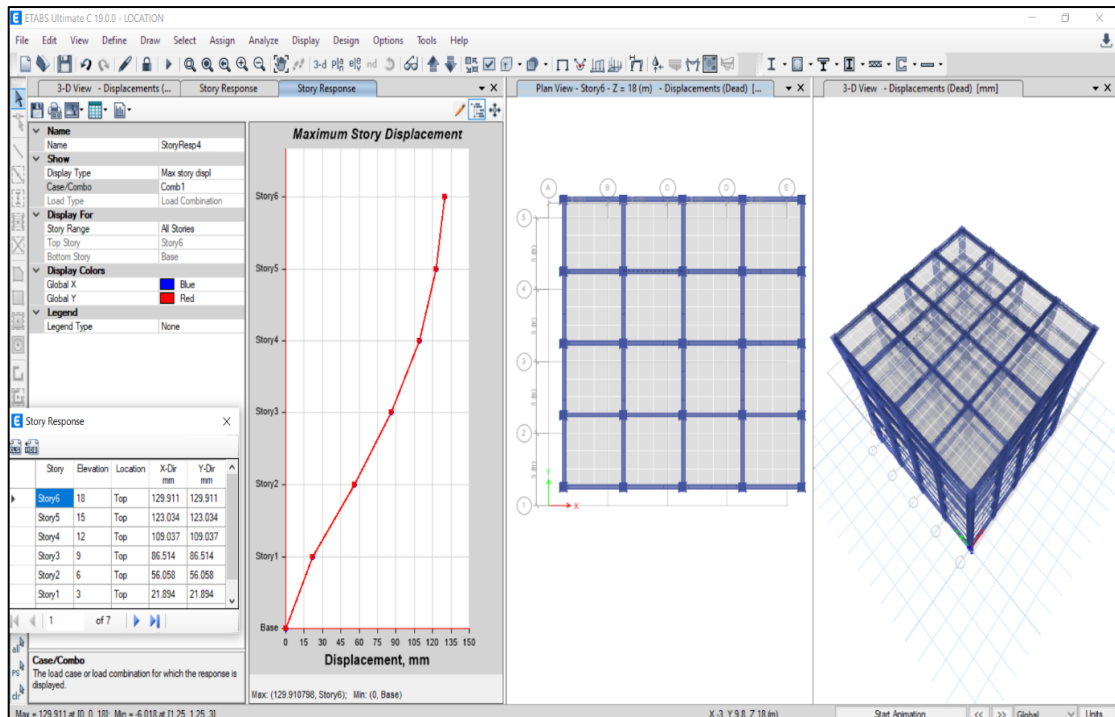


Figure 5.5 Storey displacement of model 1 at 20m standoff distance

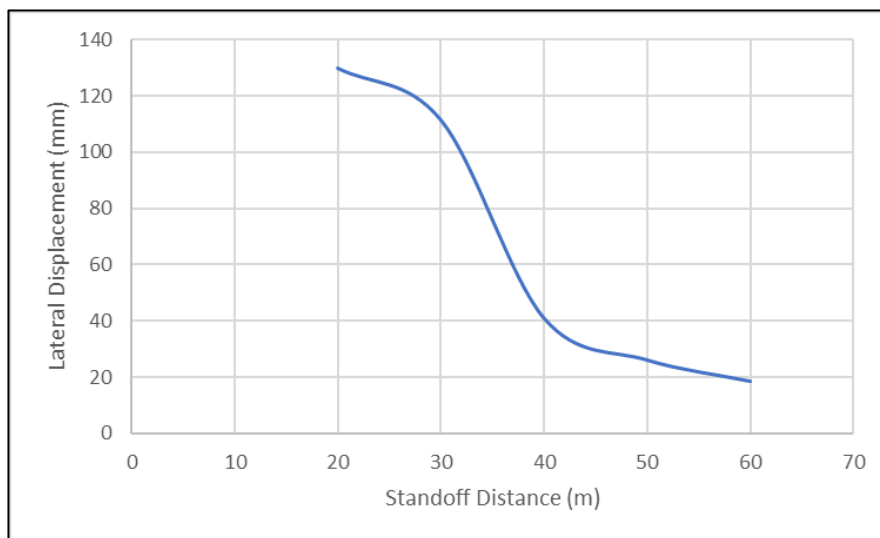


Figure 5.6 Lateral displacement vs standoff distance

The next parameter taken into account is the storey drift of the building. The storey drift of the structure due to blast at a standoff distance of 20m is shown in figure 5.7. The variation of storey drift with the standoff distance is found out and plotted as shown in figure 5.8. The storey drift shows a non-linear variation with the standoff distance. As the standoff distance increases the storey drift decreases. The curve shows a sudden decrease in storey drift when the standoff distance changes from 30m to 40m. Thereafter the storey drift varies gradually with standoff distance. The storey drift was found to be maximum at second storey of the model. The permissible limit of storey

drift is less than or equal to 0.004/m for the building considered. So from the graph, it was found that safe standoff distance lies beyond 38m.

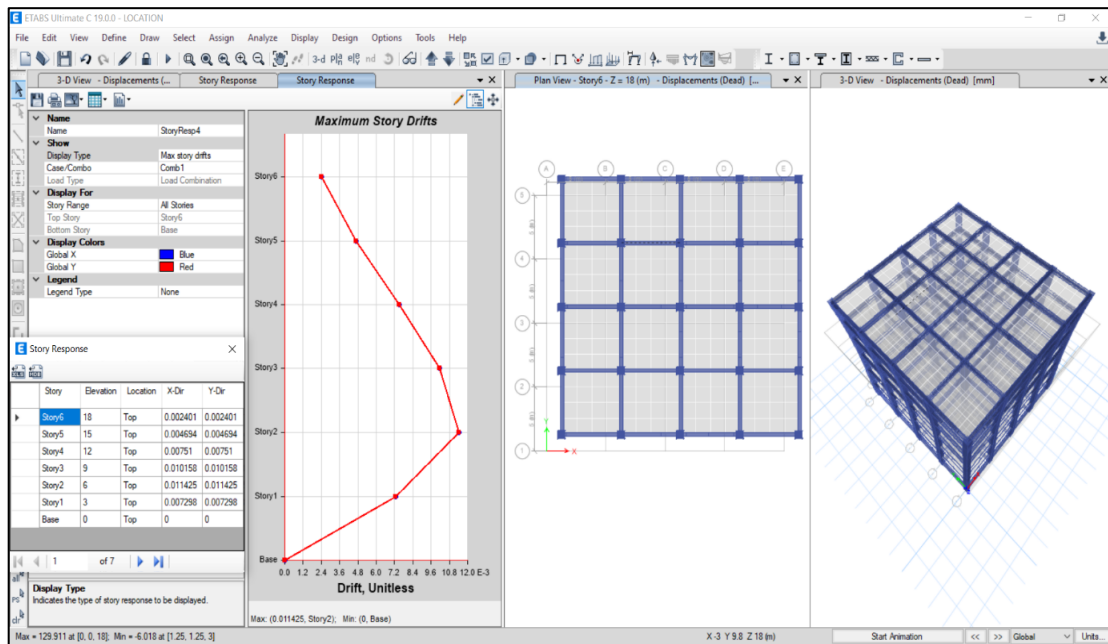


Figure 5.7 Storey drift of model 1 at 20m standoff distance

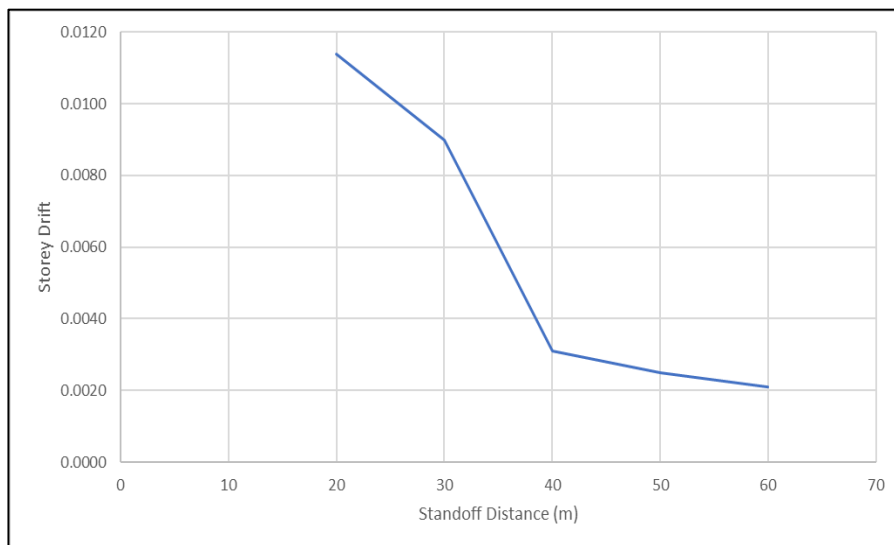


Figure 5.8 Storey drift versus standoff distance

By analysing the results of static analysis of the bare frame model subjected to blast load at corner side, it can be concluded that the exact safe standoff distance lies between 40m to 50m, where the buildings have permissible values of lateral displacement and storey drift. So, the range of safe standoff distance for the dynamic analysis is fixed as 40m to 50m.

5.2.2 DYNAMIC ANALYSIS OF MODEL 1

The effect of dynamic nature of the blast load on the structure can be identified performing dynamic analysis on the modelled building. In this step, the pressure time history analysis is carried out to find the exact safe standoff distance that lies between the range obtained from static analysis. The blast load parameters are computed as per IS: 4991-1968 and the blast load is multiplied with its tributary area and these pressures are applied as a joint load on the front face of the building i.e., in the direction of 'x' and pressure time history method is carried out. The dynamic analysis of bare frame model with blasting at two different locations are discussed in the following sections:

❖ BLASTING AT THE FRONT FACE

The dynamic analysis of model with blasting at the front face of the building is considered. The time history analysis is carried out on the building to find the exact safe standoff distance. From the static analysis of model 1 with front face blast load, the safe standoff distance was found to be within the range 50m to 60m. Thus, dynamic analysis is done with blasting at standoff distance of range 50-60m. The maximum lateral displacement and the storey displacement for each 1m between the range is determined. Fig 5.9 shows the lateral displacement of model 1 when subject to dynamic blast load at 60m standoff distance. The lateral displacement increases with the storey level. The variation of lateral displacement with the standoff distance as observed after the dynamic analysis is shown in figure 5.10. The lateral displacement is found to decrease almost linearly with the standoff distance. Thus, by comparing the lateral displacement with the permissible limit the exact safe standoff distance of model 1 was observed to be 53 m when subjected to blast load from the front face.

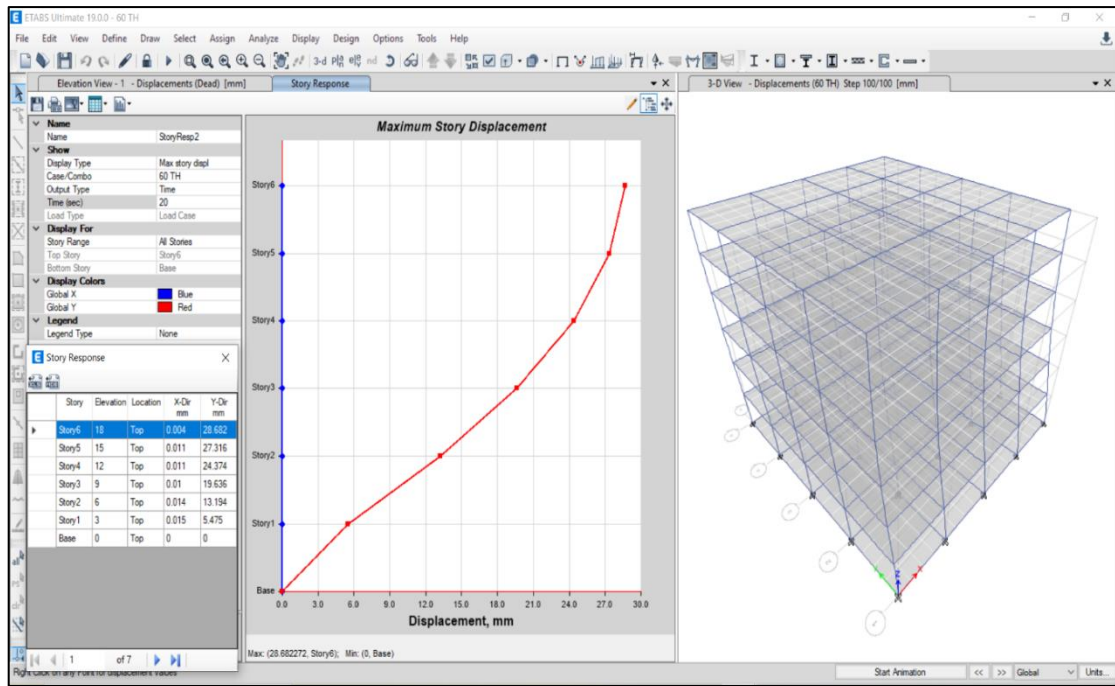


Figure 5.9 Storey displacement of model 1 at 60m standoff distance

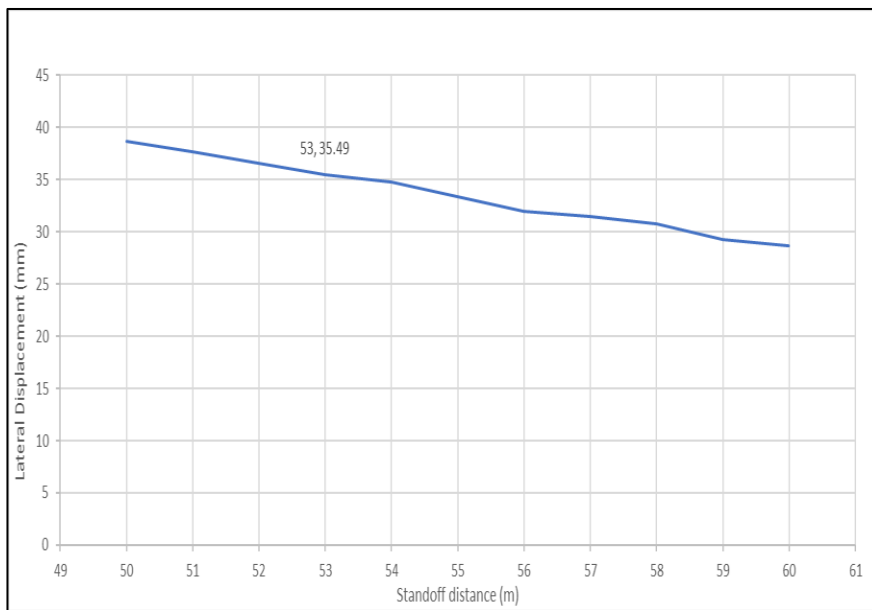


Figure 5.10 Lateral displacement vs. standoff distance

The storey drift of the building was also found out. Figure 5.11 shows the storey drift of model 1 subjected to dynamic blast at a standoff distance of 60m from the front face of the building. Storey drift was found to be maximum at the second storey level. The variation of storey drift with standoff distance between the range 50m to 60m is plotted as shown in figure 5.12. The storey drift decreased linearly with the increase in standoff distance. It was observed that the Story drift was less than the permissible limit in the range of standoff distance.

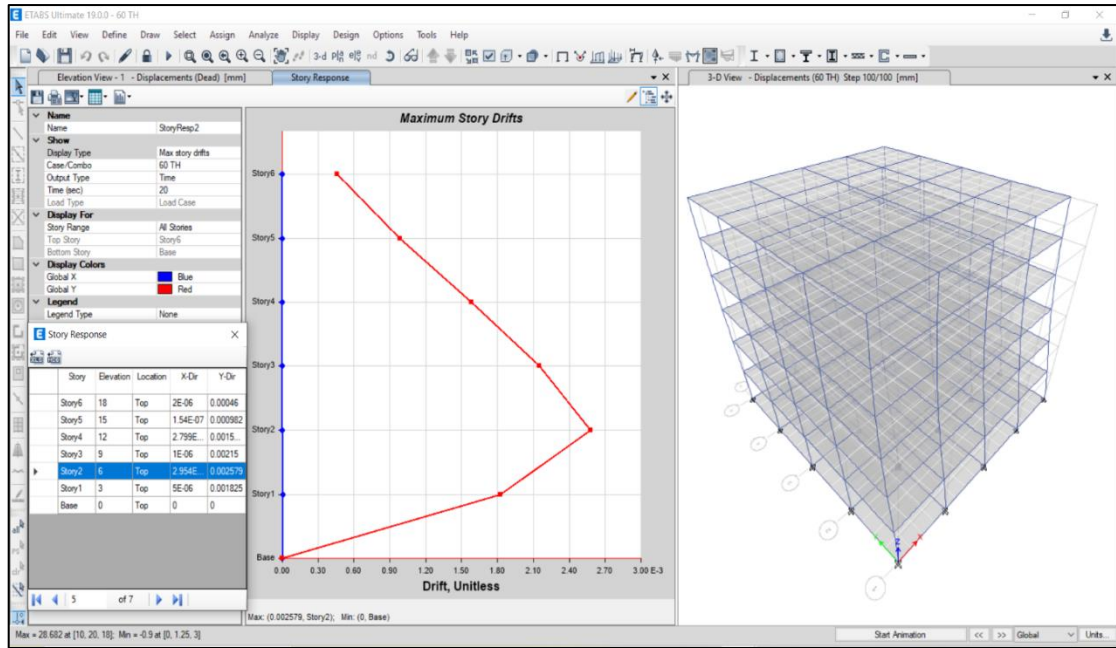


Figure 5.11 Storey drift of model 1 at 60m standoff distance

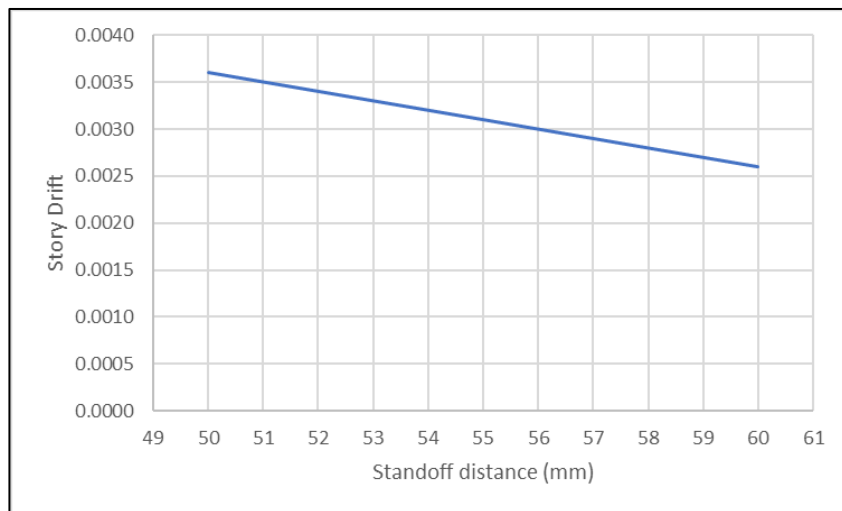


Figure 5.12 Storey drift versus standoff distance

Both lateral displacement and the storey drift are considered to find the exact safe standoff distance. Since storey drift is found to be within the permissible limit for the model 1 at the 50m to 60m range of the standoff distance as observed from both static and dynamic analysis, the lateral displacement can be taken as the only factor deciding the exact safe standoff distance. The results of the dynamic analysis shows that the lateral displacement was less than 36mm for the standoff distance more than 53m. Thus, the exact safe standoff distance of the bare frame model was observed to be 53m when subjected to blast load at the front face of the building.

❖ BLASTING AT THE CORNER SIDE

The dynamic analysis of model with blasting at the corner side of the building was considered. The time history analysis is carried out on the building in this case also to find the exact safe standoff distance. From the static analysis of model 1 with front face blast load, the safe standoff distance was found to be within the range 40m to 50m. Thus, dynamic analysis is done with blasting at standoff distance of range 40-50m. The maximum lateral displacement and the storey displacement for each 1m between the range is determined. Figure 5.13 shows the lateral displacement of model 1 when subject to dynamic blast load at 60m standoff distance. The lateral displacement increases with the storey level. The variation of lateral displacement with the standoff distance as observed after the dynamic analysis is shown in figure 5.14. The lateral displacement is found to decrease almost linearly with the standoff distance. From the graph, by comparing the lateral displacement with the permissible limit the exact safe standoff distance of model 1 was observed to be 45 m when subjected to blast load from the corner side.

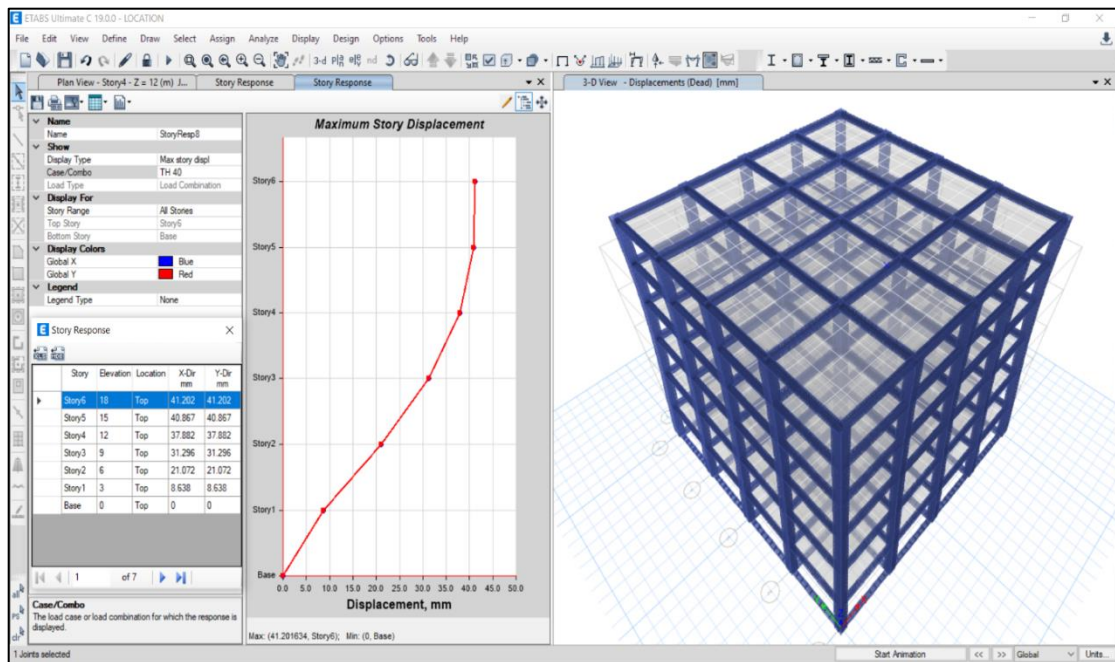


Figure 5.13 Storey displacement of model 1 at 40m standoff distance

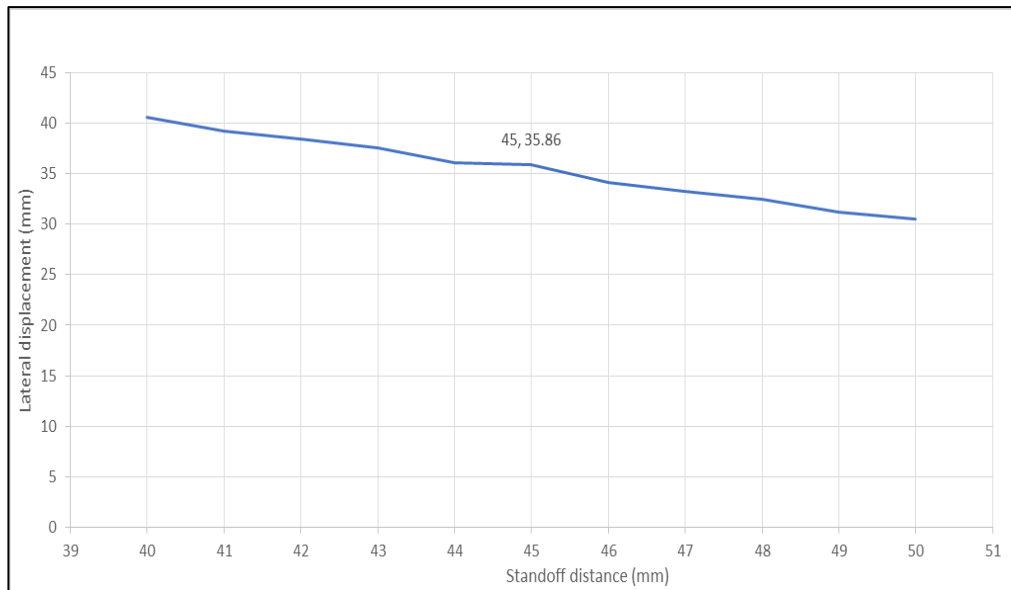


Figure 5.14 Lateral displacement vs Standoff distance

The storey drift of the building was also considered. Figure 5.15 shows the storey drift of model 1 subjected to dynamic blast at a standoff distance of 60m from the corner side of the building. Storey drift was found to be maximum at the second storey level. The variation of storey drift with standoff distance between the ranges 40m to 50m is plotted as shown in figure 5.16. The storey drift decreased linearly with the increase in standoff distance. It was observed that the Story drift was less than the permissible limit in the range of standoff distance.

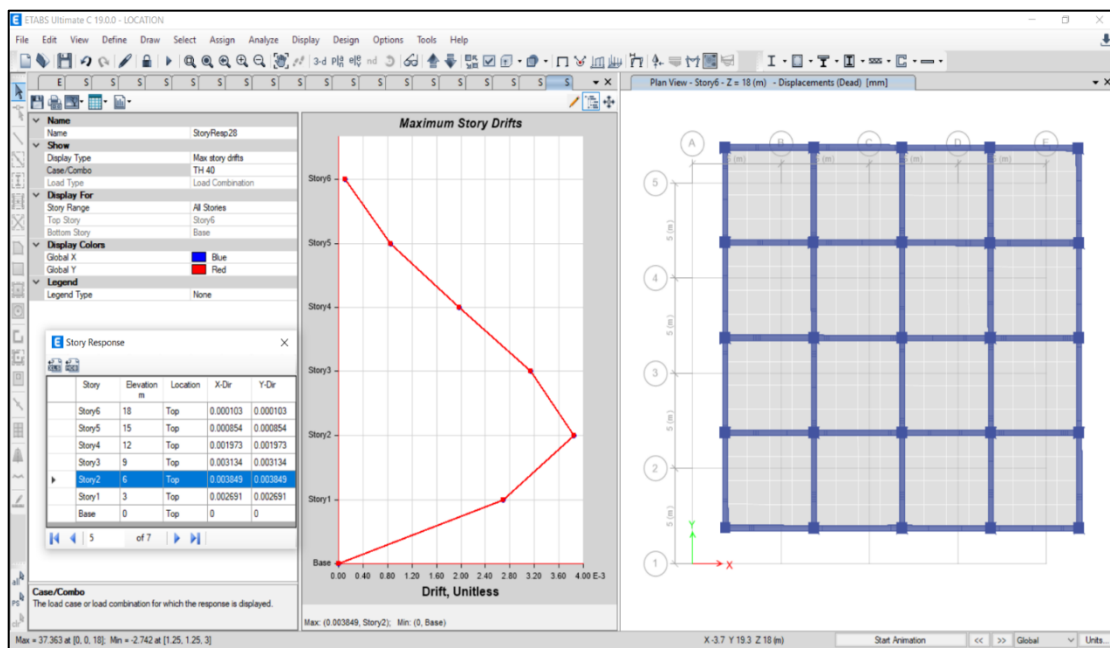


Figure 5.15 Storey drift of model 1 at 40m standoff distance

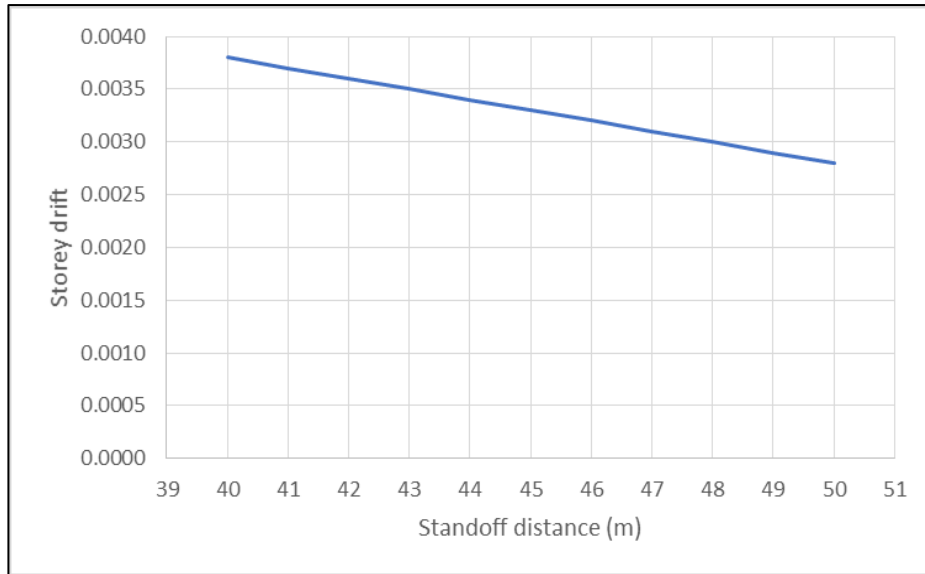


Figure 5.16 Storey drift versus standoff distance

Both lateral displacement and the storey drift are considered to find the exact safe standoff distance. Since storey drift is found to be within the permissible limit for the model 1 at the 40m to 50m range of the standoff distance as observed from both static and dynamic analysis, the lateral displacement can be taken as the only factor deciding the exact safe standoff distance. The results of the dynamic analysis shows that the lateral displacement was less than 36mm for the standoff distance more than 45m. Thus, the exact safe standoff distance of the bare frame model was observed to be 45m when subjected to blast load at the corner side of the building.

4.2.3 INFERENCE

The results of static and dynamic analysis of the model 1 subjected to blast load due to the explosion of 100 kg TNT at the front face and corner side are listed in table 4.1. The exact safe standoff distance of bare frame model is 53m and 45m when the blasting is at the front face and corner side of the building respectively. So it can be concluded that other models subjected to blasting at front face are analysed for standoff distance up to 53m. Similarly, for blasting at the corner side the models are need to be analysed for standoff distance up to 45m. By analysing the building responses, it was observed that the storey drift of the building is within the permissible limit for the range of standoff distance considered, irrespective of the blast location. Consequently, the lateral displacement is considered as the only factor deciding the safe standoff distance for the further analysis of the building models.

Table 5.1 Static and dynamic analysis results of bare frame model

Blast Location	SSD Range by Static Analysis (m)	Exact SSD by Dynamic Analysis (m)
Front face	50-60	53
Corner side	40-50	45

4.3 SAFE STANDOFF DISTANCE OF OTHER MODELS

The next step of the thesis work is to find the extent to which the safe standoff distance can be reduced by modifying the bare frame model i.e., model 1 to model 2, model 3, model 4 and model 5 as mentioned in chapter 3. For that the models are analysed dynamically by using time history analysis. The results obtained and their subsequent inferences are discussed below.

4.3.1 DYNAMIC ANALYSIS OF MODEL 2

The effect of addition of bracings on the blast response of the structure is analyzed by time history analysis. Three arrangements of V bracings and X bracings are considered as mentioned in chapter 3. The safe standoff distance is obtained and compared for each of the three cases and also with the bare frame model. The response of the building with X and V bracings when subjected to dynamic blast load due to blasting at the front face of the building are discussed in the following sections:

❖ MODEL 2 WITH V BRACINGS

The dynamic analysis is carried on model 2 with the three cases of V bracings. The storey displacement of the building without bracings and with the three cases of the bracings, when subjected to blasting at 53m standoff distance from the front face are shown in figure 5.17. In general, the graph shows that the storey displacement of braced building is less than that of the unbraced building. The graph of case 1 and case 2 overlaps, indicating similar behaviour under blast load. As mentioned earlier, the lateral displacement of the building considered is less than or equal to 36mm. The maximum storey displacement of case 1 was 33.11mm, case 2 was 33.29mm and case 3 was 31.27mm at a standoff distance of 53m. The storey displacement is reduced significantly for case 3, by around 13%. The introduction of bracings clearly improved

the performance of the building. The reduction in storey displacement indicates that the safe standoff distance of the building with V bracings is less than 53m. Thus, the time history analysis of model 2 is carried out for standoff distance less than 53m until the exact safe standoff distance is obtained for each case.

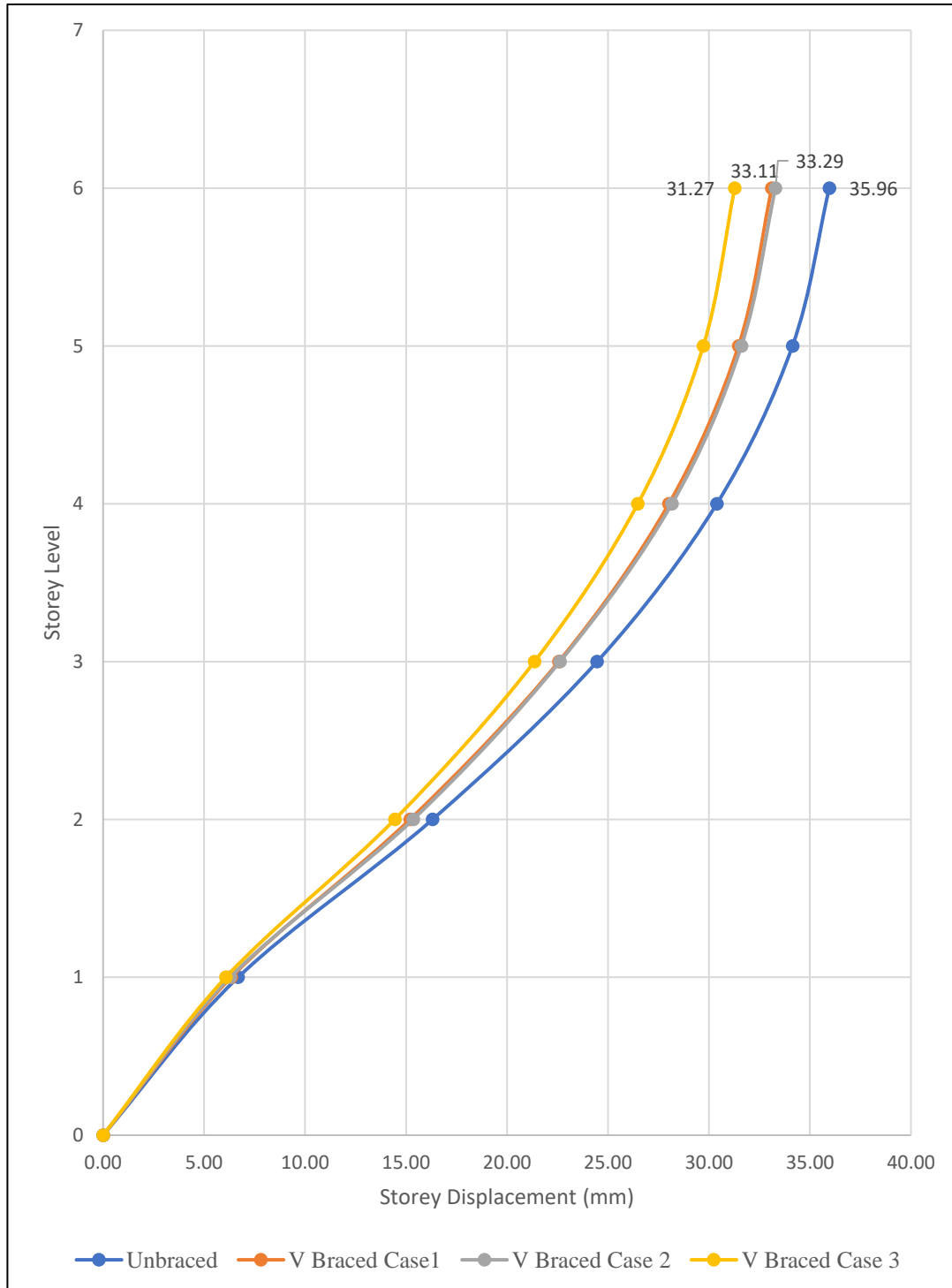


Figure 5.17 Variation of storey displacement for V braced model

Table 5.2 shows the variation of the safe standoff distance with the introduction of bracings. For case 1 and 2 which showed similar behaviour had the safe standoff distance reduced to 48m. The safe standoff distance was reduced to 45m for the case 3 of V bracings. It is thus evident that the providing V bracings in the building can reduce the safe standoff distance and make the building more blast resistant. It is found that the safe standoff distance may be reduced up to 45m when the V bracings are provided at all floor levels.

Table 5.2 Safe standoff distance of V braced model

Model		Safe Standoff Distance (m)
Unbraced		53.00
V braced	Case 1	48.00
	Case2	48.00
	Case 3	45.00

❖ MODEL 2 WITH X BRACINGS

The dynamic analysis is carried on model 2 with the three cases of X bracings. The storey displacement of the building without bracings and with the three cases of the bracings, when subjected to blasting at 53m standoff distance from the front face are shown in figure 5.18. In general, the graph shows that the storey displacement of braced building is less than that of the unbraced building. The graph of case 1 and case 2 overlaps, indicating similar behaviour under blast load. As mentioned earlier, the lateral displacement of the building considered is less than or equal to 36mm. The maximum storey displacement of case 1 was 32.68mm, case 2 was 32.89mm and case 3 was 30.49mm at a standoff distance of 53m. The storey displacement is reduced significantly for case 3, by around 15%. The introduction of bracings clearly improved the performance of the building. The reduction in storey displacement indicates that the safe standoff distance of the building with X bracings is less than 53m. Thus, the time history analysis of model 2 is carried out for standoff distance less than 53m until the exact safe standoff distance is obtained for each case.

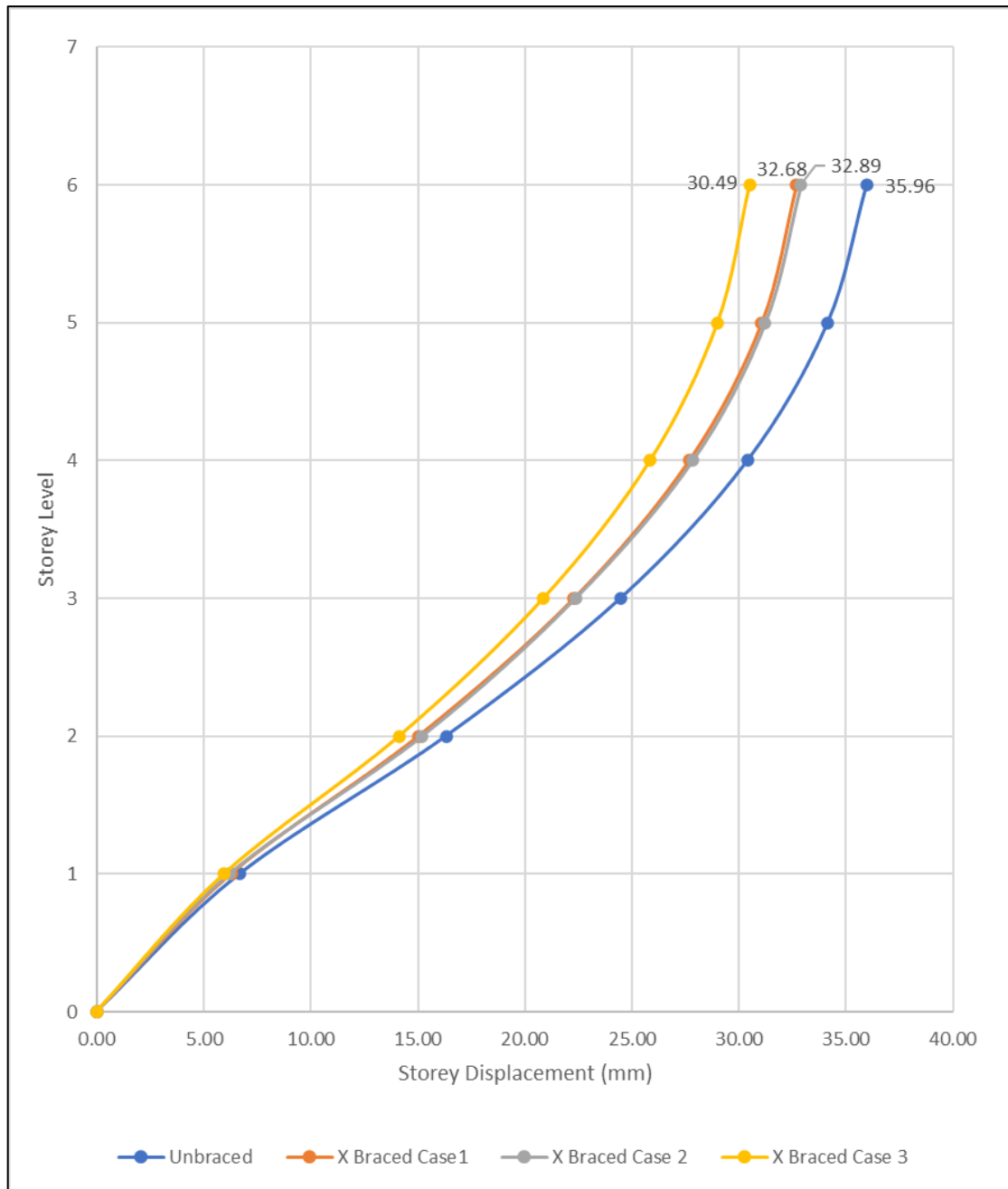


Figure 5.18 Variation of storey displacement for X braced model

Table 5.3 shows the variation of the safe standoff distance with the introduction of bracings. For case 1 and 2 which showed similar behaviour had the safe standoff distance reduced to 46m. The safe standoff distance was reduced to 40m for the case 3 of X bracings. It is thus evident that the providing X bracings in the building can reduce the safe standoff distance and make the building more blast resistant. It is found that the safe standoff distance may be reduced up to 40m when the bracings are provided at all floor levels.

Table 5.3 Safe standoff distance of X braced model

Model		Safe Standoff Distance (m)
Unbraced		53.00
X braced	Case 1	46.00
	Case2	46.00
	Case 3	40.00

❖ INFERENCE

The dynamic analysis of model 2 indicates that the provision of bracing is an efficient method to increase the blast resistance of the buildings. It was found that the model with bracings at all floor level on the outer periphery (case 3 of V and X bracing) had the lowest value of storey displacement in comparison to other cases of braced models. The safe standoff distance can be reduced up to 45m and 40m when V and X bracings are provided at all floor levels respectively. Accordingly providing X bracings is more effective in improving the performance of the building subjected to blasting at the front face compared to V bracings.

5.3.2 DYNAMIC ANALYSIS OF MODEL 3

The effect of column size of the model on the response of the building subjected to blasting at the front face is discussed in this section. In model 3, as mentioned in chapter 3, building with four sizes of column was considered for the dynamic analysis. The size of the column of the bare frame model is fixed as 600mmx600mm throughout the study. The response of the building with various column size when subjected to dynamic blast load due to blasting at the front face of the building are discussed in the following sections:

❖ EFFECT OF COLUMN SIZE

The dynamic analysis is carried on model 3 with the four cases of different column size. The effect of internal and external column size on the overall response of the building subjected to blast load was found out. The variation of lateral displacement with respect to storey height for models with different sizes of column sizes are shown in the figure 5.19. from the graph it is clear that the column size only has a very little effect on the response of the structure subjected to blast load. The maximum storey displacement of

case 1 was 35.96mm, case 2 was 35.02mm, case 3 was 34.34mm and case 4 was 33.26mm at a standoff distance of 53m. That is as column size increases the story displacement gets decreased. There was around 7.5% reduction in story displacement when the column size was increased to 750mmx750mm instead of 600mmx600mm.

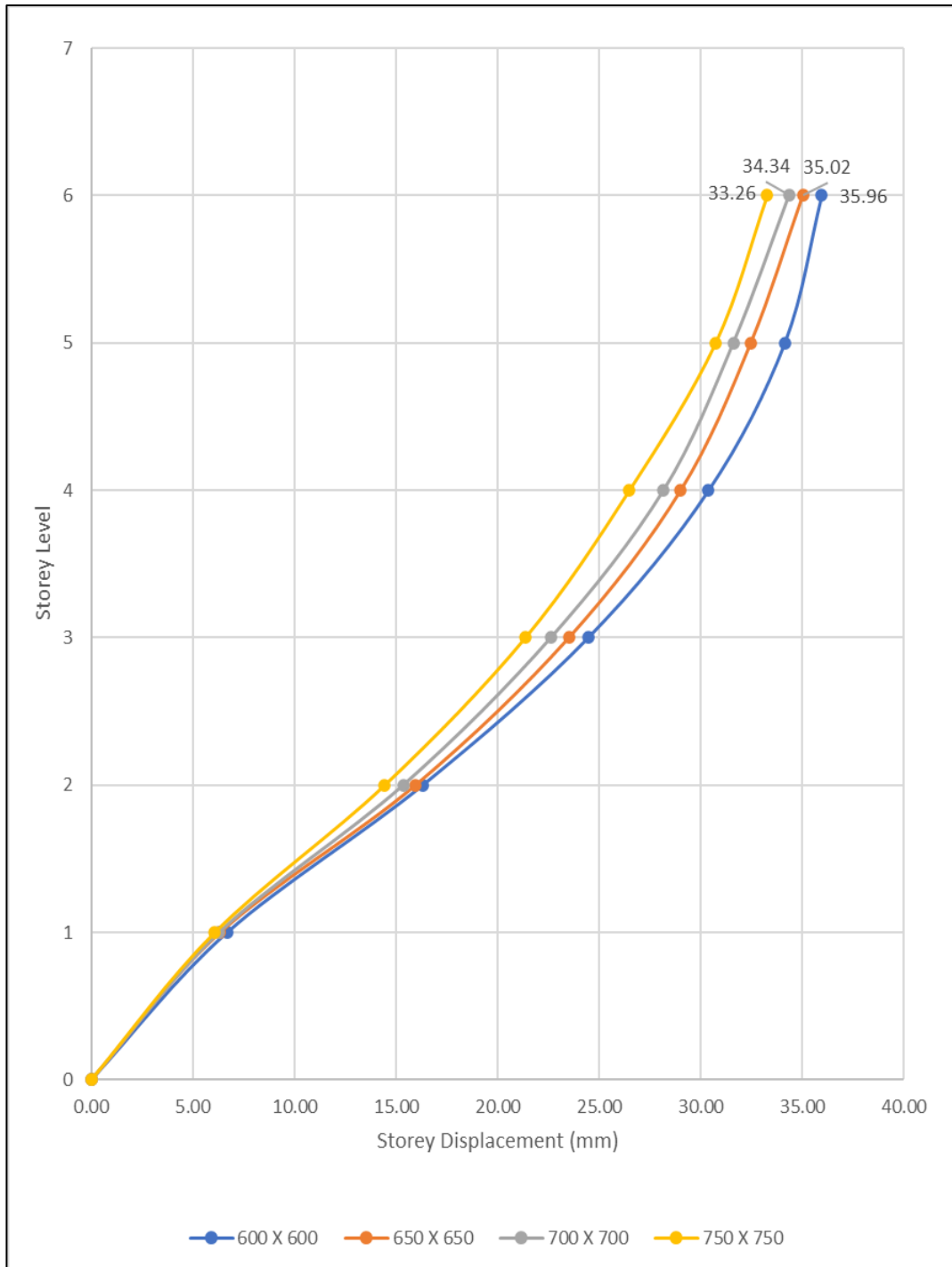


Figure 5.19 Variation of storey displacement with column size

The table 5.4 shows the variation of the safe standoff distance with the column size. The safe standoff distance was reduced by 2m when the column size increased from 600mmx600mm to 650mmx650mm and it was reduced by 3m when the column size increased to 700mmx700mm. It was found that by changing internal and external column size to 750mmx750mm instead of 600mmx600mm, the safe standoff distance can be reduced up to 48.5m.

Table 5.4 Safe standoff distance for different column size

Cases	Column Size (mm x mm)		Safe Standoff Distance (m)
	Internal	External	
Case 1	600x600	600x600	53.0
Case 2	650x650	650x650	51.0
Case 3	700x700	700x700	50.0
Case 4	750x750	750x750	48.5

❖ INFERENCE

The dynamic analysis of model 3 shows that the column size has very little effect on blast resistance of the structure. In all the cases, there is only a slight variation in the story displacement in comparison with bare frame model i.e., case 1. By providing a column size of 750mmx750mm, the lateral displacement can be reduced up to 33.26mm at a standoff distance of 53m. It was found that by providing a column size of 750mmx750mm, the safe standoff of distance can be reduced up to 48.5m. Although the structure with large sized column can perform well when subjected to blast load. But in practically it is uneconomical and it also affect the overall free space inside the building.

4.3.3 DYNAMIC ANALYSIS OF MODEL 4

The effect of shear wall on the response of the structure is discussed in this section. In general, providing shear wall is an effective method to increase the load carrying capacity of the building especially lateral loads. The load resisting capacity mainly depends on the type, size and location of the shear wall. The building is analysed by providing shear wall of specifications as mentioned in chapter 3. The response of the building with various arrangement of shear wall when subjected to dynamic blast load due to blasting at the front face of the building are discussed in the following sections:

❖ EFFECT OF SHEAR WALL

The dynamic analysis is carried on model 4 with the four cases of different arrangement of shear wall. Here the response of the model with four different arrangement of shear wall is compared with the bare frame model. The lateral displacement of the bare frame model and all the other cases due to the blast load is shown in figure 5.20. From the graph, the bare frame model and case 1 of model 4 shows almost similar behaviour with a maximum storey displacement of 35.96mm and 34.35mm respectively at a standoff distance of 53m i.e., the shear walls in case 1 of model 4 do not take much load. Case 2 and case 4 of model 4 shows almost similar behaviour with a maximum storey displacement of 7.64mm and 5.46mm respectively at a standoff distance of 53m, i.e., the shear walls in those cases are capable of resisting large intensity of blast load.

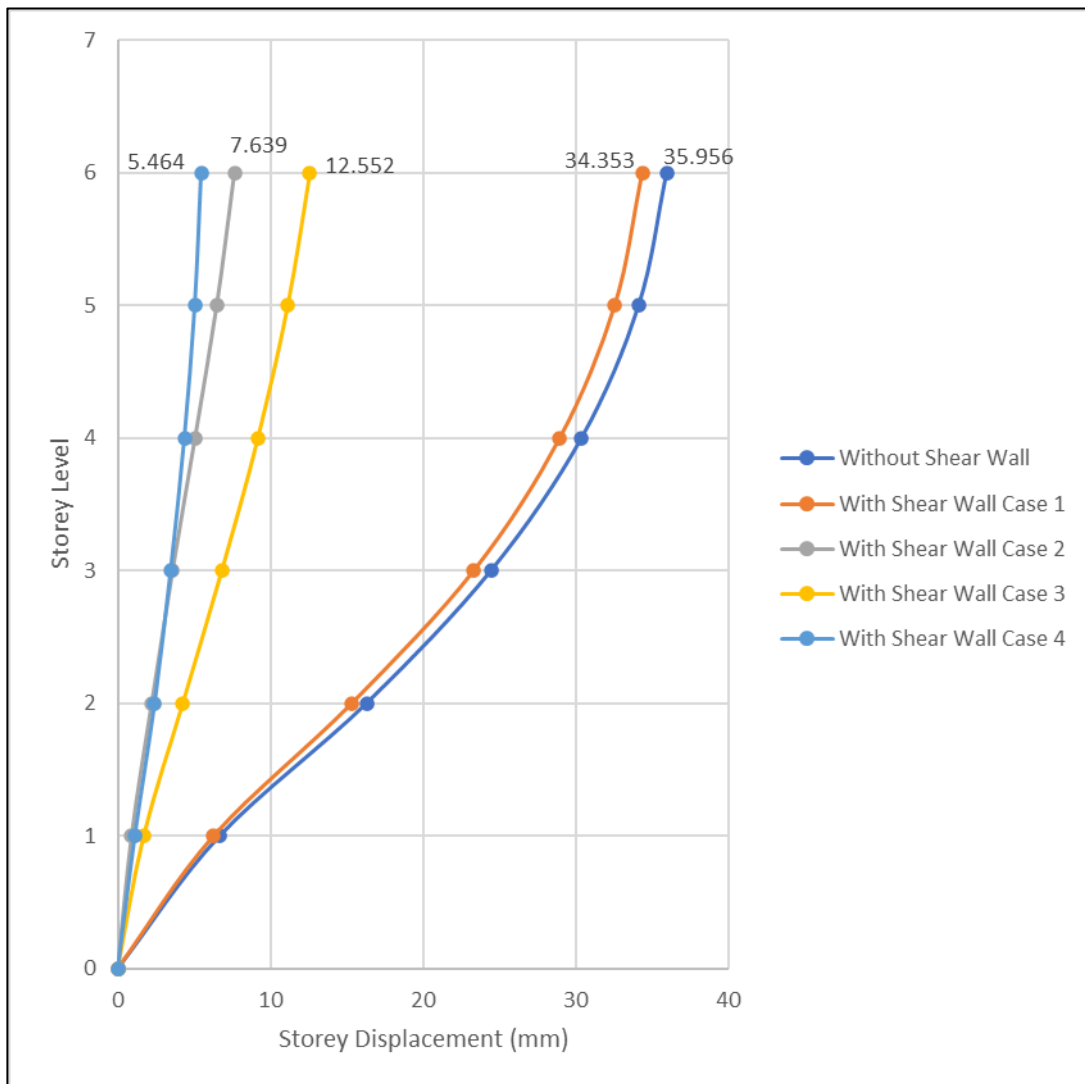


Figure 5.20 Variation of storey displacement with shear wall arrangements

Table 5.5 shows the influence of the shear wall arrangements on the safe standoff distance of the structure. There is only a small reduction in the safe standoff distance by 5m for case 1 of model 4 in comparison to the bare frame model. This is because the shear wall in the case 1 is subjected to out of the plane loading and shear wall have low resistance to out of the plane loading. However, there was substantial reduction in the safe standoff distance by 20m for case 3 of model 4 i.e., shear wall subjected to in plane loading. The safe standoff distance decreased by 16m for case 3 where the shear walls with a combination of in plane and out plane loading is provided. The safe standoff distance was reduced by maximum amount by 22m for case 4 of model 3 in which the shear wall is provided at the inner core of the structure. The safe standoff distance was reduced to 31m.

Table 5.5 Safe standoff distance for different shear wall arrangement

Model		Safe Standoff Distance (m)
Without Shear Wall		53.00
With Shear Wall	Case 1	48.00
	Case2	33.00
	Case 3	37.00
	Case 4	31.00

❖ INFERENCE

The use of shear wall as a blast load resisting element was found to be very effective. The performance of the building is improved to a great extent. By analysing the response of the structure subjected to blasting at the front face of the structure, it was clear that shear wall subjected to in plane loading is more effective than out of plane loading. Of all the cases, the case 4 of model 4 with shear wall at the inner core of the building exhibited best performance under blast load. The standoff distance was reduced up to 31m by providing shear wall at the core.

5.3.4 DYNAMIC ANALYSIS OF MODEL 5

The effect of plan configuration on the response of the structure is discussed in this section. The change in plan configuration changes the arrangement of structural elements of the building that resist the load due to blasting. Four different cases of building configuration as mentioned in chapter 3 are considered. The response of the building with various plan configuration when subjected to dynamic blast load due to blasting at the front face of the building are discussed in the following sections:

❖ EFFECT OF PLAN CONFIGURATION

Pressure time history analysis is carried on model 5 with the four cases of plan configuration. Here the response of the model with four different plan configuration is compared with the bare frame model. The lateral displacement of the bare frame model and model 5 due to the blast load is shown in figure 5.21. Examining the graph, it can be seen that the plan configuration does have some effect on the response of the structure subjected to blast load. The case 1 of model 4 had the maximum decrease in storey displacement compared to the bare frame model. This can be attributed to the fact that case 1 model 4 has the structural members arranged in such a way that it has the largest area of resistance in the direction of the load. Contrary to case 1, the storey displacement is increased for case 2 of model 5 compared to the bare frame model. This may be due to the arrangement of structural elements in case 2 is exactly opposite to that of case 1 thereby reducing the area of resistance in the direction of load. However, case 3 and case 4 of model 5 when subjected to blast load have similar response to that of the bare frame model. It is because in these cases, even though the structural arrangement is different, the effective area of resistance is almost the same in the direction of the load.

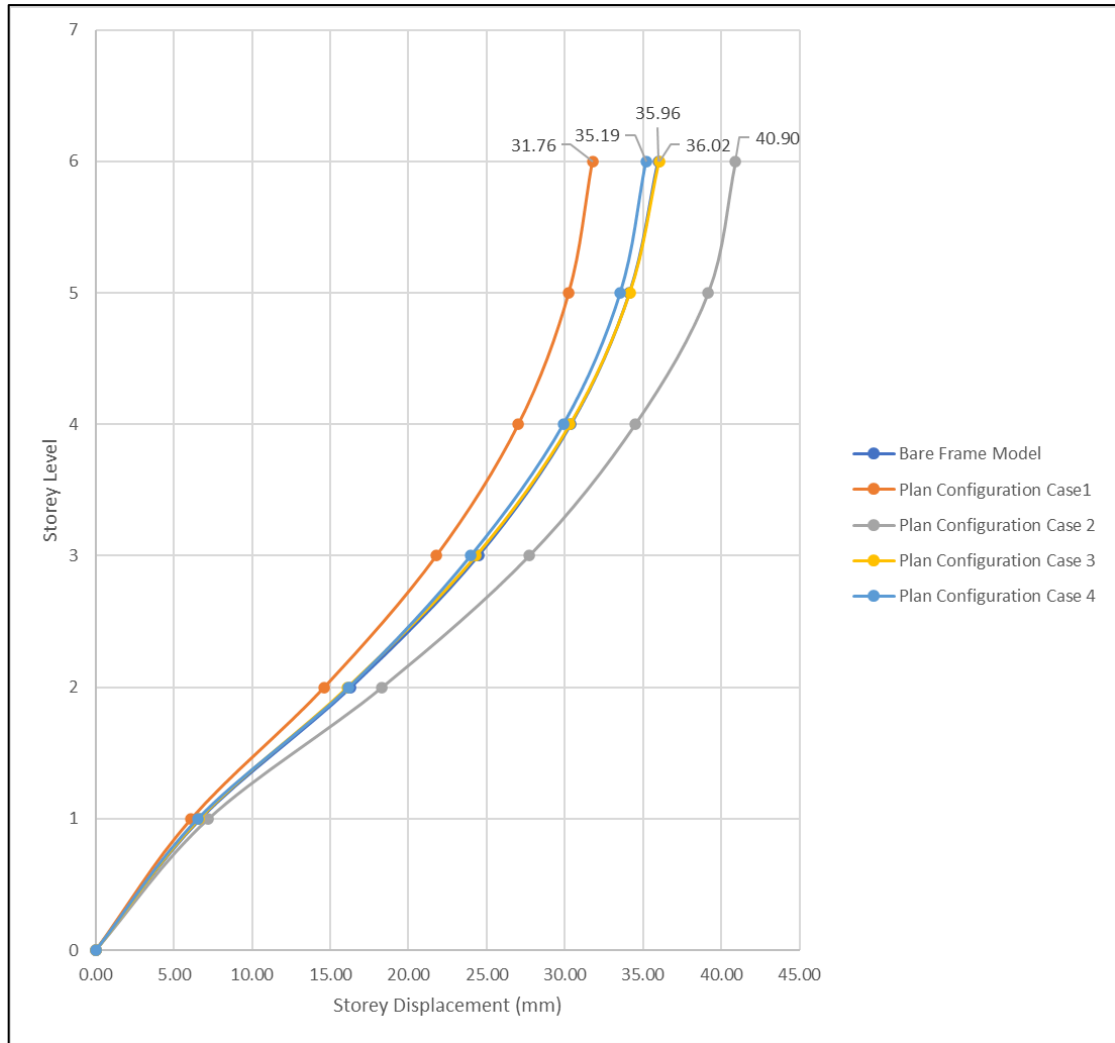


Figure 5.21 Variation of storey displacement for different plan configuration

Table 5.6 shows the effect of plan configuration on the safe standoff distance. The case 1 of the plan configuration causes the maximum reduction in the safe standoff distance up to 47m. however the safe standoff distance was increased to 57m for the case 2 of model 5. The case 3 and 4 had similar performance to that of the bare frame model with a safe standoff distance of 53m and 52m respectively.

Table 5.6 Safe standoff distance for different plan configuration

Model		Safe Standoff Distance (m)
Bare Frame Model		53.00
Plan Configuration	Case 1	47.00
	Case 2	57.00
	Case 3	53.00
	Case 4	52.00

❖ INFERENCE

The change in the plan configuration of the model only slightly varied the building response. By analysing the response of the model 5, the plan configuration with four bays of 6.25m in X direction and 4m in Y direction yielded better results compared to other cases. The safe standoff distance can be reduced up to 47m by changing the plan configuration.

5.3.5 EFFECT OF BLASTING AT THE CORNER SIDE

This section deals with the response of the structures when subjected to blasting at the corner side. All the model is dynamically analyzed by changing the location of blast from front face to corner side. The main object is to find the influence of location of blast on the safe standoff distance of various model considered. The loads are calculated and applied as mentioned in chapter 3. Each case of the different building models is subjected to blasting at the corner side at various standoff distance are analysed and the results are summarized in table 5.7. As mentioned earlier, the bare frame model (model 1) with corner side blasting was found to have a safe standoff distance of 45m. Here also, the safe standoff distance of the building subjected to blast load and the extent to which it can be reduced when modifications are made in the building are found.

The results of model 2 shows that when bracings are used in the building, the safe standoff distance is reduced significantly. The case 3 of both V and X bracings yielded the best results with the safe standoff distance reduced to 37m and 32m respectively. For Model 3, there is only a slight reduction in the standoff distance. The safe standoff distance was reduced by a maximum amount for case 4 of model 3. Thus, the column size has comparatively less effect on the safe standoff distance of the building with corner side loading.

The results of pressure time history analysis on model 4 at various standoff distance shows considerable reduction in the safe standoff distance. In case 1, the shear wall is capable of resisting large magnitude of load in the X direction than Y direction. But in case 2, the shear wall is capable of resisting large magnitude of load in the Y direction than X direction. In case 3, shear wall is capable of resisting load in both X and Y direction. The arrangement of shear wall in case 4 is in such a way that the wall is capable of resisting load in all direction. The safe standoff distance was reduced to 25m

for case 4 of shear wall. The plan configuration, however, only had minor influence on the safe standoff distance of the building. Case 1 and 2 showcased similar behaviour with an increase of standoff distance to 49m as the effective area is reduced in Y and X direction respectively. However for case 3 and 4, the safe standoff distance was nearly the same as that of the bare frame model, as the area of resistance remains the same.

Table 5.7 Safe standoff distance for different models with corner side blasting

Model		Safe Standoff Distance (m)
Model 1 (Bare Frame Model)		45
Model 2 (V Bracings)	Case 1	41
	Case 2	41
	Case 3	37
Model 2 (X Bracings)	Case 1	38
	Case 2	38
	Case 3	32
Model 3 (Column Size)	Case 1	45
	Case 2	43
	Case 3	42
	Case 4	40.5
Model 4 (Shear Wall)	Case 1	35
	Case 2	35
	Case 3	29
	Case 4	25
Model 5 (Plan Configuration)	Case 1	49
	Case 2	49
	Case 3	45
	Case 4	44

Fig 5.22 shows the variation in the storey displacement when various modifications are made in the building. Each of the graph represents the storey displacement of the best case of each building model when subjected to blasting at 45m standoff distance from the corner side. As discussed earlier, the permissible value of storey displacement of the building is 36mm. Here for the bare frame model, storey displacement was 35.83mm when blasting is at 45m from the corner side. From the graph, it is clear that changing the plan configuration is the least effective method and provision of shear wall is the most effective method in increasing the blast response of the structure. The change in the plan configuration is the least influential factor affecting the storey displacement. The case 3 and 4 of model 5 had storey displacement almost similar to the bare frame model. The column size of 750mm X 750mm is the most efficient of all

the cases, reducing the storey displacement by 6% when the blasting is at 45m from the corner side. The plan configuration also had very little impact on the storey displacement of the model. The results show that providing bracing is an effective method to increase the load resistant capacity of the building. Out of the two kinds of bracing provided, X bracings made the building more blast resistant compared to V bracings. For both bracings, the case 3 resulted in the least value of storey displacement. The storey displacement is reduced by 16% for V bracing and 25% for X bracing. Most efficient method to improve the blast resistance of the building is by providing shear wall. The case 4 of shear wall in which shear wall is provided at the core caused highest decline of about 36% in the storey displacement. This can be attributed to the increase in the load resisting capacity of the building when shear wall is provided in the inner core.

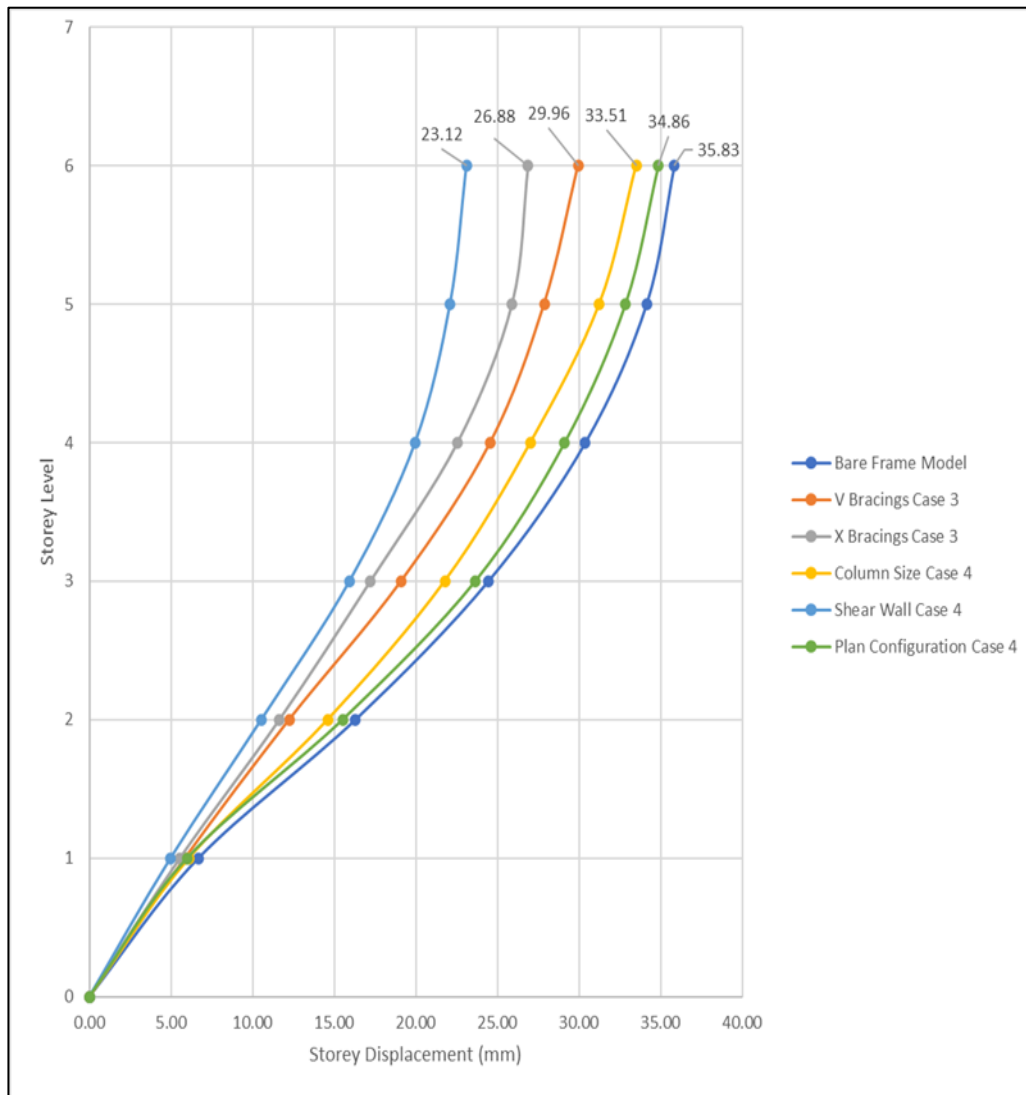


Figure 5.22 Variation of storey displacement for corner side blasting

❖ INFERENCE

After the analysis of the results of this section, it is confirmed that the location of blasting has a significant influence on the blast response of the structure. For bare frame model, safe standoff distance is obtained as 45m when the building was subjected to blasting at the corner side. Providing bracings is very effective in improving the performance of the building subjected to blasting at the corner side. It was found that the model with bracings at all floor level on the outer periphery (case 3 of V and X bracing) had the lowest value of storey displacement in comparison to other cases of braced models. From the graph it is clear that the column size only has a very little effect on the response of the structure subjected to blast load. The maximum storey displacement of case 4 was 33.51mm at a standoff distance of 45m and the safe standoff distance can be reduced up to 40.5m for the same. By analyzing the response, it was clear that shear wall subjected to in plane loading is more effective than out of plane loading. Of all the cases, due to similarity in arrangement case 1 and 2 showed similar responses and case 4 of shear wall exhibited best performance under blast load. The standoff distance was reduced up to 25m by providing shear wall at the core. The change in the plan configuration of the model only slightly varied the building response. By analysing the response, the case 4 of the model 5 yielded better results compared to other cases. The safe standoff distance can be reduced up to 44m by changing the plan configuration.

5.3.5 COMPARISON OF RESULTS

Results of dynamic analysis of building models with varying structural specifications subjected to blasting at the front face of the building is depicted in figure 5.23. Comparing to the bare frame model, the models with varying structural specifications was able to reduce the safe standoff distance to a certain extent. Analysing the results, it was observed that the most efficient method to increase the blast resistance of the building is to incorporate shear wall. Specifically in this study adding shear wall at the inner core yielded the best results and the safe standoff distance was reduced to 31m. Providing bracings also significantly improved the performance of the structure subjected to blast load. Out of the two bracings considered, the X bracings showcased better blast resistant behaviour than V bracings. When X bracings were used the safe standoff distance was reduced to 40m which is much preferable than V bracings where

the safe standoff distance was reduced to only 45m. Plan configuration only has very less effect on the safe standoff distance of the structure and it was reduced to 47m in this study. Column size was the least significant factor influencing the blast resistance of the structure. The safe standoff distance was reduced to 48.5m only with the modification of column size.

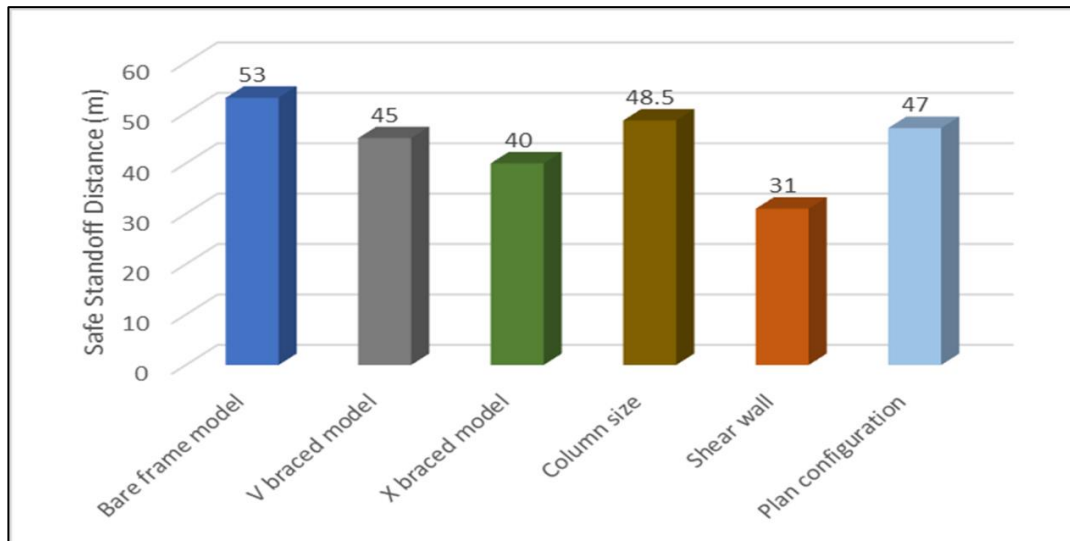


Figure 5.23 Comparison of results

Figure 5.24 shows a comparative result of the two cases of blast location i.e., location of blast at front face and location of blast at corner side. The comparison of the two cases is made in terms of the safe standoff distance. As shown in figure, there is a significant variation in the safe standoff distance when the location of blast was changed from front face to corner side. And in all the cases the building is safer when the location of blast is at the corner side. In either blast locations, the model with shear wall performed better than other models. Providing bracings is next best method to increase the blast resistance of the building.

Thus, after the dynamic analysis of various models subjected to blasting at the front face and corner side, it is clear that the blast resistance of the structure can be improved by addition of structural elements or by varying the structural configuration of the building. The safe standoff can therefore be reduced to some extent by incorporating such modifications in the structure.

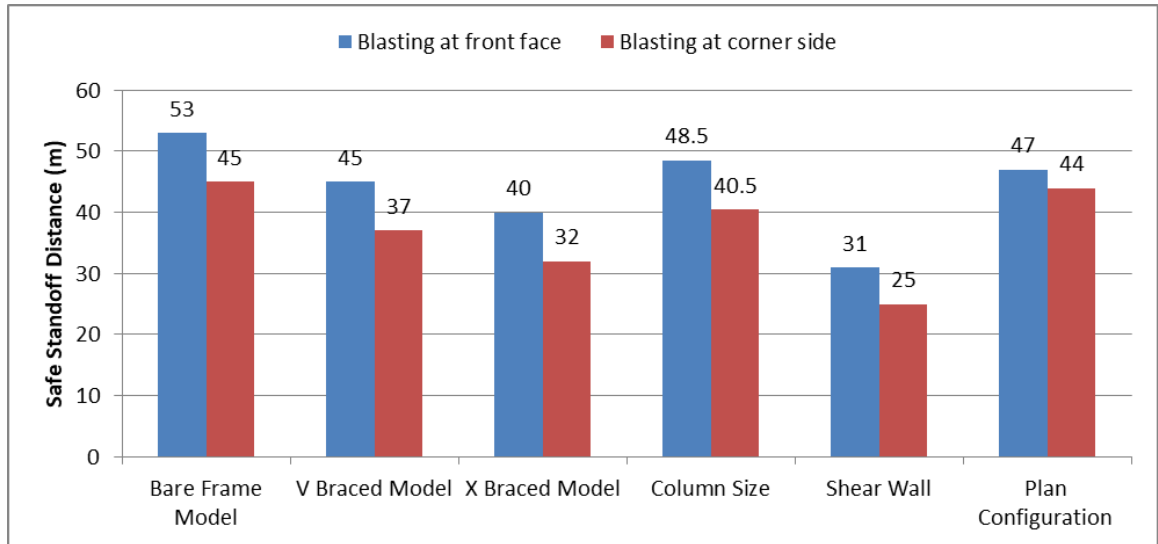


Figure 5.24 Comparison of results of two blast locations

CHAPTER 6

CONCLUSION

Blast resistant building is a concept that requires wide attention in the current scenario. Since the blast load is highly unpredictable and dynamic in nature, it very unlikely to design a fully blast resistant structure. However, studying the response of the structure when subjected to such dynamic blast load helps in understanding how a structure behaves under blast load. This can help in making a building a safe under blast load, although not completely blast resistant. In this study, static and dynamic analysis is carried on ETABS 2019 to analyze the response of a G+5 storeyed building subjected to blast effect due to the blasting of 100kg TNT explosive at various locations. The storey displacement and the storey drift are the two factors considered to evaluate the safety of the building. The behavior of the building under the blast load is expressed in terms of safe standoff distance. The blast response and safe standoff distance of a bare frame model is compared with other models with structural modifications. In this study various structural modifications are made on the building to improve the blast response of the structure, namely, addition of bracings, shear wall, change of column size and change in the plan configuration. Two different locations of blasting, i.e., at the front face and corner side are also considered for the study.

The static and dynamic analysis of the bare frame model subjected to blast load due to the explosion of 100 kg TNT at the front face and corner side was considered in the first stage of the study. The storey displacement and the storey drift were found to increase with the decrease of the standoff distance. After the static analysis of bare frame model the safe standoff distance was found to lie between 50 to 60m for front face loading and 40 to 50m for corner side loading. The pressure time history analysis is then carried out and the exact safe standoff distance of the bare frame was obtained as 53m and 45m when the blasting is at the front face and corner side of the building respectively.

In the next stage of the study, the blast resistance of the building is improved by adding structural elements or by changing the structural configuration of the building. Five different models and two blast locations are considered for the analysis. Dynamic analysis is carried out to find the safe standoff distance of each model. Comparing with

the bare frame model, the models with varying structural specifications were able to reduce the safe standoff distance to a certain extent. The location of blast was also had significant influence on the blast response. When the location of the blast is at the corner side of the building the safe standoff distance was lesser than that for the front face blasting. From the time history analysis of the modeled building, it was concluded that the provision of shear wall is the most efficient method to enhance the blast resistance of the building. The safe standoff distance was reduced to 31m and 25m when the shear wall is added at the core of the building subjected to blasting at the front face and corner side respectively. Adding bracings is another effective way to lessen the blast response of the structure; in particular, X bracing performed better than V bracing under blast loading. When X bracings were provided at all the floor levels, the safe standoff distance was reduced to 40m and 32m for blasting at the front face and corner side respectively. Variation of the column size and the plan configuration only had slight influence on the safe standoff distance of the structure.

At the end of the study, it was clear how reinforced concrete buildings behaved when subjected to blast loads. it was obvious that a building subjected to a blast load that is highly dynamic in nature can be made safe to some extent by the provision of specific structural elements or by modifying the structural design of the building. Thus, the safe standoff distance can be reduced considerably by modifying the building and thereby creating a moderately blast resistant building.

6.1 SCOPE FOR THE FUTURE STUDY

Present study is only confined to the analysis of G+5 storey building subjected to surface blasting of 100kg TNT. Study can be expanded to analyse the behaviour of structure with a greater number of storeys. It is possible to study the case of air blasting, blasting inside the structure, underground blasting instead of surface blasting. The results get through the work can make more accurate by doing the analysis with the help of better software's and the study can be extended to compare the responses of many other blast and building parameters.

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