Effect of Infill Wall on the Seismic Behaviour of Steel Framed Structures

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ABSTRACT

Most of the structural steel frame design is carried out with the assumption that the frame is bare and ignores the possible contribution of partition wall to the lateral load resisting system. Whilst presence of partition walls can strengthen the frame against lateral loads on the other hand it can make the frame stiff, which may reduce its ductility. This may be more important for the braced frames where bracings are designed to react freely to lateral loads and hence presence of partition walls may hinder this action. There is very limited work in the literature on this important subject. This thesis presents an experimental study on the behaviour of braced and moment frames with and without infill walls when subject to lateral loads. For this purpose, eight half-scale frame specimens were constructed by using two steel columns and one beam members. Equal leg angles were used as cross bracing for the braced frames only. To ascertain the impact of each specification, global drift proportions are taken into account for each test frame and are compared with each other. Test results showed that infill walls built of masonry blocks increase the rigidness of structural frames, resist lateral loads and limit frame deflections. Experimental test results reveal that the test frames having infilled walls had less damage than either the moment frame or braced frame without infill walls.This attributed to the increased stiffness of frames with infill walls.

Keywords: Lateral loads; Infill partition wall; Braced steel frame; Cross Bracing systems; Moment frame

ÖZ

Birçok yapısal çelik çerçeve tasarım esnasında çerçevenin içinin boş olduğu varsayımıyla tasarlanır ve bölme duvarların yatay yük taşıma sistemine olası etkileri gözönüne alınmaz. Bir yandan bölme duvarlar çelik çerçeveyi yatay yüklere karşı daha güçlü yaparken diğer yandan da daha sert yaptığından sünekliğini azaltabilir. Bölme duvarların çarprazlı çerçevelerde daha çok önemli olabilir, serbestçe hareket ettiği varsayılan çarprazların hareketine engel olabilir. Bu önemli konu üzerine yapılmış çok kısıtlı araştırmalar vardır. Bu tezde bölme duvarlı ve duvarsız, çarprazlı ve rijit çerçevelerin yatay yükler altında davranışını deneysel olarak incelenmiş ve sonuçları paylaşılmıştır. Bu gerekçeyle sekiz adet yarı ölçek çerçeve örneği iki adet çelik kolon ve bir adet çelik kiriş kullanılarak imal edilmiştir. Eşit kenar köşebent çarprazlarlı çerçevede kullanılmıştır. Her örneğin yatay yüklere karşı davranışının etkisini görmek için global yatay yer değiştirme oranları biribiriyle karşılaştırılmıştır. Sonuçlar bölme duvar kullanılan çerçevelerin daha rijit davrandığını ve yatay yüklere karşı çerçeve yatay yer değitirmelerini kısıtladığını göstermiştir. Deney sonuçları rijit veya çarprazlı bölme duvar içeren çerçevelerin yatay yük altında bölme duvarı olmayan çerçevelere göre daha az hasar gördüğünü göstermiştir. Bölme duvarın çelik çerçevelerin katılığını arttırdığından dolayı bu sonucun elde edildiği düşünülmektedir.

Keywords: Yatay yükler; Bölme duvarlar; Çarprazlı çelik çerçeveler; Çarpraz bağlantı sistemleri; Rijit çerçeveler

DEDICATION

I dedicate my study to my family and friends. A special feeling of gratitude to my late loving mother, Mabrooka Mohammed and loving father Muftah Lehweaj whose words of encouragement and precious advice ring in my ears. My sisters, who have never left my side and are very special to me. I also dedicate this study to my friends and extended family, who have supported me throughout the process of success. I will always remember and appreciate all that they have done. I also dedicate this work and special thanks to my brother Benmadi Milad, who has supported and encouraged me throughout the entire master program. It is with the help of family that I was able to make it here today.

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Successfully finishing this thesis has only been possible with the help of Allah whom has blissfully provided me with guidance. I thank Allah immensely for making it possible for me to complete this project.

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A	is the Cross-section area of the section
B	is the width of the section
H	is the depth of section
h_i	is the length of the web
I_y	is the moment of Inertia about Y-axis
I_z	is the moment of Inertia about Z-axis
L_o	is the initial lent of specimen before tensile test
L_f	is the final length of specimen after the tensile test
L_1	is legs of the angle
L_2	is legs of the angle
M	is the diameter of bolts
MIN-MF1	minor axis column moment frame without infill wall
MIN-MFIN2	minor axis column moment frame with infill wall
MIN-BR3	minor axis column braced frame without infill wall
MIN-BRIN4	minor axis column braced frame with infill wall
MAJ-MF5	major axis column moment frame without infill wall
MAJ-MFIN6	major axis column moment frame with infill wall
MAJ-BF7	major axis column braced frame without infill wall
MAJ-BFIN8	major axis column braced frame with infill wall
r	is the root radius of section
t_f	is the web thickness
W_{pIy}	is the elastic modulus of the section about Y-axis
W_{pIz}	is the elastic modulus of the section about Z-axis

Chapter 1

INTRODUCTION

1.1 Introduction

Annually, there are many casualties due to earthquakes in different parts of the world. Lateral stability is noted as one of the important problems of steel structures specifically in the regions with high seismic hazard. Steel structures often face difficulties in providing adequate strength and stiffness against such loading [1]. The Kobe earthquake in Japan and the Northridge earthquake in USA were two examples where there were damages to steel structures. Inadequate ductility level in steel members and connections and lateral instability were among the main reasons of structural failure. Therefore, during the last two decades there had been more intense research into the ductility levels of frames, particularly when they are subjected to lateral loads.

Jagadis and Doshi [2] stated that the major concern in the design of multistoried steel building is the provision of an efficient lateral load resisting system along with a good gravity load carrying system. They investigated the effect of using different types of bracing systems on the lateral load resisting capacity of multi storied steel buildings. For this purpose they uses Ground+15 stories steel building models with same configuration but different bracing systems, such as Single-Diagonal, X bracing, Double X bracing, K bracing, V bracing. A commercial software package

STAAD.Pro V8i was used for the analysis of steel buildings and different parameters were compared.

Since bracing system is not the only method for lateral load resistance this research was aimed at looking into the behaviour and ductility levels of half scale test frames that have cross bracing, moment frame, with and without infill walls.

1.2 Causes of Earthquakes

Earthquake is the ground motion or movement caused by a sudden release of energy in the earth’s crust called the lithosphere. It arises from stresses built up during tectonic processes of plate movements. The earth's outermost layer, the lithosphere, consists of several large and fairly stable slabs called plates, which have a depth of 80 km. Moving plates of the earth's surface account for most of the seismic activity of the world. The boundaries of the lithospheric plates coincide with the geographical zones, which experience frequent earthquakes. However, while the simple plate-tectonic theory is an important one for a general understanding of earthquakes, it does not explain all seismicity in detail, since devastating earthquakes sometimes occur away from these boundaries [3].

1.3 The Seismicity of the World

It is estimated that approximately 6,000 earthquakes are detected annually throughout the world. The number of minor earthquakes sensed by humans, without infrastructure damage or injury, is approximately 450. An estimate of 15 events yearly can be extremely damaging and deadly. Earthquakes in Northridge-California, 1994 and Kobe-Japan, 1995 were among the largest earthquakes that are particularly important due to extensive damages to steel framed buildings [3].

Northridge earthquake occurred in the morning of January 17 1994; where it was felt by around 10 million people in the Los Angeles region of southern California. Northridge earthquake had a magnitude of 6.7 and it was the worst earthquake in United States history causing an estimated loss of 20 billion in revenue, 55 people died as a result. Particularly infrastructure was significantly damaged [3]. Two amongst some of the deadliest earthquakes in the world are given in table 1.1 below.

Table 1.1: Some of the largest and most devastating earthquakes of the worlds

Year	Location	Fatalities	Magnitude
1994	Northridge, USA	55	6.7
1995	Kobe, Japan	5,500	7.2

Masayoshi Nakashima & Praween, 2003, [4] stated that, on January 17th, 1995, Kobe earthquake was the most destructive earthquake in Japanese history causing serious impact on infrastructural and economy with over 6,000 people were dead, 26,000 people were injured and more than 100,000 buildings were damaged beyond repair, making more than 300,000 people homeless.

For that reason, the science of civil engineering looks at designing earthquake resistant structures which is essential. It is important that structures are designed to resist earthquake forces, in order to reduce causalities. The science of Earthquake Engineering and Structural Design has improved tremendously and now structures are safer and can withstand earthquakes of reasonable magnitude.

1.4 Significance of this Research

Possible weaknesses of the strength and resistance of buildings are often against wind and earthquake loads, which are mainly lateral loads. Therefore, researchers are continuously working on the improvement of building resistance against such loads. Over the years there had been many researches on the behavior of various types of structural frames against lateral loads [5]. Another excellent approach towards strengthening and stiffening buildings for lateral loads is bracing systems [5]. However, there are very limited researches on the effect of infill walls on the behavior of frames [6].

Results from previous experiments reveal that these two components of building which are in-filled walls and the shear wall increase the stiffness and strength of the structure [7]. Majority of the research carried out were concentrated on the reinforced concrete walls rather than brick, aerated blocks, concrete blocks, etc, [8].

The most common structural framing methods for steel framed buildings are moment framed and braced framed. Often infill walls are also used for braced frames and there is very limited research on the effect of infill wall on the behavior of bracings and hence braced frames.

1.5 Objective of this Study

The main objective of this study is to investigate the contribution of BIMs block infill walls on the structural behavior of steel cross-bracings and the structural frame when it is subject to lateral loads. For this purpose an experimental test program is scheduled to investigate this behaviour. After careful consideration of the past literature on the subject the objective of this study was formed. Accordingly a typical

2-D frame model was selected, which represents a half model of a real size building frame. [9] ETABS (Computers and Structures, 2002) a commercial computer program for the analysis of structures, was used to analyse and design the frame. Then eight frame tests were carried out to find out the behaviour of these frames when subject to gravity and lateral loads.

1.6 Organization of the Thesis

The following outline gives the general content of the thesis and a rational arrangement by which the objectives stated in the previous section are fulfilled, investigated and presented.

Chapter 1 provides a general introduction on the Nature of earthquake ground motions in Kobe and Northbridge, and significance and hence the objective of study is also presented.

Chapter 2 provides an overview of engineering literature and research on moment frame and braced frames, and the behaviour of infill walls on steel frame.

Chapter 3 provides an overview of the design process through experimental data. This chapter will include calculations, models, analysis and design, test specimens and the load cases that this thesis will be dealing with.

In Chapter 4, there is discussion and analysis of the results obtained from experimental tests which will be shown in more detail. The influence of BIMs block infill walls, effect of major and minor axis stiffer and their relative impact on lateral load behavior are examined by the investigation of two models.

Lastly, conclusion and recommendations for future studies are presented in chapter 5.

Chapter 2

LITERATURE REVIEW

2.1 Introduction

Over the years, failure and collapse of buildings in earthquake zones has been studied by engineers. Therefore, there is need to find a feasible solution towards increasing the strength and ductility of buildings in such areas. This research was done so as to highlight the effect of infill walls on the behaviour of steel braced and moment frames, in particular, lateral displacement and ductility of such frames when subjected to horizontal wind or earthquake loading. For this reason, moment and braced 2-dimensional frames were used with and without infill walls and beams framing into columns flange and web to find the effect of infill wall on such frames.

2.2 Background

This chapter presents a literature review about resistance of various structural steel framing systems to earthquake loads and the contribution of infill walls to such systems in resisting earthquake loads.

With advances in science and technology, steel framed structures are becoming crucial in modern architecture. Some of the advantages of using steel framing for structures include: high strength, long spans, ductility, light weight, etc. Steel framed structure are considered as one of the most resistant materials in seismic conditions is believed to be a steel frame. Nevertheless over a 100 building steel structure frames in the Northridge that occurred in United States and Kobe earthquake in Japan suffered major damage and this drew attention from experts worldwide to study this phenomenon. Accordingly, the importance on research of the seismic performance of steel frame has significantly increased. A serious study was undertaken of a two-story steel frame model with ANSYS Software, and the analysis of the steel frame in different earthquake response. The result show that the steel frame structure has good seismic performance [10].

2.3 Lateral Load

According Kapse and Shinde [11], the lateral loads closely resemble live loads, who's main horizontal force component is acting on different members of structure. Analysis of lateral force effects due to wind and earthquake loads is usually done as an equivalent static load in a lot of forms of high rise buildings.

2.3.1 Earthquake Loads

Every year storms case many structural damages and loss of life. In some regions earthquakes occur more frequently. It is an instantaneous lateral movement in the ground under a structure that may shift in any direction and the horizontal components of this movement creates a wave action which usually transferred vertically to the structure. Earthquake loads have more consistence than the wind load. The stiffness, mass of the structure, and the motion of the earth surface causes changes in the magnitude of an earthquake because of seismic forces. Modifying location of building, importance factor, type of soil, and achieving good construction practices may help in resisting these lateral forces [11].

2.3.2 Wind Loads

Wind loads are the most common lateral loads. Magnitude of force is directly proportional with the overall height and shape of the structure and it also acts externally but creates internal pressure or suction. The design of the structure should be in such a way that the effective surface area subjected to wind must effectively be minimized. Wind generates positive pressure on the windward side and a negative pressure on the leeward side when there is resistance from the building. Factors that affect the wind load include the geographical conditions, height, surface area of exposure, nearby structures, building shape and size, winds direction, wind velocity and pressures associated with architectural design. It should be noted that the wind loads, as well as the pressure applied on the wall and roof elements, should be static and uniform [11].

2.3.3 Story Drift

Story drift or lateral drift is the horizontal sway of two adjacent stories of a building due to lateral forces. It is obtained by dividing the relative difference of design displacement between the top and bottom of a story, by the story height. IBC sets the maximum drift for normal buildings at between 0.7% and 2.5% of story height. Drift may be caused by both flexural and shear, because of the axial deformations of a column. In low rise buildings the addition of lateral forces is not very important for the shear strength of the building. However in high rise buildings the higher axial forces and deformations in the columns cause bending of structural members and high lateral displacements [11].

2.3.4 Story Displacement

The floor displacement profile is at maximum with the maximum story drift ratio depending upon the height, the time period, and the column-to-beam strength ratio. It is measured in terms of mean coefficient of variation. The parameters under which displacement is studied are sections and variations in reinforcement. This term is proportional with the mechanism of plastic hinges formation in structural members [11].

2.4 Structural Design

The determination of the structural design of a high rise building would ideally include the extensive selection and arrangement of the structural elements to efficiently counteract the different combinations of gravity and horizontal loading. Factors that have to be considered in determining the structural form comprise of the internal planning, the material, method of construction, the nature and magnitude of horizontal loading [12].

Use of steel frame has been extensive in the United States for mid- to high-rise structures. Majority of systems constructed before 1994 included steel moment resisting frames that provided lateral resistance during an earthquake. Northridge Earthquake of 1994 and Hyogoken–Nanbu Kobe Earthquake led to unanticipated damage to most of these systems because of fracture of welded beam-to- column connections that led to undesirable large lateral displacements [13].

2.5 Moment Frames

The principal intent of building code seismic provisions is to provide buildings with the capacity to support harsh ground movements and prevent collapse, nevertheless with just a little structural damage. Various structural configurations, structural systems and materials are used to accomplish this purpose. Moment resisting steel frames (MRSF) are one of the popular structural systems due to their flexibility for space usage and ductility capacity [14].

FEMA-310 (1998) determined [15], the flexural strength shown by moment frames causes improvement in resistance horizontal forces. Additionally continuity of beam and column elements contributes to resistance to loading. During severe earths movement, a structural system that possess appropriate dimensions/ specifications energizes plastic hinges which accommodate the implied force there by equipper the overall system to tolerate the displacement. The serial loads aren't always anticipated during design and calculation. Present day moment frames, the locations of maximum seismic moment are at the ends of beams and columns and this design allows inelastic characteristics that are related to plastic hinging when considered in a successive manner of layers and reversible loads. "Special moment frames" whose details are configured to ductile sensitivity. Frames that do not have particular seismic capacity extract support from the redundant strength incorporate in the system. For ordinary moment frames, a sudden brittle mechanism will cause failure, for example, shear failure in concrete members. The fundamental requirements for all ductile moment frames are that:

- They have sufficient strength to resist seismic demands
- They have sufficient stiffness to limit interstory drift,
- Beam-column joints have the ductility to sustain the rotations they are subjected to,
- Elements can form plastic hinges,
- Beams will develop hinges before the columns at locations distributed throughout the structure (the strong column/weak beam concept).

Moment resisting frames (MRFs) have very good performance when strength and ductility are involved. However, for taller buildings, their relatively high flexibility makes it cost effective to design for the necessary stiffness to meet drift requirements [16].

2.6 Braced Frames

According, Tafheem & Khusru [17] advocated that Steel braced frame is one of the structural systems used to resist lateral loads in multistory buildings. Among the advantages of steel bracing are economical, easy to erect, occupies less space and has flexibility to design to satisfy the required strength and stiffness. Braced frames are commonly meant to resist lateral loads but braces can interfere with architectural features. The steel braces commonly installed vertically aligned spans. This system allows obtaining a large increase of stiffness with a minimal added weight. Therefore, it is very effective for existing structure for which the poor lateral stiffness is the main problem. To increase stiffness and stability of the structure generally bracings are used to resist lateral loading and displacement, significantly.

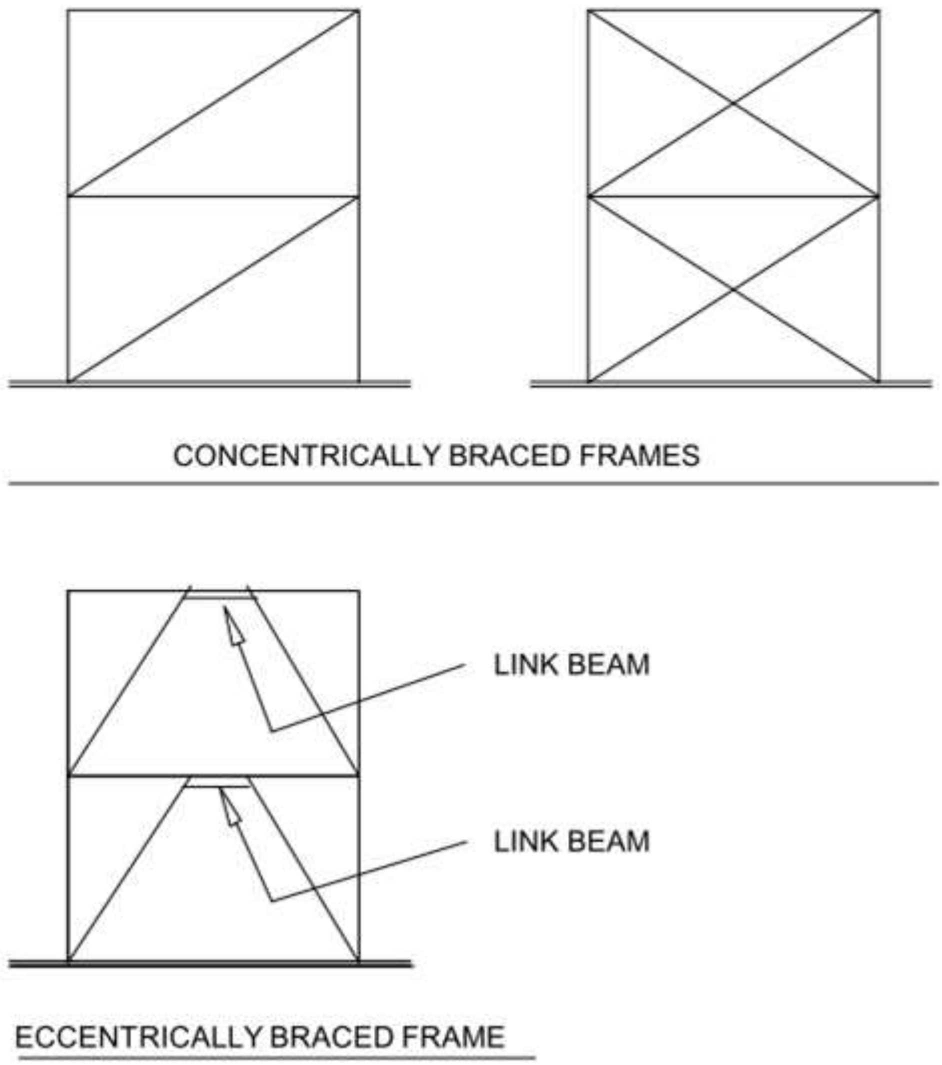


Figure 2.1: Braced frames [6]

Braced frames maybe utilized for seismic reconstruction of existing steel, composite steel–concrete and reinforced concrete building structures [9,10]. The most commonly utilized systems include concentrically-braced frames (CBFs), eccentrically-braced frames (EBFs) and the knee-brace frames (KBFs) Fig. 2.1. Common compositions for CBFs encompass V and inverted-V bracings, K, X and diagonal bracings [19]. However, V bracings are not recommended for seismic retrofitting due to the possibility of destruction in the beam mid-span. When subjected to horizontal forces, the compressed braces may buckle, in turn not having enough load-bearing capacity. Contrarily, the force in the tension braces increases constantly approaching yield strength and eventually strain-hardening. The net result is an unbalanced force concentrated at the brace-to-beam connection [20]. The effects on the beam, e.g. additional bending and shear, should be added to those due to gravity loads [21]. Alternatively, the unbalanced force in the beams may be eliminated through ad hoc bracing configurations, such as macro-bracings, e.g. two, three stores X-bracings or V-bracings with a zipper columns [22].

2.6.1 Concentrically Braced Frames

Concentric bracing is known to increase lateral drift and reduce the frame stiffness [17]. Although increase in the stiffness may attract a larger inertia force due to earthquake. Furthermore, while the bracings decrease the bending moments and shear forces in columns, they tend to increase the axial compression in the columns to which they are connected.

Concentrically braced structures continue to be used as lateral load resisting systems. As new systems and design approaches are developed there is anticipation of increase in their use. Structural damages during the past earthquakes suggest that braced systems may perform poorly due to their limited ductility and energy dissipation. Even though there has been an increase in the use of braced frame systems, failure of the connection between the braces and the frame can cause unbalanced behavior of the brace in tension and compression, [23].

Concentrically braced frames (CBF) withstand displacement and lateral forces mostly through the stiffness and axial strength of the brace members [24]. In CBF’s, the neutral axis of the columns, beams and braces relate to common point known as a work point. CBF’s have different arrangements, some of which are shown in Figure 2.2.

There are two types of concentrically braced frame systems: Ordinary Concentric Braced Frames (OCBFs) and Special Concentric Braced Frames (SCBFs).

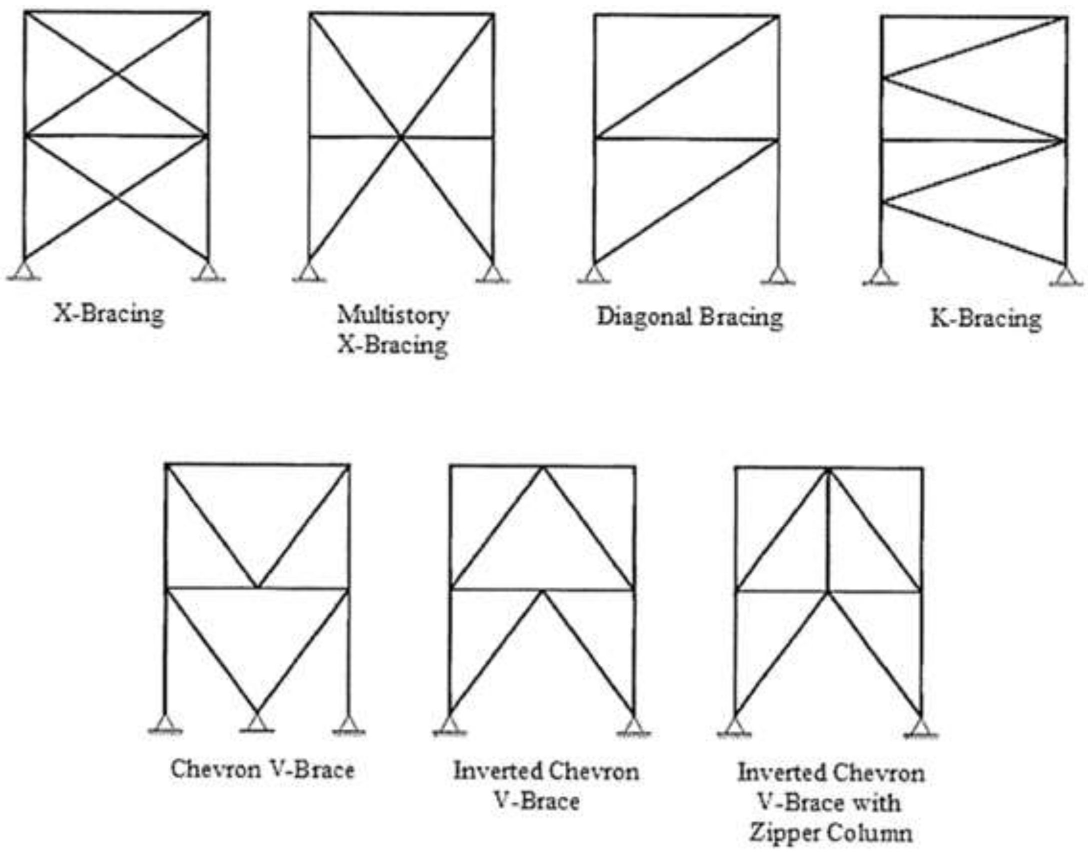


Figure 2.2: Typical braced frame configurations.

2.6.2 Ordinary Concentric Braced Frames

Ordinary Concentric Braced Frames (OCBFs) are expected to give little inelastic deformation and are designed using higher seismic forces to counter their limited ductility. They are suitable for small buildings in areas without much seismicity, due to their simple design and detailing requirements. ASCE Minimum Design Loads for Buildings and Other structures (ASCE 7-02) specifies the seismic design load requirements and height limitations for OCBF which is summarized in Table 2.1.

Table 2.1: Design coefficients, factors and limitations for OCBF Systems

Response Modification Coefficient	System Limitations and Building Height Limitation (Feet) By Seismic Design Category				
R	A or B	C	D	E	F
3,25	NL	NL	35	35	NP

*NL = Not Limited and NP = Not Permitted (from ASCE7-05).

2.6.3 Special Concentric Braced Frames

Special Concentric Braced Frames (SCBFs) have the same properties of OCBFs but have increased detailing specifications to increase the ductility of both brace and the gusset plate. The detailing specifications increase the system ductility and energy dissipation ability, allowing the system to be designed using a higher response Modification Coefficient than that of an OCBF. Table 2.2 sums up the seismic design load specifications and height limitation for SCFMs.

SCBF system can be used for areas of high seismicity where OCBF are not permitted (Table 2.2).

Table 2.2: Design coefficients, factors and limitations for SCFMs Systems

Response Modification Coefficient	System Limitations and Building Height Limitation (Feet) By Seismic Design Category				
R		C	D	E	F
6	NL	NL	160	160	100

*NL = Not Limited and NP = Not Permitted (from ASCE7-05).

2.6.4 K-Bracing

FEMA-310 [15] for K-brace setup, column between floor levels are intersected by diagonal braces (Fig. 2.3). Immediately after the compressive brace buckles, the column will be under load with the horizontal part of the bordering tension brace.

Large mid height requires which might compromise the balance of the column and vertical reinforcement of the building will be induced.

Generally columns are not modelled to fight against this force. The danger to the vertical support modelling creates unwanted bracing setting.

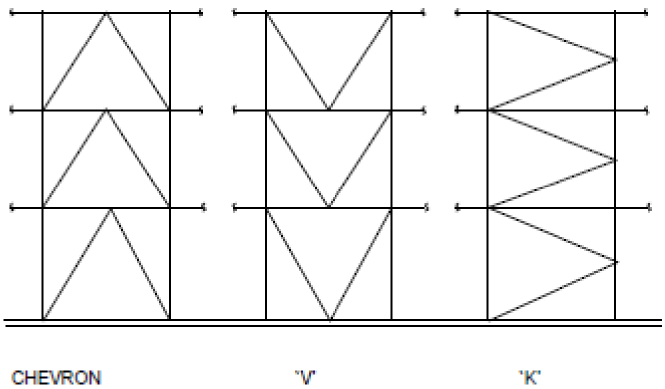


Figure 2.3: Bracing Types

2.6.5 Chevron Bracing

Figure 2.3 shown the chevron braces and are one of the most economical in terms of fabrication and erection costs versus structural effectiveness, and the most accommodating in terms of flexibility for locating door and window openings was derived by Bubela, 2000 [25].

2.6.6 Eccentrically Braced Frames

Eccentrically braced frames are intentionally located away from joints and their connections cause shear and flexure on the system. It is modelled to force a concentration of inelastic movement at expected position which inturn regulate the nature of the framework. Nowadays eccentrically braced frames are modelled with stern control on the system dimensions and important out-of-plane bracing at the links to make sure the frame operates like planned. The eccentrically braced frame is almost a current form of frame that is distinguishable by a diagonal with one end appropriately offset the joints (Fig. 2.4). Just like any other braced frame, utility of the diagonal is to give stiffness and pass on lateral forces from lower levels up to the top. The uncommon aspect of eccentrically braced frames is an offset zone in the beam, called the link. The link is uniquely comprehensive for regulated yielding. This detailing is conditional to very specific demands, so a usual ordinary braced frame with an offset zone that looks like a link may not necessarily behave like an eccentrically braced frame.

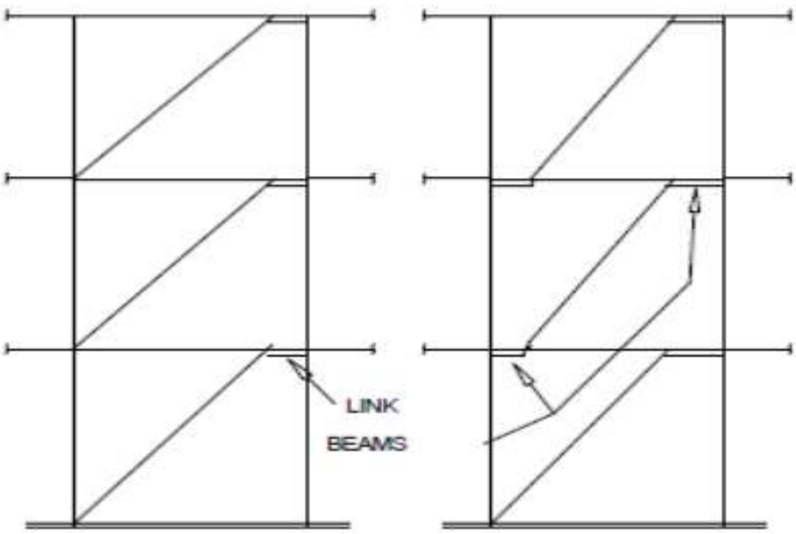


Figure 2.4: Eccentrically Braced Frames

An eccentrically braced frame has important qualities listed below:

1. At one end of each brace there is a connection beam.
2. Connecting length of the beam is confined to domination on the shear deformations and rotations due to flexural yielding at the ends of the connections.
3. The brace and the connections are modelled to establish forces persistent with the strength of the link.

4. Lateral bracing is given to avoid out-of-plane beam rearrangement that would disturb the expected result.

Frequently when frames comprise the whole lateral force opposing modelling eccentrically braced frames are utilized. A few high rise buildings, eccentrically braced frames are added as stiffening members to assist regulate drift in moment resisting steel frames.

2.7 Seismic Performance of Concentrically Braced Frames (CBFs)

Lateral load resisting systems are needed in most buildings to withstand horizontal forces, static or dynamic, due to wind pressure or seismic accelerations [25]. For steel structures there are two very popular systems, moment resisting frames (MRFs) and CBFs are, each with their own associated costs and benefits.

MRFs work better in terms of strength and ductility. However, for high rise structures, their relatively high flexibility makes it costly to design for the necessary stiffness to reach drift requirements have been proposed by Mazzolani, F.M et al [16]. Adding on, moment connections are more expensive than simple connections in braced frames because of the particular detailing needed for high ductility as well as the substantial amount of welding involved. Welded connections, especially those done on-site, are likely to fracture because of irregularities in the welds. Due to their exceptional energy absorbing qualities MRFs were the framing system selected in the United States in high-risk seismic zones. The overall damage to welded MRF connections during the 1994 Northridge earthquake in California seriously destroyed this belief, which resulted in an extensive research and development project to fix the design, detailing, fabrication and inspection of these connections (SAC, 1995).

When compared to other systems, CBFs have very good strength and stiffness and effortlessly reach rigidity requirements, however, have less ductility than MRFs [25]. CBFs are widely utilized in low-medium- and high-rise construction and can be designed in a number of bracing configurations, including cross (X-) bracing, diagonal bracing, K-bracing, V-bracing and chevron (inverted V-) bracing (Figure 2.5).

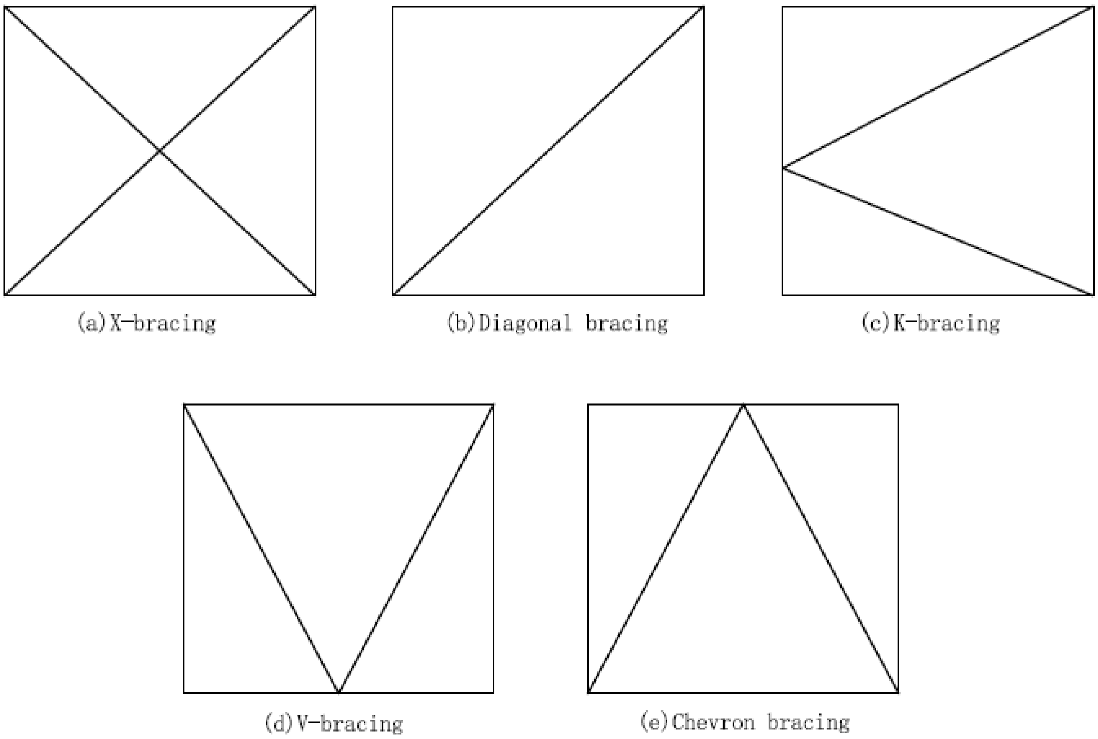


Figure 2.5: Typical concentric bracing configurations

2.8 Frame with Infill Wall

Infill wall means partition wall that consists of a kind of masonry. Infill walls have different varieties, such as brick masonry infill wall, concrete infill wall, timber framed infill walls, light steel framed infill walls. The infill material is intended not to take part in transmission of any overcoming loads to the structural system. But the research and analysis results showed that the use of masonry infill walls constructed in between the adjacent columns of reinforced concrete framed structures are very important in the damage of component parts and deterioration of structure during harsh earthquakes. This is due to the use of heavily dense masonry material. A lot of site evidence reveals that the infill masonry having low unit weight can help to extensively lessen the damage of a reinforced concrete structure. Due to infill walls between columns and beams substantially higher shear force in columns are created due to horizontal component of the force in equivalent diagonal strut of masonry. That resulted in failure of columns in infill frames in previous earthquakes [11]. However, during construction of the building frame, infill wall is not recognized as a structural element. Due to this, stiffness of infill wall is not predicted and not recognized in design of structure.

2.9 Structural System of Masonry Infill Wall

Two varying methods are involved when constructing masonry infilled concrete frames dependent on local construction site. The initial method, masonry infill is a part of the structural system and they are believed to brace the frame against horizontal loading. The other method, the frame is modelled to support the overall vertical and horizontal loading. Furthermore, masonry infill walls are free to bypass load being transferred to them. In areas where earthquakes are likely to occur for example Turkey, hollow masonry infill walls are enlisted as non-structural elements. Consideration is not put at design stage. Despite that they are designed to be seperate from the load carrying system, commonly noticed diagonal cracking on masonry infill walls reveal that the method is inconclusive [26].

2.10 Behavior of Masonry Infilled Steel Frames

Paulay & Priestley [27] recommended an ideology on the behavior of masonry infilled frame under earthquake loading and their design method. The writers mentioned despite the fact that overall lateral load capacity may be increased this will change the structural reaction and attract unwanted forces to various parts of the structure with an unbalanced arrangement. This in effect will lead to masonry infill wall causing structural deficiencies. However, from the recommendations of other studies [28], investigations of the behavior of masonry infilled wall date back to over 50 years but there is no clear approach to design these structures as with other common structural types. In case of large horizontal loads it has been observed that infill wall had poor behavior especially under seismic loads.

A typical example of the masonry infill walls influence on the behaviour of one-bay steel frame can be seen in Fig. 2.6. The masonry infill panel was modelled by linear and nonlinear equivalent compression strut models and nonlinearity was included by appropriate spring models. Results of the push-over analysis indicated the infill walls suitable influence at small drifts increasing the structural stiffness and strength. However, after the peak value was reached there was damage to the masonry and the panel frame became weaker as clearly shown in the graph below. The consequences were significant degradation of stiffness and strength and only low- to medium displacement ductility's could be achieved. The bare steel frame, on the other hand, had high possible ductility. The behaviour modes, of infilled and bare steel frames are different under horizontal loading.

According, Liu & Manes [29] mostly mansory walls are used to infill concrete or steel frames optionally as walls to divide open space interior of the structure or as a cover for the building outside. But, a shortage of scientific data on the actual amount of infill wall-frame synergy is observed. As a result a design procedure where masonry infills are frequently seen as non-structural elements and bounding frames are intended for both gravity and lateral loading [1]. The idea rooted on this method might end up expensive. However, if masonry infill walls were constructed with the objective to take part in the load sharing with the bounding frame, not considering the input of the infill wall to the stiffness and strength of the frame then this system may cause an unsafe design. Infill walls extremely stiffen the frame system, which can be destructive to lateral stability of the frame by attracting larger forces [30].

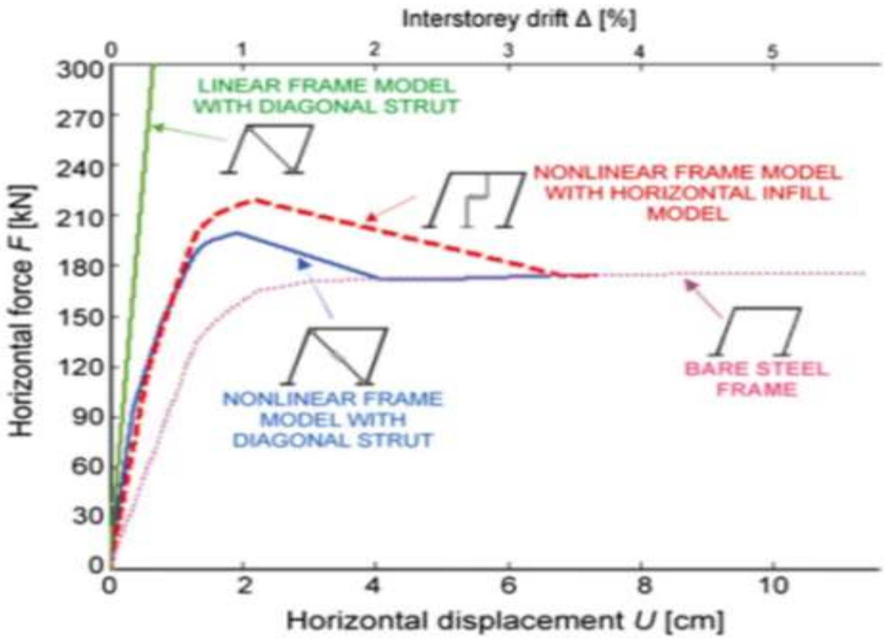


Figure 2.6: Effect of masonry infill on the behavior of steel frame (results of numerical analysis from (Zovkic, Sigmund, and Guljas 2013)).

According, Maheri, Kousari, and Razazan (2003) [31] confirmed that, the frame has been investigated with and without openings. The frame with brick masonry infill wall and having modulus values ranging between 500MPa to 8000MPa were analyzed. The material density and possion's ratio used for RC members, masonry and link are (2500 Kg/m^3, 0.20), (1920Kg/m^3, 0.18) and (zero*, 0.2) respectively. Three combinations have been compared according to their stiffness i.e., solid infill, 1D2W and 1D1W. It was seen that there was decrease in the stiffness (i.e., up to

49%). For the condition of 1DW2 and 1D1W, for all the moduluses, there was a decrease in stiffness of an average of 50% and 18% respectively. Similarly, the comparison was done between the solid infill, 1DW2 and 1D1W as in Fig 2.7.

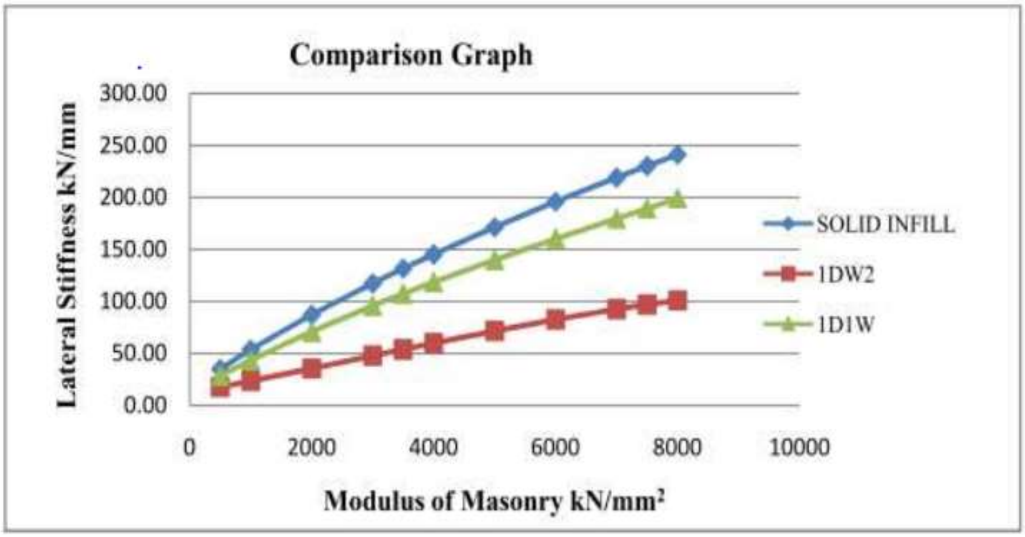


Figure 2.7: Comparison of experimental results between solid infill wall and walls with different types of window openings [31].

2.11 Possible Modes of Failure of Infill Walls

Ozturk [26] Proposed a design method on diagonally braced frame principle for infilled frame. There were three possible ways of failure for infill walls where the first one was crushing of corner of infill, shear along masonry, and diagonal cracking through masonry. An assumption was made for theadequate width of diagonal compression strut as the same as one-tenth of the diagonal length of the infill panel as shown in the diagram below. The adequate width will be assumed as $w = \frac{1}{10}d$ (Fig. 2.8). At the first design phase, the frame has to be modelled on the basis of the gravity loading.

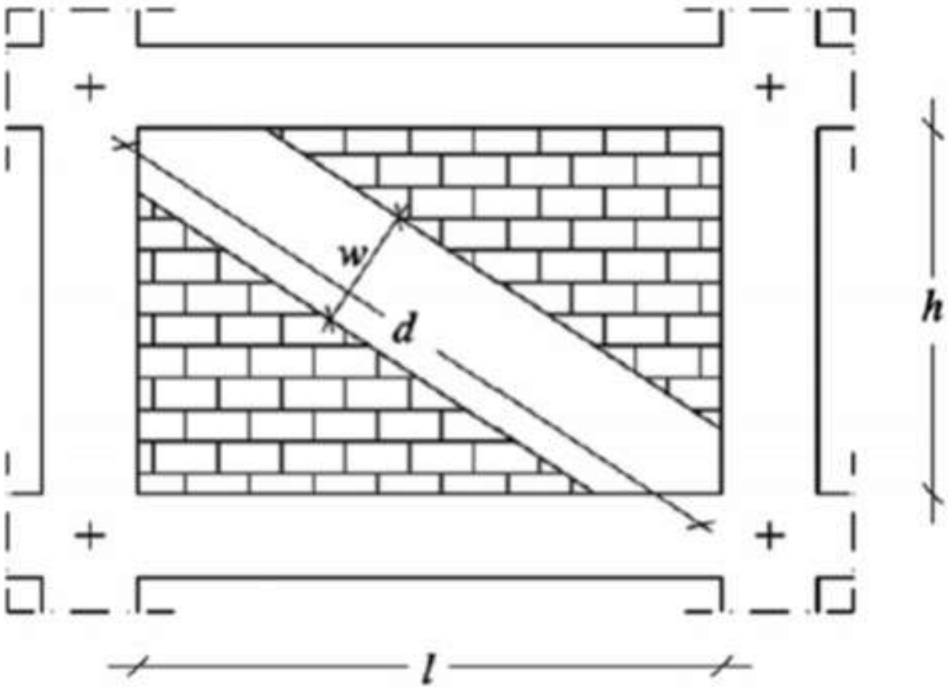


Figure 2.8: Diagonal strut model [32].

Chapter 3

TESTING PROGRAM

3.1 Introduction

This research was focused on the experimental investigation of the behavior of steel moment and braced frame with and without infill walls that are subject to lateral load, for example, earthquake and wind loads.

This chapter includes modeling, analysis and design of test frame in sections 3.2 to 3.4. These sections also give details on material properties and steel sections. Experimental set up is given in section 3.5 and test procedure is presented in section 3.7. The connections design for the test frames are given in section 3.8.

3.2 Modelling, Analysis and Design of Test Frames

The test frames are half scale models extracted from a full scale one-story office building with plan dimensions of 4m x 3m. The lateral stability of the building was assumed to be provided by either using Moment Frame (MF) or Braced Frame (BF). Lateral stability was again possible earthquake or wind loading. The test frames were modeled and subjected to lateral load which was increased until one of the members reached failure and the corresponding lateral displacement was also recorded. The steel sections used for the test frame model, analyzed and designed for lateral loading were chosen to build the test specimens.

Test frames with different bracing systems were designed to assess which bracing system has the most efficient behavior. The choice of test frame steel sections and their geometry are very important for this study, since they effect the results of frame inelastic behaviour [2,3] and economical aspects [4,5]. Thus, some of the aspects of the frame geometry were chosen from a previous experimental work [34]. The X bracing has been chosen because it is widely used in the area specified in the background to study. The Kobe and Northridge area also have extensive use of the X bracings due to their simplicity and high performance when it comes to attributes such as strength and ease of installation.

For this study, the experimental work involved the half-scale testing of eight frames. Four of them were moment frame with beams connected to the flange of the columns (major axis, with and without infill walls and the other four were braced frames with beams connected to the web of the columns (minor axis) with and without infill walls. Dimensions of the test specimens are given in Figures 3.1 and 3.2. Steel section dimensions for columns, beams and bracings are shown in Figures 3.3 and 3.4. Four of these test frames had walls built in the frame, where BIMs blocks (Fig. 3.4) were used for this purpose. Comparison of results, the lateral load resisted by the frames and the corresponding lateral displacement, for test frames with and without infill walls are expected to give information on the effect of using infill wall on the lateral load resisting capacity and the ductility of moment and braced frames.

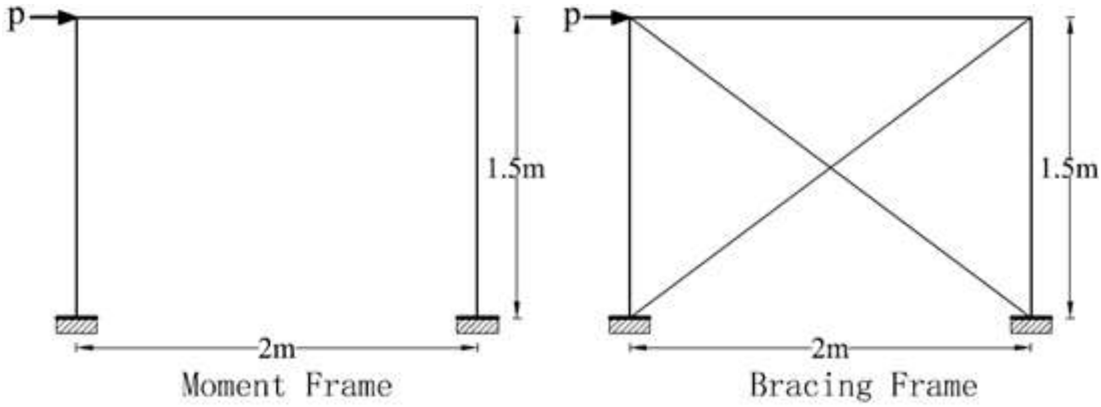


Figure 3.1: Moment and braced frames without infill wall.

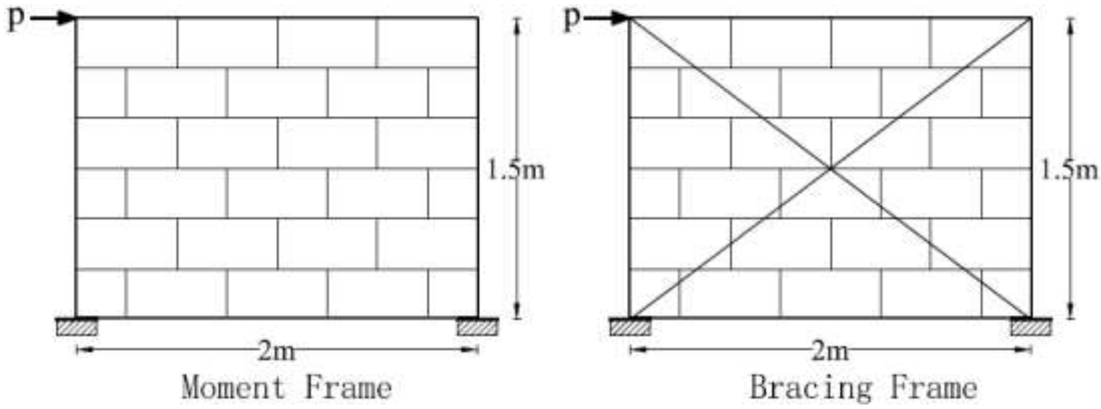


Figure 3.2: Moment and braced frames with infill wall.

3.3 Load Cases

A lateral load resisting system has to give the building adequate strength and stiffness to resist lateral loads mainly caused by wind or earthquake [35]. Lateral load was applied at the top of frame center of intersection point for column and beam. The procedure of lateral load application can be found in section 3.7.

3.4 Analysis and Design of Test Frames

In general, there are three main methods for determining the ultimate capacity of a member in a steel framed structure subject to lateral loads. These are analytical, experimental and numerical methods. Two of these methods are used in this study, analytical and experimental. The analytical method is important for simulation of the results obtained from the experimental work, and estimating the lateral displacement and vertical deflection values (ΔV) for the columns and the beam member. Eurocode EN 3-1993 was used to analyze and design the test frames.

ETABS is a general purpose analysis and design program developed specifically for building systems. ETABS Version 13.2.0 was used for test frame analysis and design.

3.5 Material Properties and Steel Sections Used

The mechanical properties of the materials used in the experiments were obtained through material tests. Two material groups were obtained: steel sections which are beam, column and bracing and BIMs blocks. According to BS EN 10025:1993 the Yield and Tensile strengths of steel sections are shown in Table 3.1. Structural steel grades of S275 and S235 were used for the test frames, the former was used for the beams, columns, bracings and connections and the latter was used for all the stiffeners.

Table 3.1: Nominal values of the yield strength and the ultimate tensile strength according to BS EN 10025:1993

Material		**Yield Strength N/mm^2**		**Tensile strength N/mm^2**
	t ≤16mm	16> t ≤40mm	40> t ≤63mm	3≥ t ≤100mm
S235	235	225	215	340-470
S275	275	265	255	410-560

The frames consisted of HE120B column section, IPE120 beam section and L60x60x6 for bracings. The dimensional details of frame sections are shown in

Figure 3.3. Therefore, the properties of sections are provided in Table 3.2. Further details on these tables are presented in Appendix A of this thesis.

Table 3.2: Steel section properties

Section	A (mm^2)	I_y (mm^4)	I_Z (mm^2)	W_pI_y (mm^3)	W_pI_z (mm^3)
HE120B	$10.96x10^2$	$864.4x10^4$	$317.5x10^4$	$165.2x10^3$	$80.97x10^3$
IPE120	$6.31x10^2$	$318x10^4$	$27.7x10^4$	$60.7x10^3$	$13.6x10^3$
L60X60X6	$6.91x10^2$	$22.8x10^4$	$22.8x10^4$	$5.29x10^3$	$5.29x10^3$

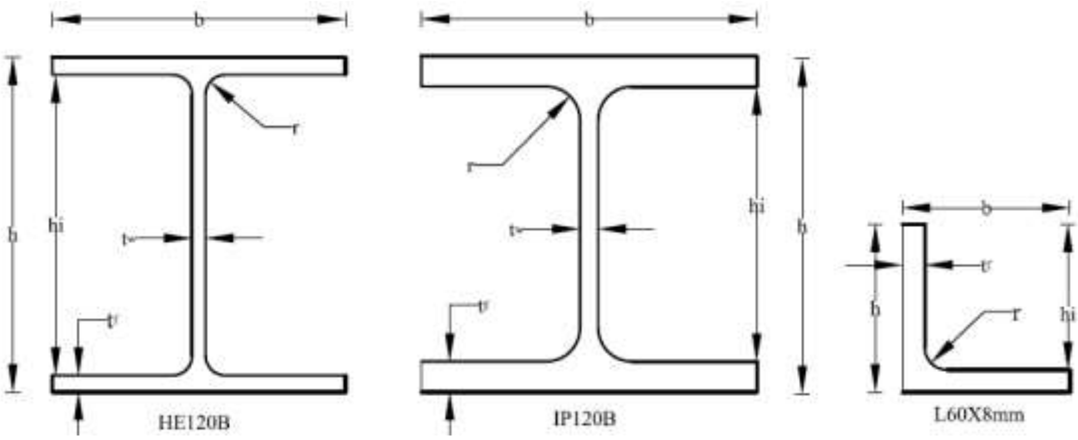


Figure 3.3: Column, Beam, and Bracing Dimension Details of Section for Frame System

Table 3.3: Sections dimension of the test frames

Section	Dimensions of section (mm)					
	h	b	t_w	t_f	R	h_i
HE120B	120	120	6.5	11	12	98
IPE120	120	64	4.4	6.3	7	107.4
L60x60x6	60	60	8		8	52

Considering Cyprus, the common materials for infill walls are; masonry, clay bricks, aerated blocks, BIMs blocks, concrete blocks or dry gypsum board walls. Therefore, BIMs blocks, which were used for the infill walls of test frames. Plastering was also used on the boundary of the wall to increase the strength by reducing the gaps between the frame and the wall (BIMs block). BIMS blocks are rectangular shape and usually 390 mm by 120 mm by 200 mm as specified in the order length x width x height (Fig. 3.4).

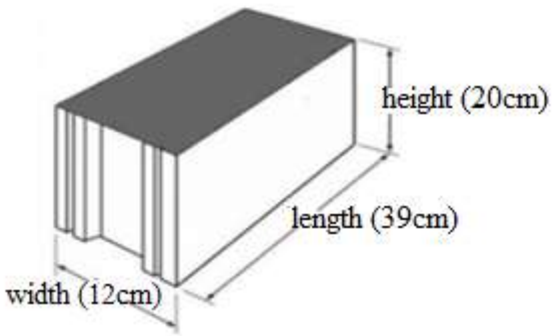


Figure 3.4: Dimensions of BIMs block.

3.6 Experimental set-up

A self-equilibrating test set-up, as illustrated in Figs. 3.5 and 3.6, were used to test moment frame without and with masonry infilled wall. These frames were subjected to lateral loading applied at the frame's beam to column connection level. The hydraulic jack loading arrangement was supported off the reinforced concrete 1250 mm thick reaction wall. The moment frame specimens with and without infill wall were made up of HE120B steel column and IPE120 steel beam and then via the columns base plates they were connected to the steel supports which were also connected to 1400 mm thick reinforced concrete strong floor. Lateral load was applied by a hydraulic jack (1000 kN capacity) to the mid height of the top beam, IPE 120 beam, and it was measured by a load cell having a capacity of (1000 kN).

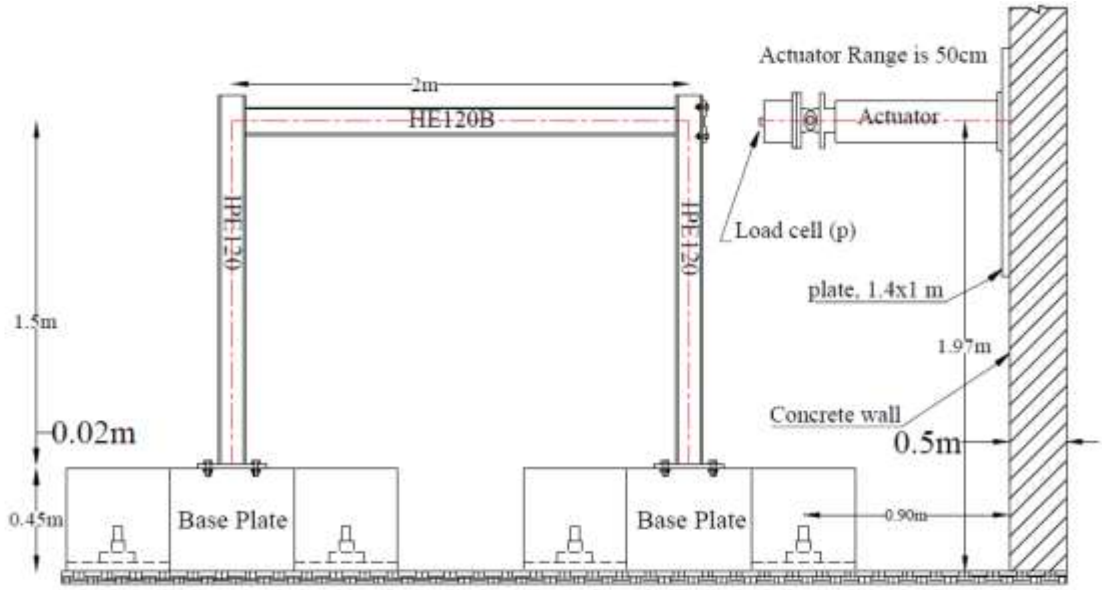


Figure 3.5: Test frame set-up for major and minor axis (without infill wall).

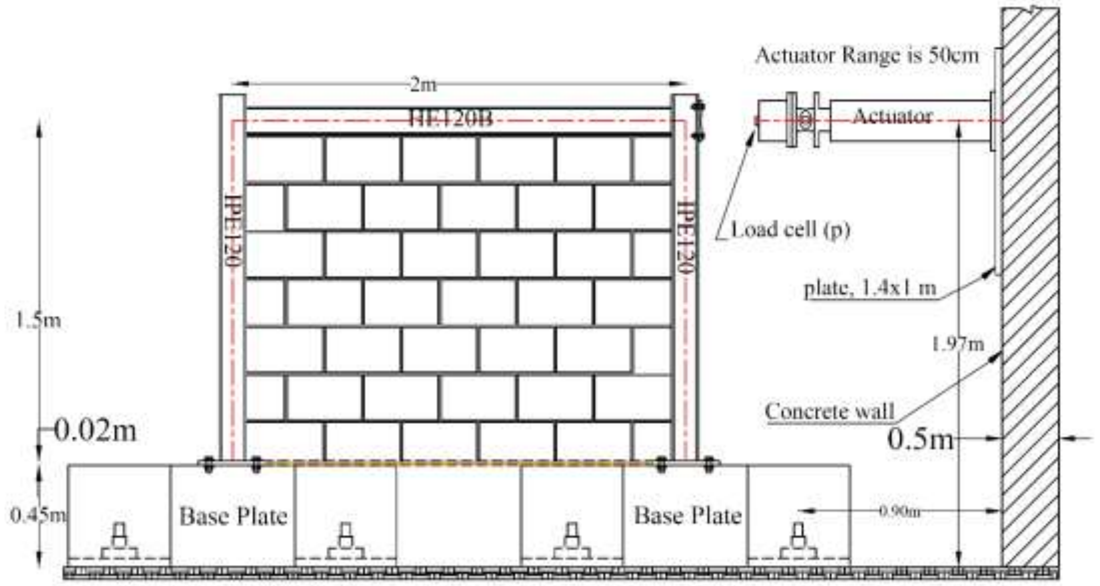


Figure 3.6: Test frame set-up for major and minor axis (with infill wall).

3.7 Instrumentation for Frame Tests

Instrumentation was consisted of linear variable displacement transducers (LVDT), were set to measure the frame drift which two have measure displacement until 100 mm and load cells having lateral loading as consented. In addition four uni-axial strain gauges were installed on the brace elements of the test specimens to measure strains parallel to the main axes of the braces (SG1 to SG4 in Figs. 3.7 and 3.8). The strain gauges on brace elements can measure both axial and out-of-plane bending strains.

The lateral loading system made it possible to apply loads proportional to top of beam level. Attachment detail of LVDTs' and cell lateral loads' on steel frames were used with and without infill walls is shown in Figures 3.5, 3.6, 3.7 and 3.8.

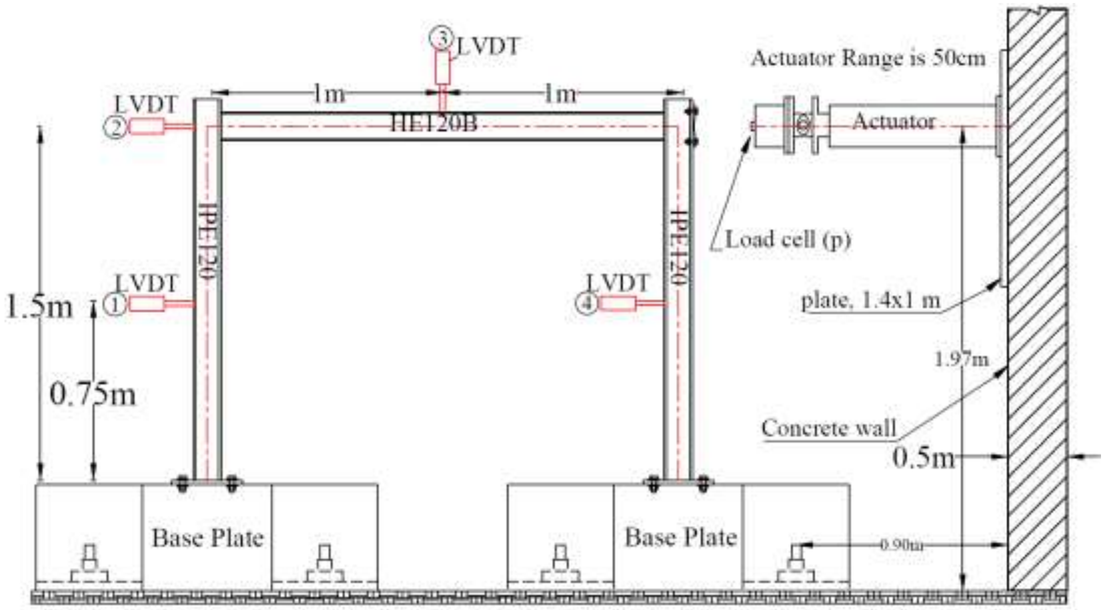


Figure 3.7: Locations of LVDTs and Load the major or minor axis for Moment Frame without infill wall (MAJ-MF5, MIN-MF1).

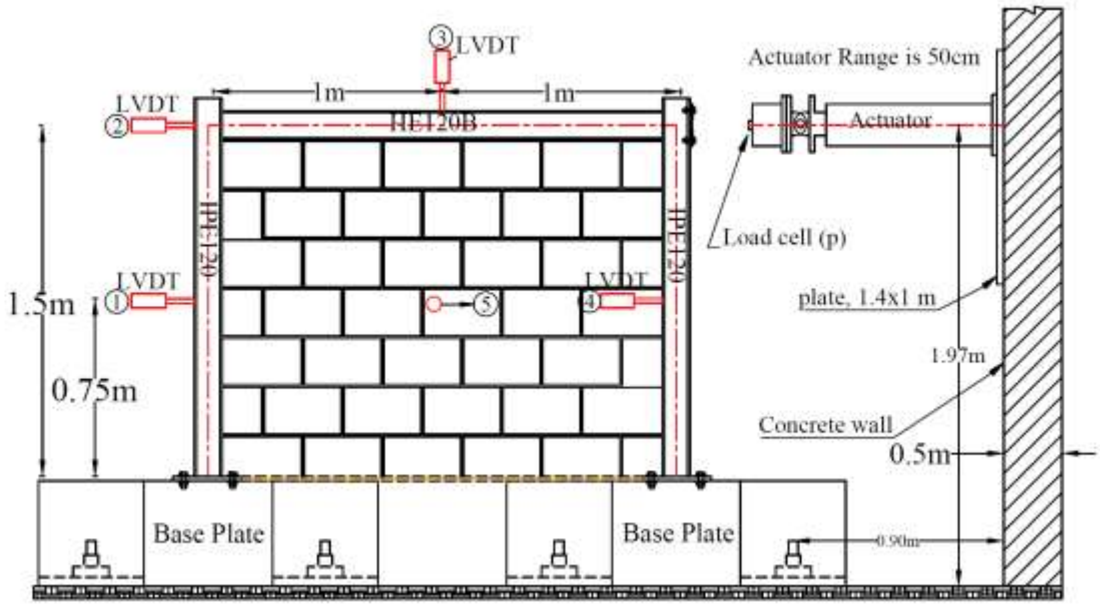


Figure 3.8: Locations of LVDTs and Load the major or minor axis for Moment Frame with infill wall (MAJ-MFIN6, MIN-MFIN2).

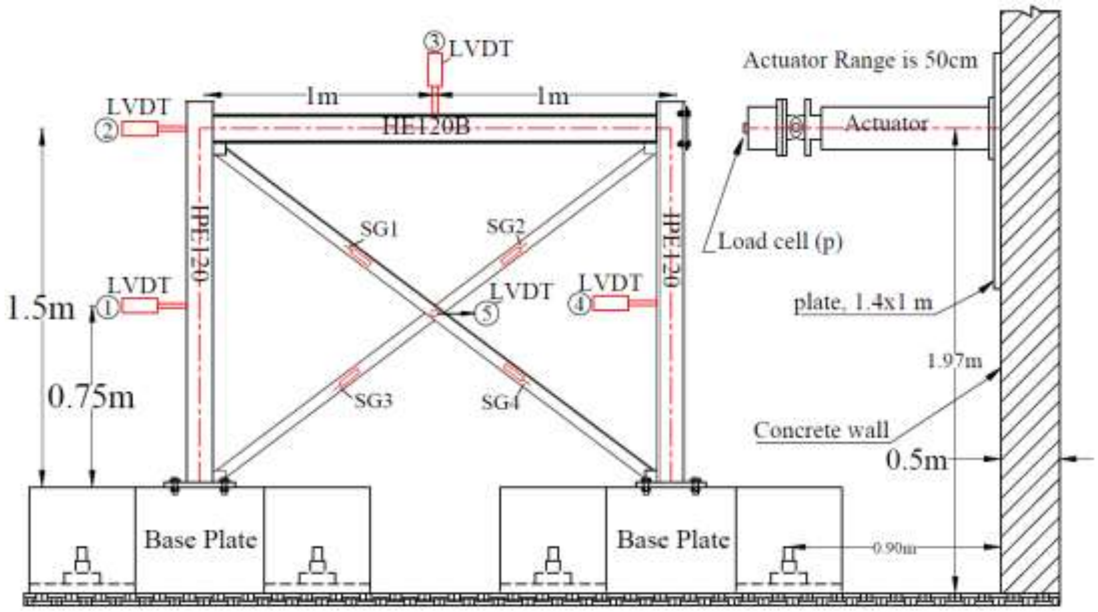


Figure 3.9: Locations of LVDTs and Load Cell on the major or minor axis for Braced Frame (MAJ-BF7, MIN-BR3).

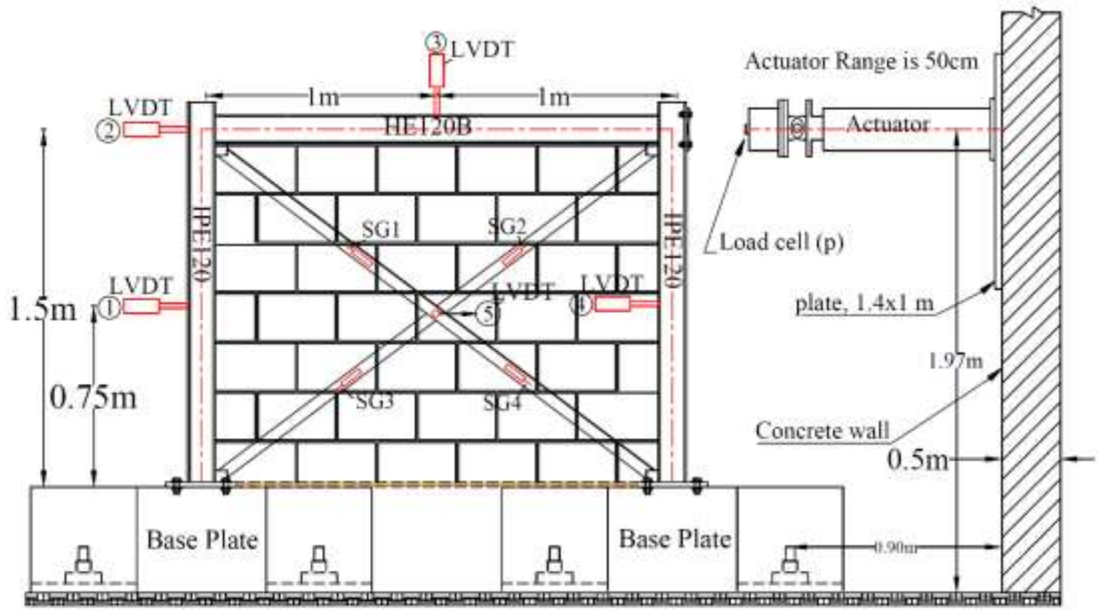


Figure 3.10: Locations of LVDTs and Load Cell on the major or minor axis for Braced Frame infill wall (MAJ-BFIN8, MIN-BRIN4).

3.8 Test Procedure

Firstly, the specimen was positioned in the test frame before any test was conducted and it was aligned properly for both in-plane and out of plane direction. LVDTs were placed at their specified areas and all readings were checked to make sure that they functioned properly before the test began. Lateral load was applied constantly at approximately a rate of 5 kN per minute up to the point where the specimen fractured. The failure was expected to happen when the specimen showed a permanent decline in the load. Load cell and LVDT readings were observed and recorded with an interval of 0.1 s overall for each test using an electronic data acquisition system. Amid each test, displacement, the crack pattern corresponding to the, ultimate load were recorded.

3.9 Connection Design of Test Frames

In this part, two types of semi-rigid connections were used. These two types of connections are stiffened extended end plate and stiffened fin plate connections. They are used for moment and braced frames, respectively. The connection was designed as Simple connections are clear in section (i) to (v). The connection design are carried out according to British Steel Design Code BS 5950-2000.

i. Pinned Connections for Moment Frame (major axis)

Distinctive flexible end plate connections are shown in figure 3.11 (a). These are assumed to transmit end shear only and to have ineffective resistance to rotation. Hence, they do not transfer substantial moments at the ultimate limit state. This explanation underlies the design of multi-storey braced frames in Britain designed as 'simple construction' in which the beams are designed as simply-supported and the columns are designed for axial load and the small moments induced by the end reactions from the beams [36].

ii. Flexible End Plate Connections for Moment Frame (minor axis)

Typical flexible end plate connections are shown in Figure 3.11 (b); it is assumed to transmit end shear only and to have resistance to rotation. The end plate is welded to the end of beam at the fabrication shop.

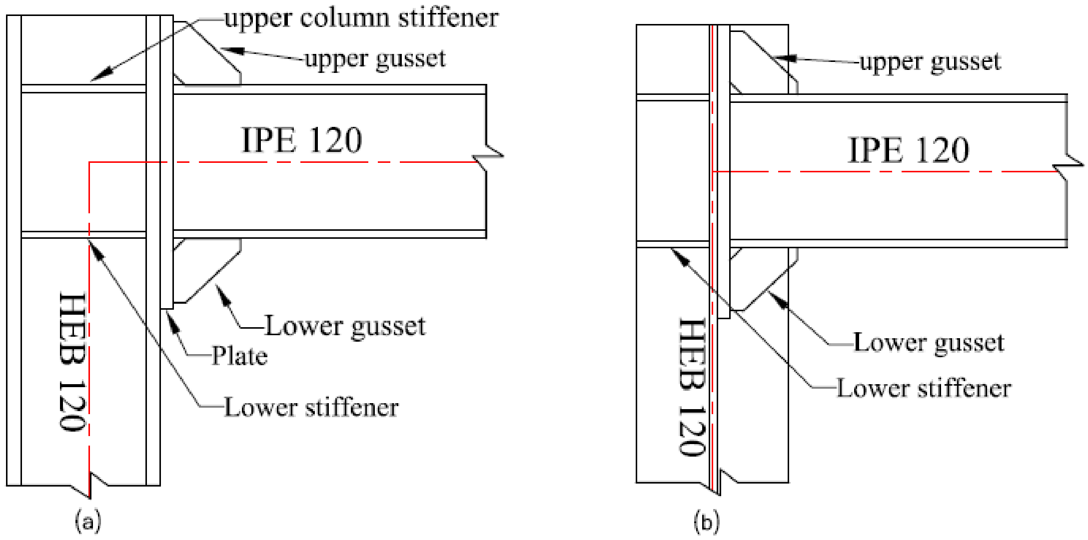


Figure 3.11: Geometric variables for major and minor axis of moment framed connection

iii. Fin Plates Connection of Bracing Frame (major axis)

Typical Fin Plate Connections are shown in figure 3.12 (a). Fin plate connections allow the use of minimum resources to fabricate and simple to erect lessening cost. These connections are widely used because of their advantages of cost and ease to erect. A fin plate connection is simply connection include, Beam-to-beam and beam-to-column connections.

A fin plate connection has a piece of plate welded during manufacturing to the supporting member, and the supported beam web and bracing is bolted on site the figure below illustrates.

iv. Fin Plates Connection of Bracing Frame (minor axis)

Typical fin plate connections are shown in Figure 3.12 (b). A fin plate connection is simple connection and includes beam-to-beam and beam-to-column web connections.

Bracing systems are assumed that all forces intersect on member centerlines to overpass the effects of any significant eccentricity. However, realizing this assumption during the connection design may result in a connection with a very large gusset plate, as given in Fig. 3.12.

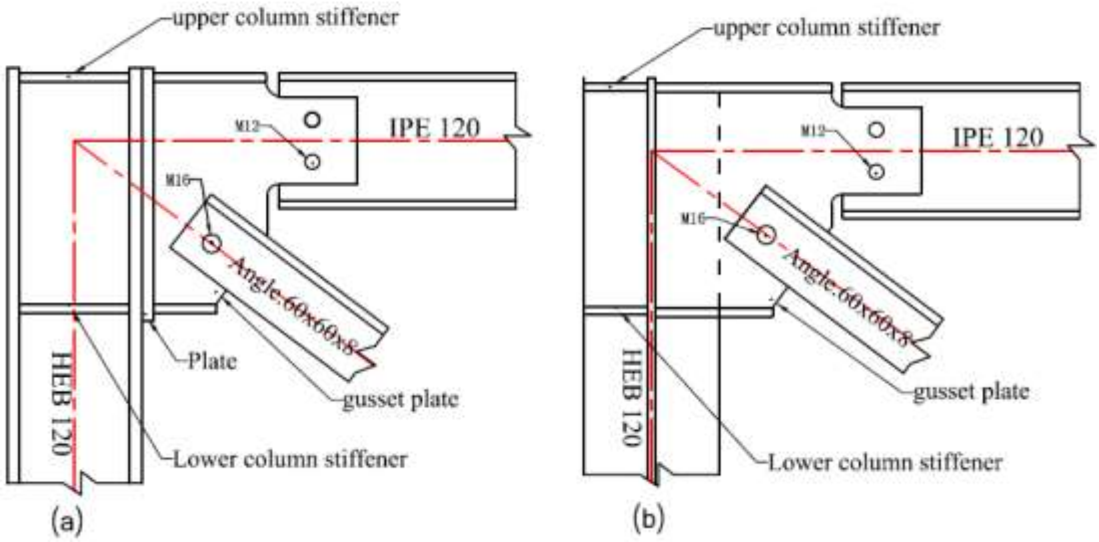


Figure 3.12: Geometric variables for major and minor axis braced framed connections.

v. Column Bases

Column bases are shown in figures 3.13 and 3.14. They comprised of one plate fillet welded to the end of the column and connected to very stiff steel bases via four bolts that are in turn connected to 1.4 m thick strong floor.

Bracing arrangements may involve the bracing members working in tension alone, or in both tension and compression. The bracing member is connected by bolting to a gusset plate, which is welded to the column, and to the base plate as shown in the Figure 3.12.

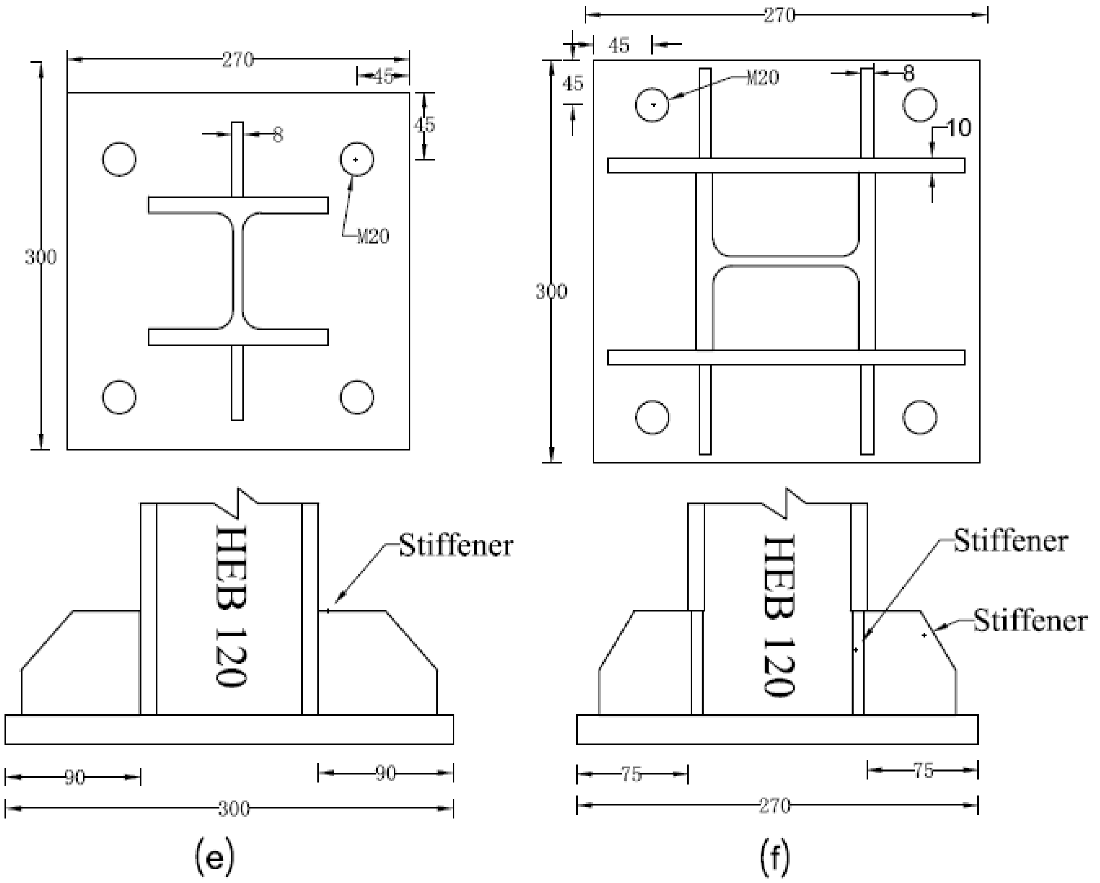


Figure 3.13: Moment frame column base plate design for major-minor (all dimension are mm).

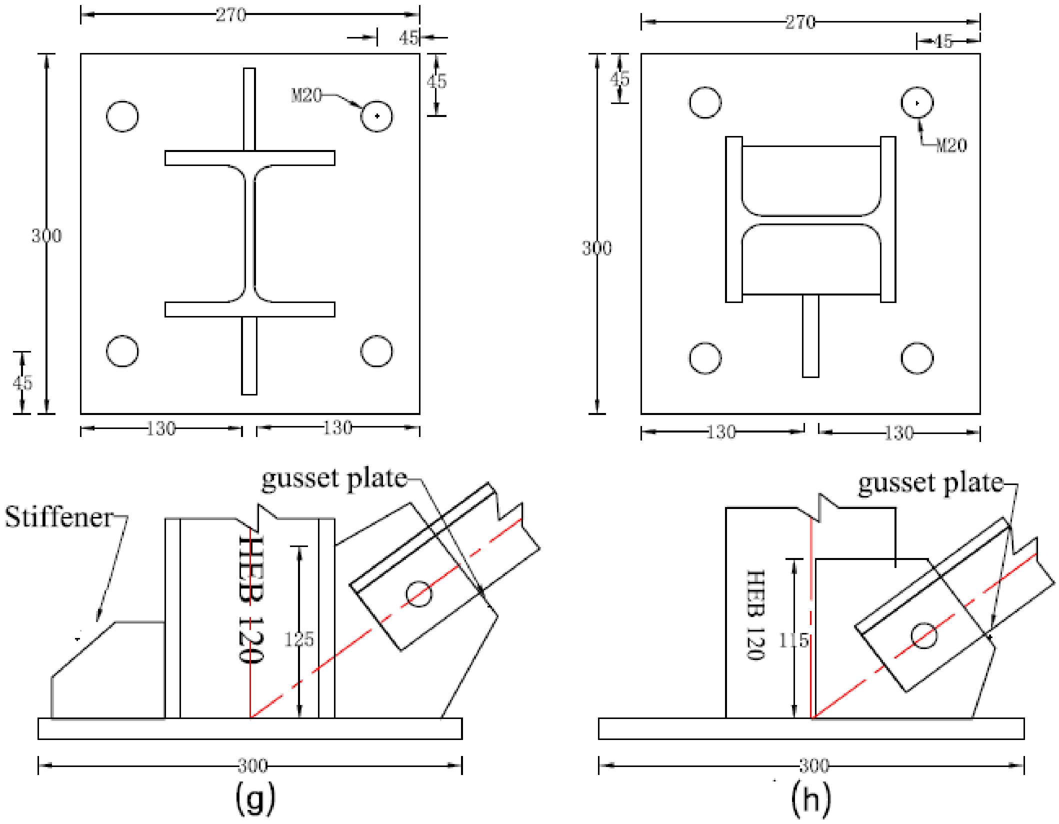


Figure 3.14: Braced Frame column base plate design for major-minor axis (all dimension are mm).

3.9.1 Structural Bolts

In this section, the background of standard hexagonal bolts is presented (Fig. 3.13). Bolt grade of 10.9 was used and the strength of the structural bolts was as per the British standards, BS 5950-2000. In this case study specifies requirements to used M12, M16, and M20 as shown in figure 3.12. To determine the length of bolts, one can follow Table 3.2, as per DIN 7990.

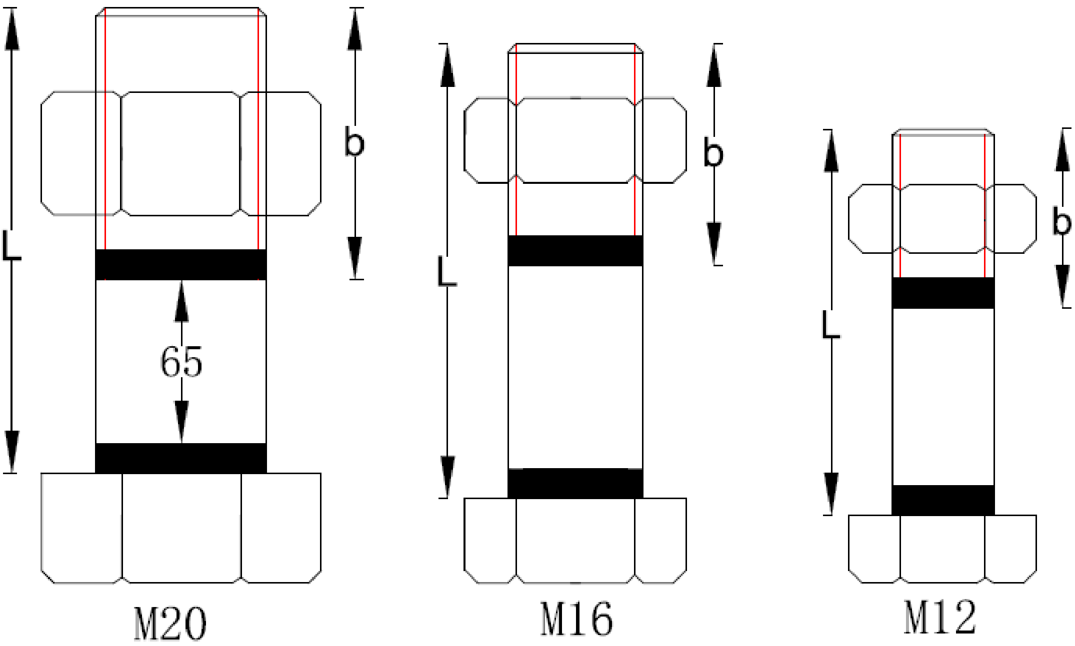
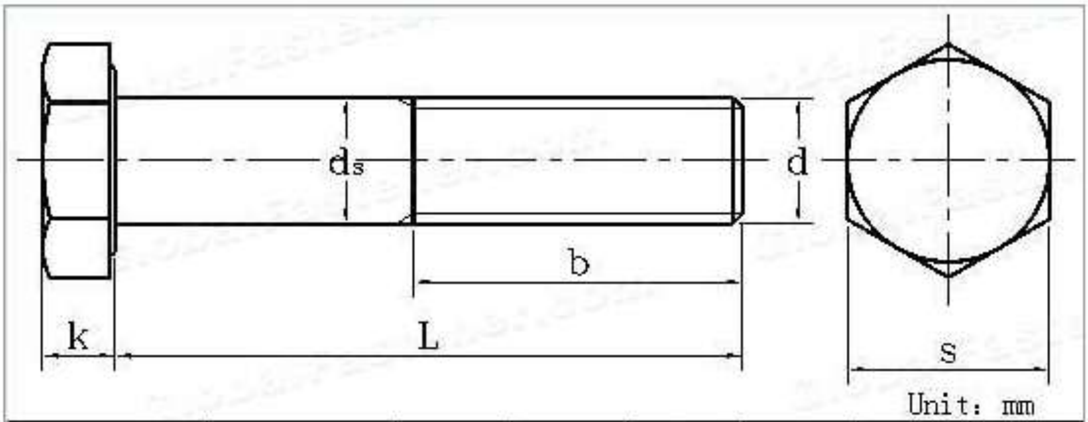


Figure 3.15: Hexagon head structural bolts used in experimental tests.

Table 3.4: Bolt and nut dimensions [5] (all dimenstion are mm).



d		M12	M16	M20	M22	M24	M27	M30
P		1.75	2	2.5	2.5	3	3	3.5
b		17.75	21	23.5	25.5	28	29	30.5
d_s	max	12.7	16.7	20.84	22.84	24.84	27.84	27.84
	min	11.3	15.3	19.16	21.16	23.16	26.16	26.16
K	max	8.45	10.75	13.9	14.9	15.9	17.9	20.05
	min	7.55	9.25	12.1	13.1	14.1	16.1	17.95
S	max	18/19	24	30	32/34	36	41	46
	min	17.57/18.48	23.16	29.16	31/33	35	40	45

Chapter 4

EXPERIMENTAL RESULTS AND DISCUSSIONS

4.1 Introduction

Included in this chapter are the results of eight half scale steel framed tests with and without infill walls. The test set up details are given in chapter 3 where all the locations of LVDTs that were used for measuring displacements and the locations for strain gauges that were used to eventually calculate the axial forces in the cross bracings are given. The readings taken from these LVDTs, strain gauges and the load cell were used to produce the tables and graphs given in this chapter. The main aim was to understand the behavioral changes between steel moment framed and braced framed structures and the effect of infilling them with walls.

Steel beam was connected to steel columns either via the column flange (major axis) or column web (minor axis). So the eight tests were divided as moment or braced frame with minor or major axis and with or without infill walls. This created the eight specimens which were identified by the following test numbers: MIN-MF1, MIN-MFIN2, MIN-BR3, MIN-BRIN4, MAJ-MF5, MAJ-MFIN6, MAJ-BF7 and MAJ-BFIN8 (Table 4.1). Two base supports, 1000 mm long 450 mm wide and 500 mm high were used to securely connect the column base plates. There was a cumulative horizontal load applied at the top right hand corner of the beam. Lateral load was applied gradually at an approximate rate of 5 to 8 kN per second until the failure of the specimen. The failure was considered to have occurred when the applied load was noticed to have an irreversible decrease. Also, the magnitudes of displacements at the top left corner (LVDT2) of the panel were recorded.

Table 4.1: Key for the steel frame test numbers.

Test No.	Axis	Framing Method	With/Without Infill Wall
MIN-MF1	Minor	Moment	
MIN-MF-IN2	Minor	Moment	Infill Wall
MIN-BR3	Minor	Braced	
MIN-BRIN4	Minor	Braced	Infill Wall
MAJ-MF5	Major	Moment	
MAJ-MF-IN6	Major	Moment	Infill Wall
MAJ-BF7	Major	Braced	
MAJ-BFIN8	Major	Braced	Infill Wall

4.2 Mechanical Properties of Steel Sections Used for Test Frames

Tensile tests provided mechanical properties of the steel sections, beams, columns and equal leg angles, used for the tests (Table 4.2). Three specimen were taken from the flanges and web of the beam (FL1, FL2, WE3) flanges and web of the column (FL3, FL4, WE5) and two legs of the angle (L1, L2). The specimens had higher yield stresses ranging from 306.5 MPa to 399.3 MPa than the nominal yield stress of 275 MPa. The ultimate stress of the specimens was approximately ranging from 462.1 to 590.8 MPa which was within the range of the nominal stress of 340 MPa to 560 MPa. The actual dimensions of these steel sections are given in Table 4.3.

Table 4.2: Mechanical properties of the tested steel sections.

Steel section	Sample No.	Thickness	Coupon dimension of sample testing with	Lo	Lf	Area	Yield stress	Ultimate stress	Ultimate strain
		(mm)	(mm)	(mm)	(mm)	(mm^2)	(N/mm^2)	(N/mm^2)	%
HEB120	FL1	9.47	25.00	200.00	258.00	236.75	312.60	466.70	29.00
	FL2	9.47	25.00	200.00	257.00	236.75	321.00	466.70	28.50
	WE3	6.60	25.00	200.00	245.00	165.00	385.50	490.90	22.50
IPE120	FL3	5.35	25.00	200.00	245.00	133.75	306.50	462.10	22.50
	FL4	5.35	25.00	200.00	240.00	133.75	306.50	462.10	20.00
	WE5	3.56	25.00	200.00	249.00	89.00	343.80	516.90	24.50
L60X6	L1	5.85	25.00	200.00	240.00	146.25	396.60	582.60	20.00
	L2	5.85	25.00	200.00	244.00	146.25	399.30	590.80	22.00

Table 4.3: Sections properties used in the test frame

Test No	HEB120 (mm)				IPE (mm)				L 60x60x6 (mm)		
	Flange Thick (t_f)	Web Thick (t_w)	Height (h_c)	Width (b_c)	Flange Thick (t_f)	Web Thick (t_w)	Height (h_c)	Width (b_c)	Leg Thick (t_a)	Leg Lengt (L_1)	Leg (L_2)
MIN-MF1	10.0	7.0	122	121	6.4	6.5	120	64		60	60
MIN-MFIN2	10.0	6.5	122	121	6.4	6.5	120	64		60	60
MIN-BR3	10.5	7.0	122	121	7.0	6.5	118	64	6	60	60
MIN-BRIN4	10.0	6.5	122	121	6.5	6.5	118	64	6	60	60
MAJ-MF5	10.0	6.5	123	121	7.0	5.6	120	64		60	60
MAJ-MFIN6	10.0	6.5	122	121	7.0	5.5	120	64		60	60
MAJ-BF7	10.0	6.5	122	121	7.0	5.0	119	64	6	60	60
MAJ-BFIN8	10.0	6.5	120	121	7.0	5.5	120	64	6	60	60

4.3 Minor Axis Frame Tests

In this test the beam two ends were connected to column webs, therefore, during the tests columns were subjected to bending mainly around their minor axis (Fig. 4.1).

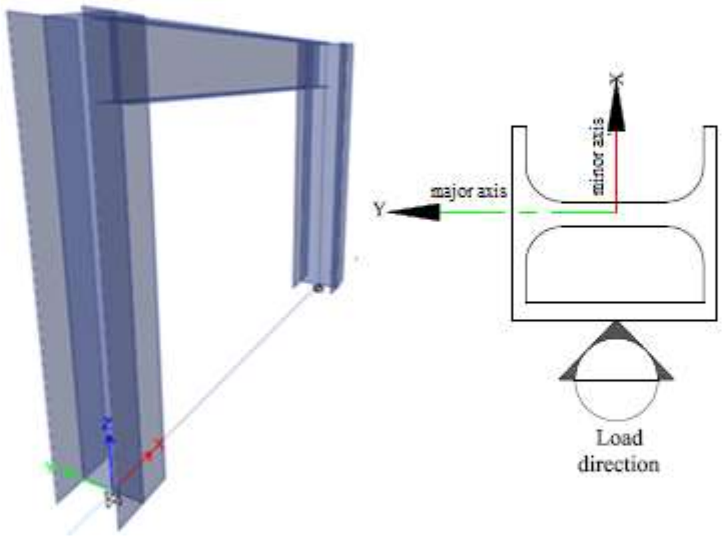


Figure 4.1: Minor axis frame and load direction for HE120B column section.

4.3.1 Moment Frame without Infill Wall (MIN-MF1)

The test setup consisted of lateral load applied by using a hydraulic jack with a capacity of 1000 kN. The test set up including the locations of LVDTs and the hydraulic jack are given in Figure 3.5 and the readings taken from these measuring devices are given in Table 4.4. It shows the results of LVDT and rotation testing for (MIN-MF1) under loading applied gradually.

Table 4.4: Displacements and rotations measured at various locations for the moment frame without infill wall (MIN-MF1).

Lateral Load P (kN)	Displacement (Δ) LVDT1 (mm)	LVDT2 (mm)	Drift ratio (α) (%)	Rotation (ø) (radians)
0.00	0.00	0.00	0.00	0.00
1.33	0.11	0.94	0.02	1.10
4.83	0.70	2.17	0.15	1.95
6.00	0.95	2.91	0.19	2.62
9.66	1.63	4.43	0.30	3.73
12.83	2.30	6.10	0.41	5.07
15.00	2.64	6.87	0.46	5.65
18.00	3.23	8.31	0.55	6.77
20.83	3.83	9.74	0.65	7.88
22.00	4.00	10.15	0.68	8.19
25.00	4.58	11.48	0.77	9.20
27.84	5.16	12.87	0.86	10.27
30.84	5.77	14.24	0.95	11.29
32.34	6.17	15.25	1.02	12.11
34.67	6.69	16.56	1.10	13.15
38.67	7.58	18.39	1.23	14.41
40.01	7.95	19.21	1.28	15.01
42.51	8.69	20.85	1.39	16.21
45.01	9.30	22.04	1.47	16.99
47.17	10.05	23.73	1.58	18.24
50.01	11.29	26.52	1.77	20.30
52.51	12.68	29.49	1.97	22.41
55.01	14.50	33.25	2.22	24.99
57.51	16.80	37.83	2.52	28.04
60.01	19.85	43.90	2.93	32.07
62.51	24.50	53.23	3.55	38.31
65.01	31.35	67.77	4.52	48.57
66.01	35.69	77.29	5.15	55.46
67.01	40.00	86.83	5.79	62.43
67.51	41.48	90.09	6.01	64.81
66.68	41.63	90.39	6.03	65.01

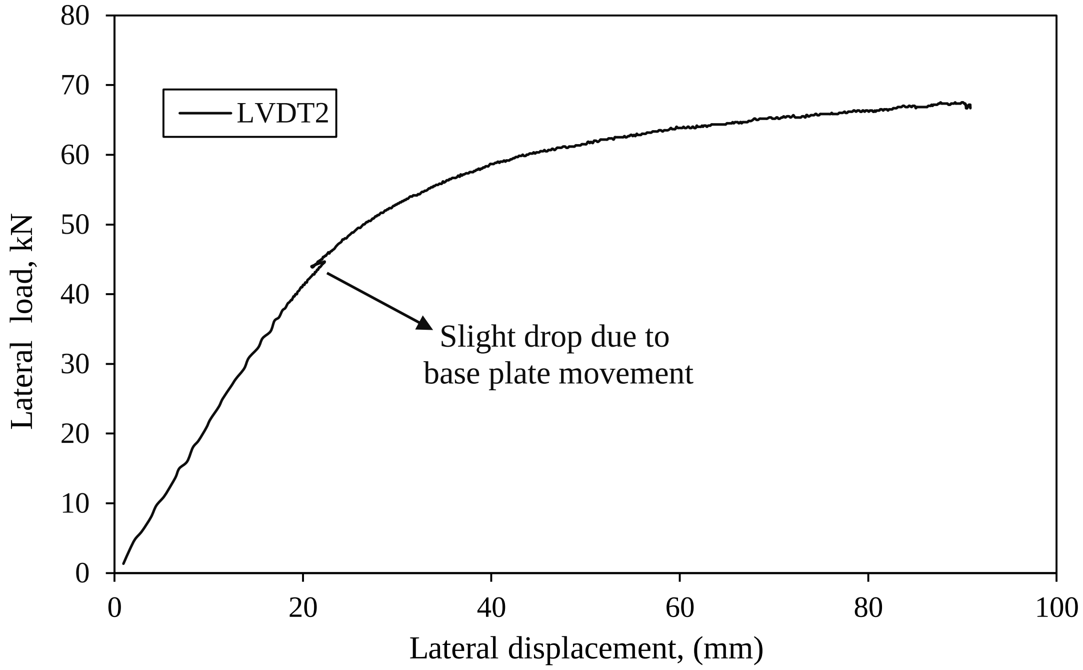
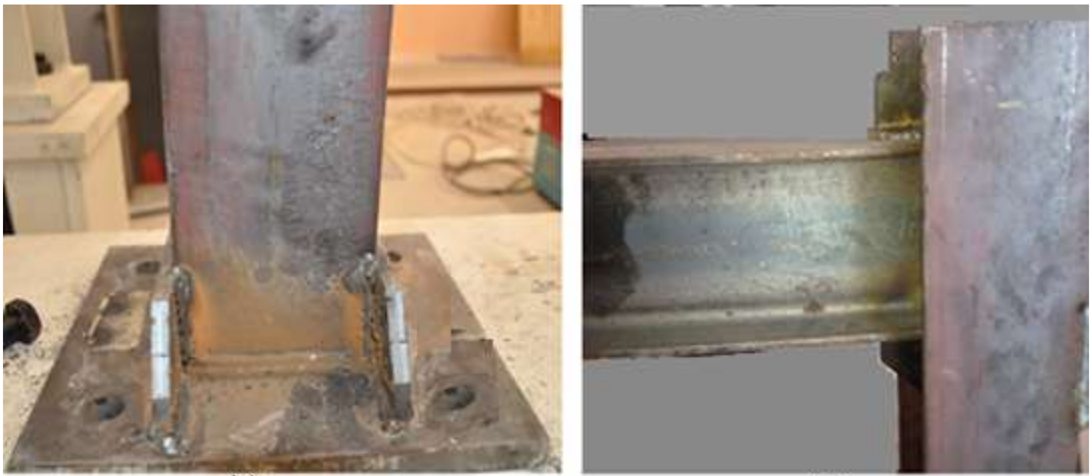


Figure 4.4.2: Lateral load versus lateral displacement for test frame MIN-MF1.

The test specimen MIN-MF1 exhibited elastic behaviour up to a lateral load of approximately 44.5 kN and a lateral displacement of 22.26 mm (Fig. 4.2). The peak load and the corresponding displacement of test frame MIN-MF1 were 67.51 kN and 90.09 mm, respectively. The graph vividly shows that plasticity starts at 45.0 kN and continues until the specimen fractures. The load-displacement curve also shows a slight drop in the lateral load with a corresponding minor reduction in displacement just at a point where the plasticity starts. Then the load continues to increase together with increase in displacement until the frame fails due to reduction in stiffness caused by yielding of column and spreading of plasticity due to the lateral force at column. This effect also transferred to the beam (Fig. 4.3).

There was no indication of local buckling in any of the steel beam-to-column connections of the test frames. Yielding occurred at column, at a location that is very close to the column base and the beam was subjected to lateral torsional buckling and the rotation of the beam flanges were particularly behaviour close to the ends of the beam of test frame MIN-MF1. Figures 4.3a and 4.3b show a typical damage pattern of the columns and one end of the beam, respectively, after withdrawing the lateral load from the test specimen. Figure 4.3 (a) shows the column and column base plate and Figure 4.3 (b) shows the beam end and the rotation of beam flange due to lateral torsional buckling.



(a) (b)

Figure 4.3: Structural damages observed during and after testing (a) Column and column base plate damage, (b) beam end damage of frame MIN-MF1.

4.3.2 Moment Frame with Infill Wall (MIN-MFIN2)

A hydraulic jack of capacity 1000 kN applied a lateral load included in the test setup. Figure 3.6 shows locations of LVDTs and the hydraulic jack and the readings from these devices are shown in table 4.5. This same table has results of LVDT and rotating testing for (MIN-MFIN2) under loading applied gradually.

Table 4.5: Displacements and rotations measured at various locations for the moment frame with infill wall (MIN-MFIN2).

Lateral Load	Displacement (Δ)		Drift ratio (α)	Rotation (ϴ)
P	LVDT1	LVDT2	$\times 10^{-3}$	$\times 10^{-3}$
(kN)	(mm)	(mm)	(%)	(radians)
0.00	0.00	0.00	0.00	0.00
1.60	0.00	0.00	0.00	0.00
4.93	0.12	0.36	0.24	0.32
6.23	0.43	0.36	0.24	0.09
9.81	0.43	0.67	0.45	3.20
16.43	0.85	1.62	1.10	1.03
15.00	0.85	1.62	1.10	1.03
21.30	0.99	2.02	1.35	1.40
26.67	1.24	2.44	1.63	1.60
30.34	1.54	3.05	2.03	2.01
32.67	1.80	3.46	2.31	2.20
35.51	2.11	3.95	2.63	2.50
34.84	2.13	3.99	2.66	2.50
39.34	2.42	4.47	2.98	2.60
38.34	2.44	4.50	3.00	2.80
42.51	2.92	5.33	3.53	3.20
44.51	3.37	6.07	4.05	3.60
46.34	4.04	7.05	4.70	4.00
47.01	4.11	7.16	4.77	4.10
46.51	4.21	7.31	4.87	4.10
47.34	4.39	7.61	5.07	4.30
45.51	4.91	8.55	5.70	4.90
45.18	5.05	8.79	5.86	5.00
44.51	5.07	8.83	5.89	5.01
48.68	6.16	10.78	7.19	6.20
52.34	7.42	12.99	8.66	7.40
54.01	8.87	15.97	1.06	9.50
52.01	8.91	16.08	10.72	9.60
51.01	8.92	16.11	10.74	9.60

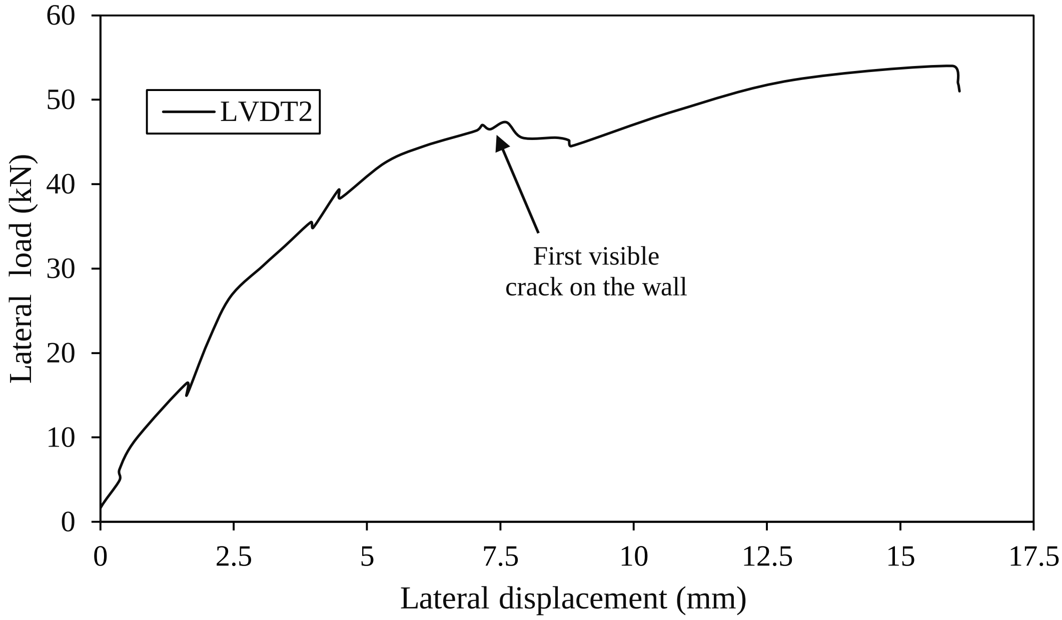


Fig. 4.4: Lateral load versus lateral displacement for test frame MIN-MFIN2.

The test specimen MIN-MFIN2 exhibited elastic behavior up to approximately 42.5 kN lateral load, which corresponds to a lateral displacement at the top of frame 5.33 mm. The first visible crack was noticed at a lateral load of approximately 47 kN and a lateral displacement of 7.16mm. Figure 4.4 shows the distribution of the data on the curve is inconsistent but there was no visible unevenness of the beam. The pattern of diagonal cracks were in such a way that a single diagonal crack occurred at the end of first ten loading steps, which corresponds to approximately 15 kN (Fig. 4.5). By increase the load, the first crack developed into another diagonal crack and few shorter branching off these main diagonal cracks, as can be seen in Fig. 4.5 (b). Two diagonal hairline cracks were started to form at the top compression corner of the infill panel at approximately 45°. This means that diagonal compression strut mechanism was fully developed as shown in Fig. 4.5 (b). This approach applies to the mansory infilled frame as a braced frame replaced by mansory infill to resist the lateral loading equivalent diagonal strut acting in compression [29].

The peak lateral load applied to the MIN-MFIN2 specimen was 54.01 kN and 15.97mm lateral displacement was achieved. At this point, cracks of infill wall were diagonal cracks and there was no crushing at the top left and top right corners of the panel.

(a) (b)

Figure 4.5: Diagonal cracks of the infill wall at the location of a compression diagonal bracing for test frame MIN-MFIN2.

4.3.3 Braced Frame without Infill Wall (MIN-BR3)

A 1000 kN capacity hydraulic jack applies lateral load to the test setup. In this test setup, locations of LVDTs and hydraulic jack are given in Figure 3.7. The readings recorded from these measuring devices are displayed in Table 4.6. It displays the results of LVDT and rotating testing for (MIN-BR3) under gradually applied load.

Table 4.6: Displacements and rotations measured at various locations for the braced frame without infill wall (MIN-BR3).

Lateral Load P	Displacement (Δ) LVDT1	Displacement (Δ) LVDT2	Drift ratio (α) $\times 10^{-3}$	Rotation (ɵ) $\times 10^{-3}$
(kN)	(mm)	(mm)	(%)	(radians)
0.00	0.00	0.00	0.00	0.00
7.50	0.11	0.01	0.07	-0.01
10.84	0.30	0.01	0.07	-0.04
15.84	0.51	0.50	3.33	0.00
22.50	0.74	0.98	6.53	0.03
25.67	0.86	1.32	8.80	0.06
30.84	1.13	1.85	12.33	0.10
35.01	1.24	2.13	14.20	0.12
40.01	1.43	2.62	17.47	0.16
45.18	1.63	3.11	20.73	0.19
50.34	1.85	3.63	24.20	0.24
55.18	2.02	4.08	27.20	0.28
60.18	2.24	4.64	30.93	0.32
65.35	2.37	4.99	33.27	0.35
70.35	2.52	5.36	35.73	0.38
75.18	2.65	5.73	38.20	0.41
80.01	2.77	6.02	40.13	0.43
85.18	2.91	6.41	42.73	0.47
90.18	3.09	6.88	45.87	0.51
95.35	3.31	7.41	49.40	0.55
100.35	3.37	7.54	50.27	0.56
105.35	3.59	8.13	54.20	0.61
110.19	3.96	9.20	61.33	0.69
114.02	4.29	10.14	67.60	0.78
113.36	4.55	10.96	73.07	0.86
113.02	4.78	11.66	77.73	0.92
98.69	4.68	11.50	76.67	0.91
104.69	4.82	11.79	78.60	0.93
110.02	5.30	13.15	87.67	1.05
115.52	5.86	14.71	98.07	1.18
120.02	6.26	15.86	105.73	1.28
125.36	6.71	17.13	114.20	1.39
130.03	7.21	18.55	123.67	1.51
135.03	7.80	20.20	134.67	1.65
138.03	8.61	22.46	149.73	1.85
137.36	8.68	22.63	150.87	1.86
135.86	8.73	22.74	151.60	1.87

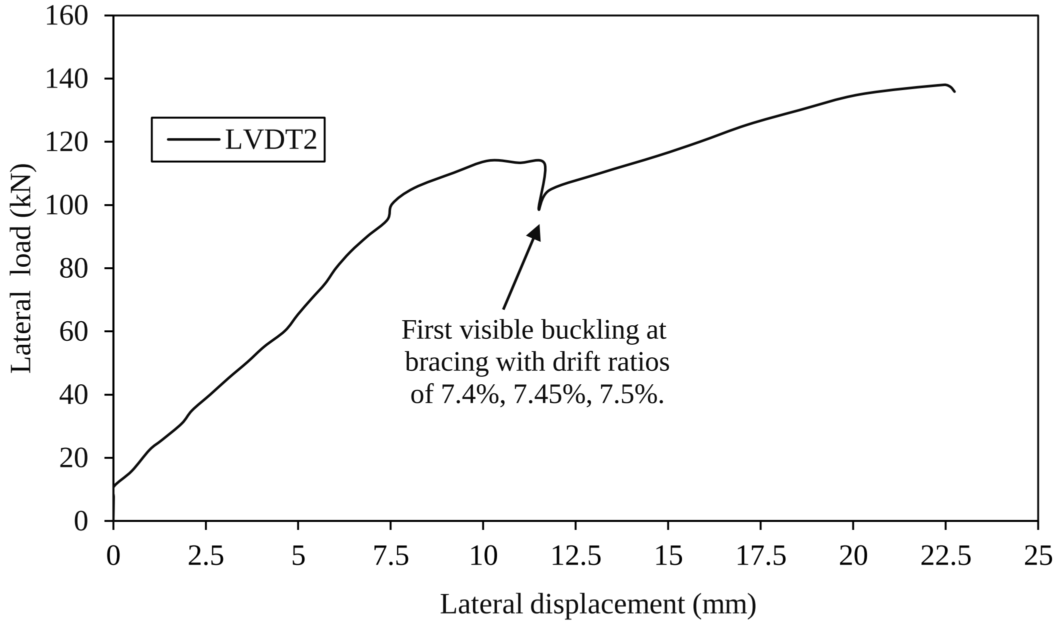


Fig. 4.6: Lateral load versus lateral displacement for test frame MIN-BR3.

During testing, in order to take pictures of significant damage to the frame elements, the displacements were paused at drift ratios of 7.77%, 7.67%, and 7.68% (Table 4.6). According to Figure 4.6 the frame was already failed at these drift ratios. Figure 4.7 (a,b,c) shows the test specimen MIN-BR3 under lateral loading tests. The location of buckling of the bracing was at drift ratios of 7.4%, 7.45%, 7.55%, where the starting of local buckling at the compression brace is evident by the sudden drop on the curve (Fig. 4.6). Figure 4.7 (b) shows the rotation at column and lateral torsion bucking of beam for the test frame MIN-BR3 at the displacement of 22.46 mm and load of 138.03 kN. The frame failed due to reduction in stiffness caused by yielding of column and spreading of plasticity due to the lateral force at column which is transferred to the beam.

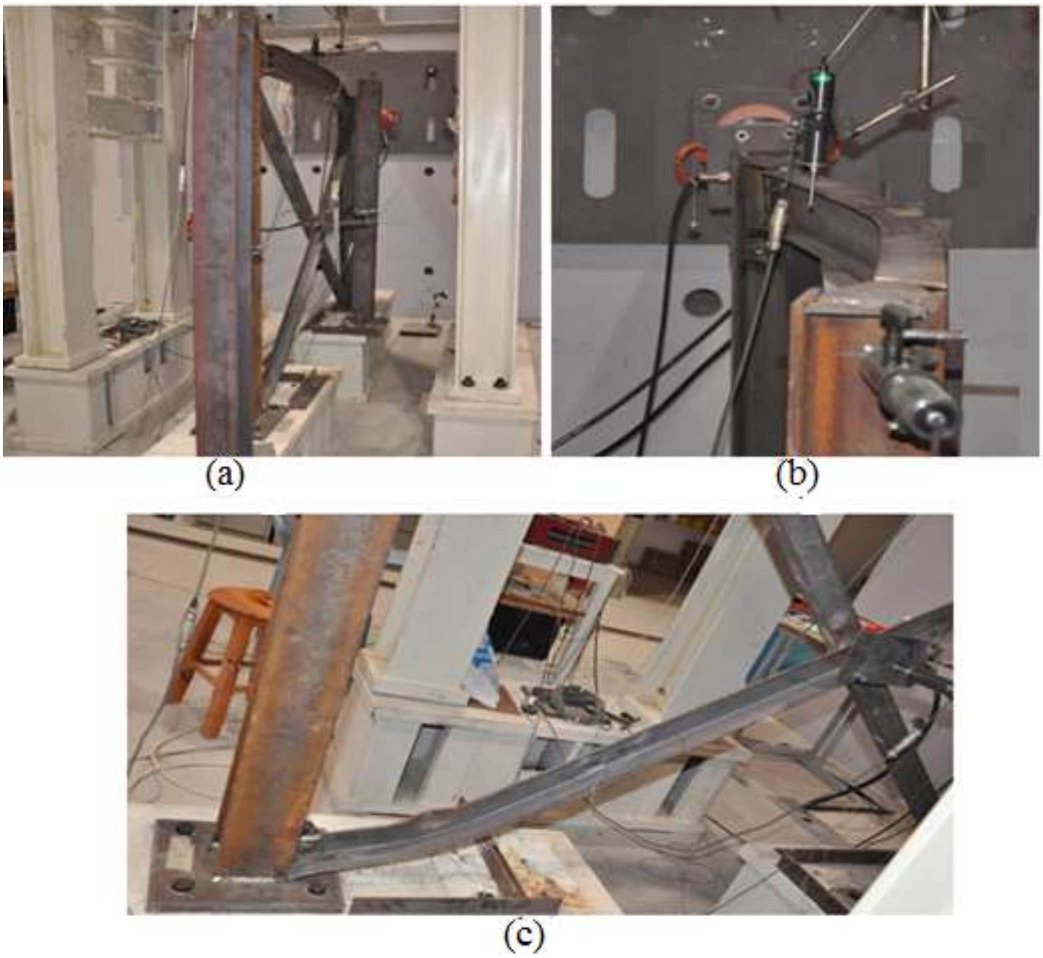


Figure 4.7: Test specimen MIN-BR3 under lateral loading (a) out-of-plane bending and local buckling of the compressive member of the cross-bracing, (b) out-of-plane bending of beam and (c) torsion of column and bucking at bracing.

4.3.4 Braced Frame with Infill Wall (MIN-BRIN4)

Lateral load was applied by a hydraulic jack with 1000KN capacity to the test setup. Figure 3.8 has locations of the LVDTs and the hydraulic jack. Readings from these measuring devices are shown in table 4.7. Rotation testing for (MIN-BRIN4) under gradual loading and LVDT results are shown in table 4.7.

Table 4.7: Displacements and rotations measured at various locations for the braced frame with infill wall MIN-BRIN4.

Lateral Load P KN	Displacement (Δ) LVDT1 (mm)	LVDT2 (mm)	Drift ratio (α) $\times 10^{-2}$ %	Rotation (ϴ) $\times 10^{-2}$ radians
0.00	0.00	0.00	0.00	0.00
1.17	0.02	0.01	0.67	0.00
8.67	0.07	0.09	0.60	0.00
15.67	0.14	0.20	13.33	0.08
22.50	0.23	0.34	22.67	0.15
27.67	0.32	0.49	32.67	0.23
33.01	0.44	0.70	46.67	0.35
37.34	0.63	1.02	68.00	0.53
40.01	0.73	1.22	81.33	0.66
45.34	0.87	1.50	100.00	0.84
50.51	1.03	1.82	121.33	1.06
55.34	1.15	2.07	138.00	1.23
60.18	1.29	2.39	159.33	1.47
65.01	1.42	2.64	176.00	1.63
70.51	1.56	2.93	195.33	1.83
75.68	1.72	3.23	215.33	2.01
80.18	1.88	3.53	235.33	2.21
85.68	2.12	3.98	265.33	2.49
90.18	2.27	4.28	285.33	2.68
95.52	2.54	4.79	319.33	3.01
110.36	3.21	6.11	407.33	3.87
117.52	3.63	7.03	468.67	4.53
125.19	4.04	7.86	524.00	5.10
135.19	4.71	9.26	617.33	6.10
140.19	5.20	10.33	688.67	6.84
150.53	5.85	11.73	782.00	7.84
160.53	6.31	12.74	849.33	8.57
170.53	6.92	14.17	944.67	9.67
180.20	7.38	15.15	1000.00	10.37
187.70	7.84	15.98	1065.33	10.85
193.04	8.23	16.66	1110.67	11.24
199.04	8.52	17.27	1151.33	11.67
197.87	8.62	17.51	1167.33	11.85
199.87	8.71	17.68	1178.67	11.97
195.87	8.99	18.43	1228.67	12.59
193.54	9.13	18.81	1254.00	12.91
190.54	9.29	19.34	1289.33	13.40

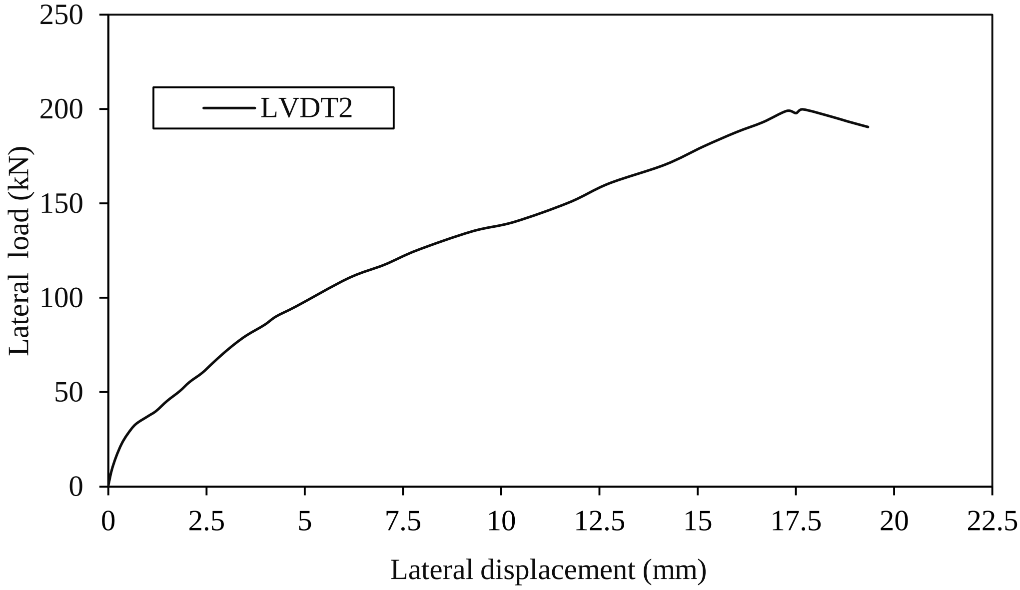


Figure 4.8: Lateral load versus lateral displacement behavior for test frame MIN-BRIN4.

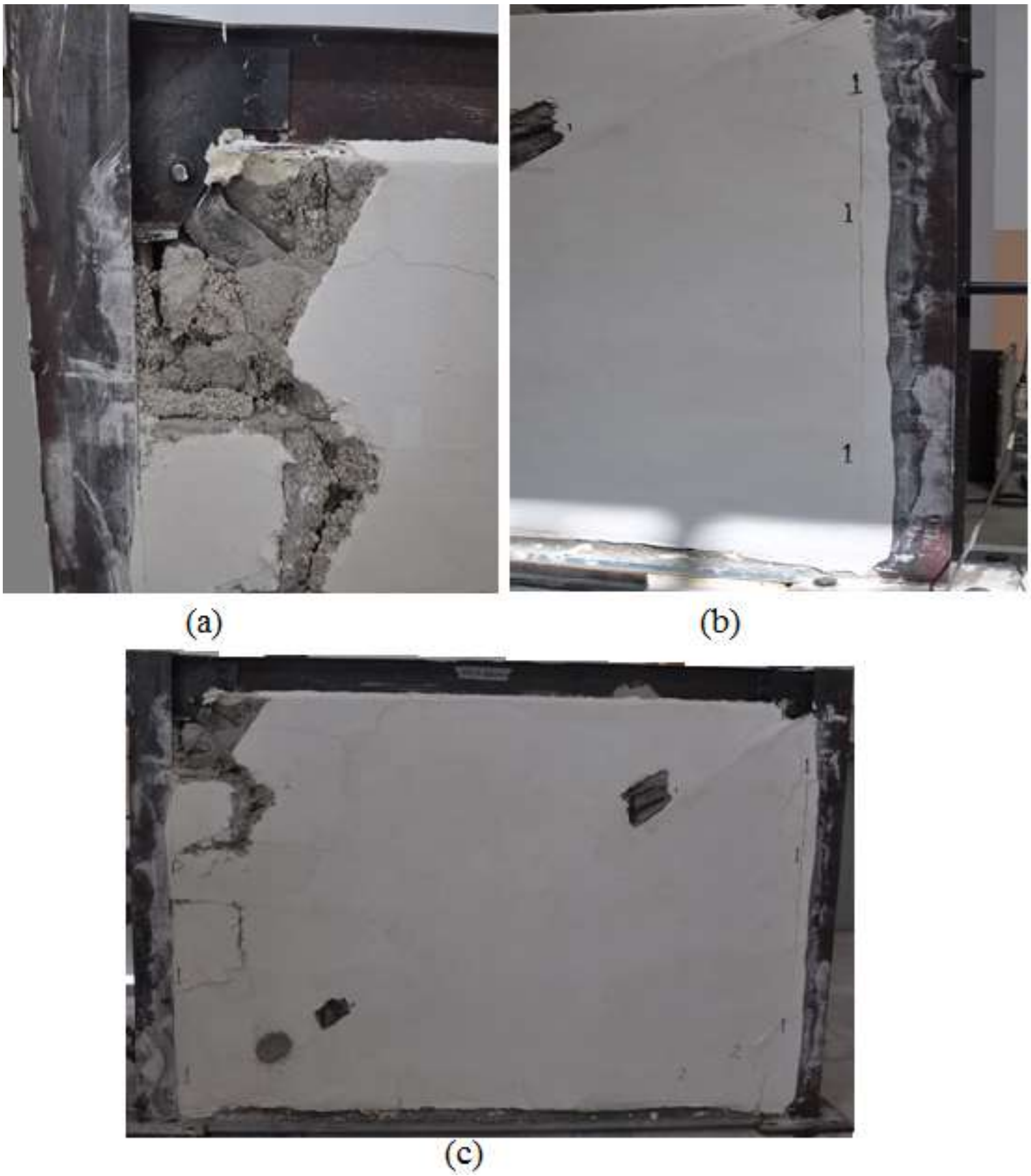


Figure 4.9: Failure modes, (a) compression corner crushing (b) vertical crack along the column height (c) front view of test frame MIN-BRIN4 after testing.

There are two main causes of the failure in MIN-BRIN4 frame. The first one is as shown in Fig. 4.9, crushing of wall at the top compressive corner and formation of longitudinal crack along the height of the column. Corner crushing was seen as the most frequent failure mode, for the test frame MIN-BRIN4 (Fig. 4.9 (a)). Figure 4.9 (b) shows the location of longitudinal crack which was initiated at an approximate load of 18.18 kN and displacement of 3.53 mm and then propagated along the column height. Second cause of failur, buckling of the bracing angle due to compressive load and this was observed as a sudden drop on the lateral load-displacement curve at around 199.87 kN lateral load with a corresponding displacement of about 17.5 mm (Fig. 4.8). Figure 4.8 shows the data obtained from the load cell and LVDT 2 as lateral load versus displacement, respectively. The elastic region stretches up to a lateral load of approximately 135.19 kN and lateral displacement of 9.26 mm.

4.4 Major Axis Frame Tests

In this test the beam ends were connected to column flanges, therefore, during the tests columns were subject to bending mainly around their major axis (Figure 4.10).

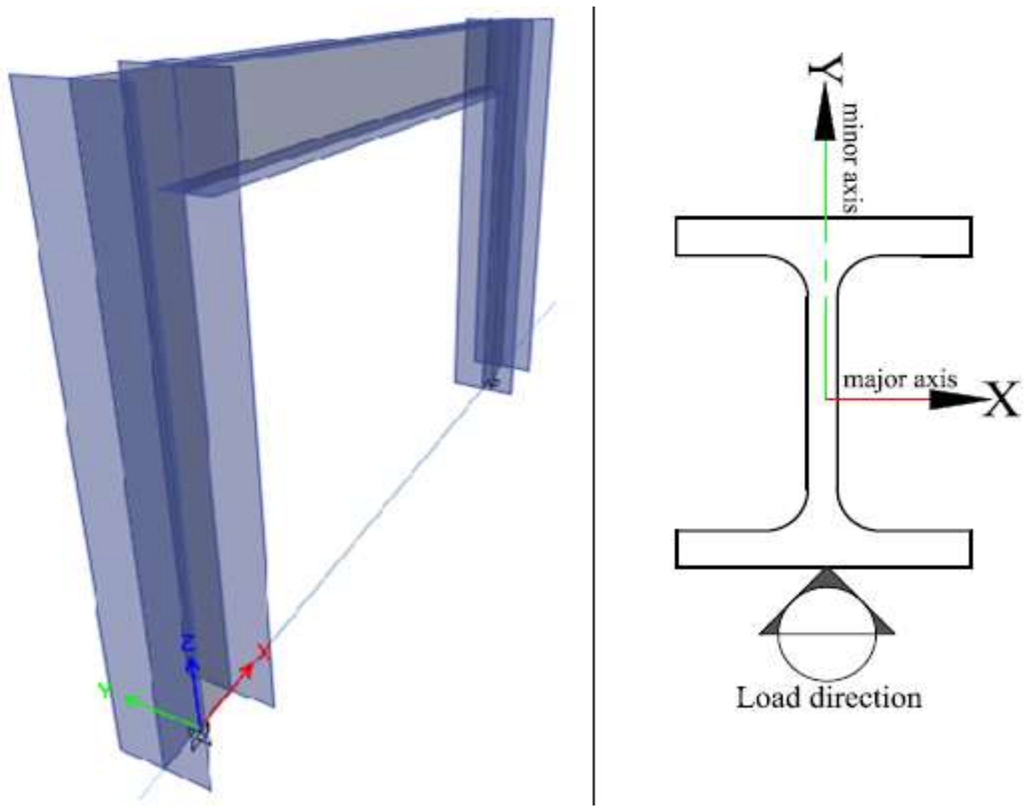


Figure 4.10: Major axis frame and load direction for HE120B column section

4.4.1 Moment Frame without Infill Wall (MAJ-MF5)

The test setup consisted of lateral load from a 1000KN capacity hydraulic jack. In figure 3.5, location of the LVDTs and hydraulic jack are given and readings obtained are shown in table 4.8. Table 4.8 shows rotation testing for (MAJ-MF5) under gradual loading and LVDT results.

Table 4.8: Displacements and rotations measured at various locations for the moment frame without infill wall (MAJ-MF5).

Lateral Load P	Displacement (Δ) LVDT1	LVDT2	Drift ratio (α)	Rotation (ϴ) $\times 10^{-3}$
kN	(mm)	(mm)	%	radians
0.00	0.00	0.00	0.00	0.00
1.33	0.20	0.45	0.03	0.34
5.00	0.58	1.32	0.09	0.99
10.17	1.16	2.64	0.18	2.00
16.17	1.70	3.70	0.25	2.67
20.00	2.18	4.75	0.32	3.43
26.01	2.99	6.48	0.43	4.65
30.84	3.69	8.06	0.54	5.83
35.67	4.38	9.47	0.63	6.79
40.01	5.06	10.97	0.73	7.89
45.01	5.82	12.60	0.84	9.04
50.01	6.56	14.18	0.95	10.16
55.18	7.35	15.88	1.06	11.37
60.18	8.02	17.32	1.15	12.40
65.01	8.82	19.07	1.27	13.67
70.18	9.74	21.12	1.41	15.17
75.01	10.68	23.28	1.55	16.81
80.18	11.89	26.11	1.74	18.97
85.01	13.18	29.07	1.94	21.19
90.35	14.96	33.13	2.21	24.23
95.02	16.77	37.08	2.47	27.10
100.19	19.40	42.81	2.85	31.21
105.69	31.57	69.30	4.62	50.31
104.19	31.70	69.61	4.64	50.55
102.85	31.77	69.65	4.64	50.51

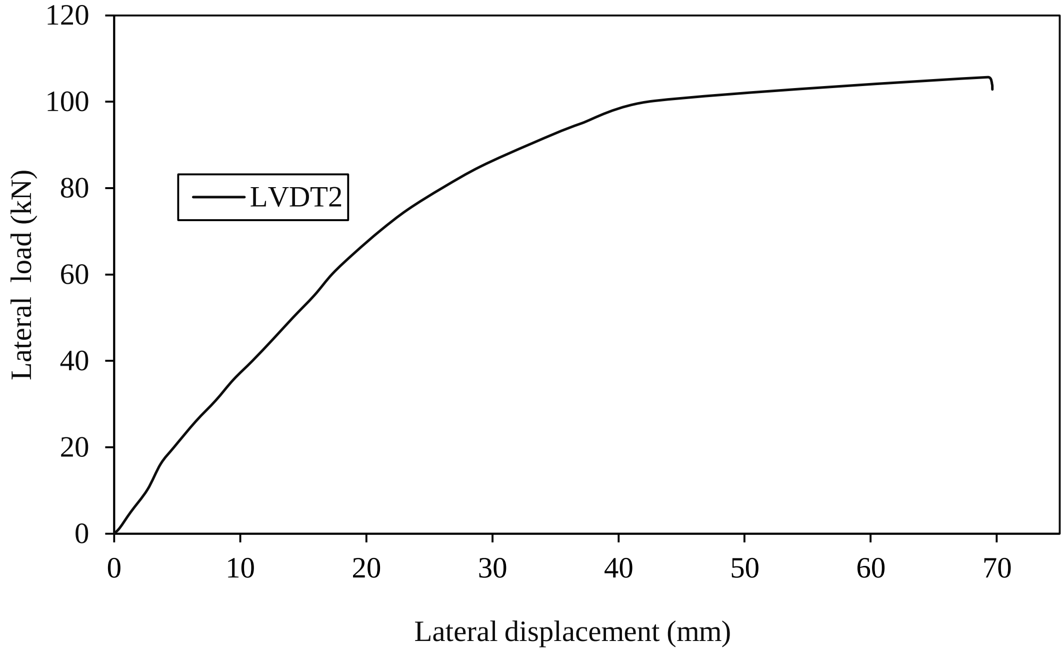
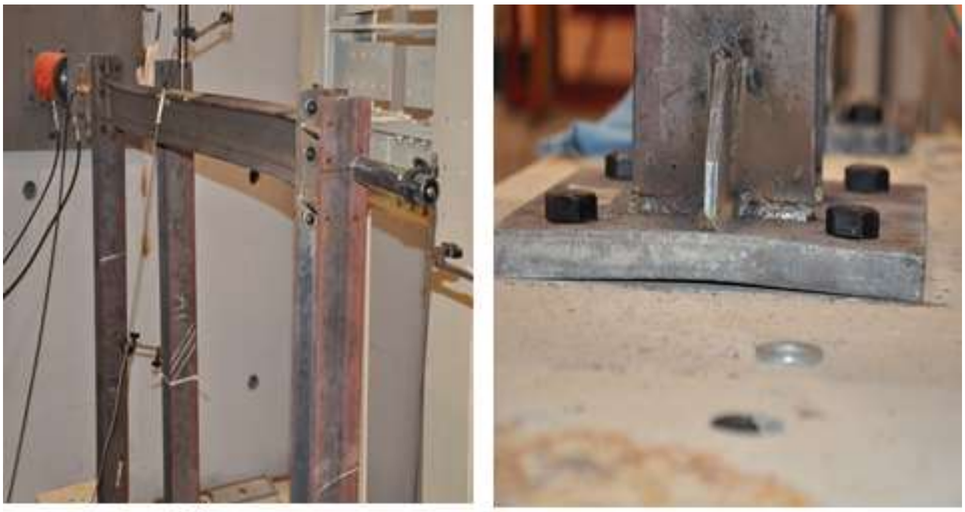


Fig. 4.11: Lateral load versus lateral displacement for test frame MAJ-MF5.

The test specimen MAJ-MF5 exhibited elastic behaviour up to a lateral load of approximately 60.18 kN and a lateral displacement of 17.32 mm (Fig. 4.11). The peak load and the corresponding displacement of test frame MAJ-MF5 were 105.69 kN and 69.3 mm, respectively. The graph vividly shows that plasticity starts at 60.18 kN up until the specimen fractures.

There was indication of lateral torsional buckling along the steel beam (Fig. 4.12 a) also there was bending at base plate (Fig. 4.12 b) but there was no damage in any of the steel beam-to-column connections of test frame MAJ-MF5. Yielding occurred at beam ends, at locations that are very close to the beam-to-column connections.



(a) (b)

Figure 4.12: View of the test frame MAJ-MF5 showing the (a) lateral torsional buckling of beam and (b) bending of column base plate.

4.4.2 Moment Frame with Infill Wall (MAJ-MFIN6)

A hydraulic jack of 1000 kN capacity applied a lateral load to the test setup. Locations of the LVDTs are included in the test setup and shown in figure 3.6 including the hydraulic jack. The readings from these measuring devices are given in table 4.9 and in this table are results of the LVDT and rotating testing for (MAJ-MFIN6) under gradual loading.

Table 4.9: Displacements and rotations measured at various locations for the moment frame with infill wall (MAJ-MFIN6).

Lateral Load P	Displacement (Δ) LVDT1	Displacement (Δ) LVDT2	Drift ratio (α) × 10^{-2}	Rotation (ɵ) × 10^{-3}
kN	(mm)	(mm)	%	radians
0.00	0.00	0.00	0.00	0.00
2.33	0.07	0.00	0.00	-0.09
9.17	0.39	0.00	0.00	-0.51
11.84	0.51	0.16	1.17	-0.47
14.67	0.95	1.13	7.53	0.25
20.34	1.33	1.97	13.13	0.85
25.51	1.64	2.64	17.60	1.33
30.51	2.04	3.52	23.47	1.98
35.17	2.30	4.02	26.80	2.29
40.17	2.58	4.58	30.53	2.67
45.01	3.02	5.43	36.20	3.22
50.18	3.55	6.37	42.47	3.76
54.01	4.02	7.01	46.73	3.99
53.34	4.04	7.06	47.07	4.03
55.51	4.31	7.61	50.73	4.40
58.85	4.62	8.10	54.00	4.65
57.18	4.99	8.49	56.60	4.67
57.84	6.36	10.85	72.33	5.99
59.01	7.05	12.15	81.00	6.81
62.35	7.92	13.89	92.60	7.97
65.68	8.32	14.69	97.93	8.49
67.51	8.66	15.40	102.67	8.99
70.01	8.96	16.09	107.27	9.51
69.85	9.03	16.26	108.40	9.64
75.01	9.54	17.48	116.53	10.59
77.52	10.15	18.82	125.47	11.57
80.18	10.47	19.62	130.80	12.20
85.18	11.07	20.96	139.73	13.19
90.02	13.30	25.97	173.13	16.90
95.02	14.23	27.95	186.33	18.29
93.52	14.27	28.02	186.80	18.33
92.52	14.27	28.00	186.67	18.31
95.85	14.40	28.31	188.73	18.55
100.02	14.88	29.39	195.93	19.35
103.69	15.87	31.83	212.20	21.29
101.69	15.96	32.10	214.00	21.52
99.19	16.01	32.17	214.47	21.55

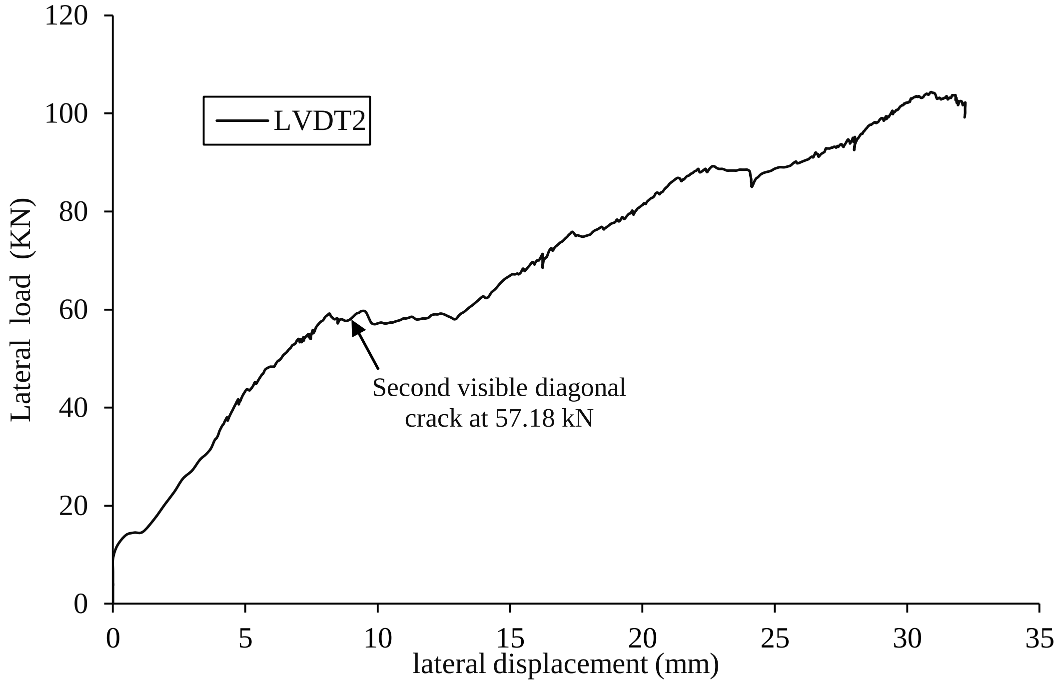


Fig. 4.13 Lateral load versus lateral displacement for test frame MAJ-MFIN6.

The test specimen MAJ-MFIN6 presented elastic behaviour up to approximately 57.18 kN and its corresponding displacement of 8.49 mm. Steel yielding occurred first in the column, and followed by the yielding of the column and beam. This behaviour can mainly be attributed to the interaction between the infill wall and the surrounding frame (Fig. 4.14 (a, b and c)). Figure 4.14 (a) and (b) shows the formation of diagonal cracking followed by the compressive corner wall crushing. The test frame stopped resisting the lateral load approximaitly at 103.69 kN with a corresponding displacement of 31.83 mm. The corner crushing for this test frame was observed well after the achievement of the ultimate load. While the diagonal cracking was like a behaviour bracing and the corner crushing was a type of compression failure of the diagonal bracing. This method treats the masonry infilled frame as a braced frame with the masonry infill replaced by an to resist the lateral loading equivalent diagonal strut acting in compression [30].

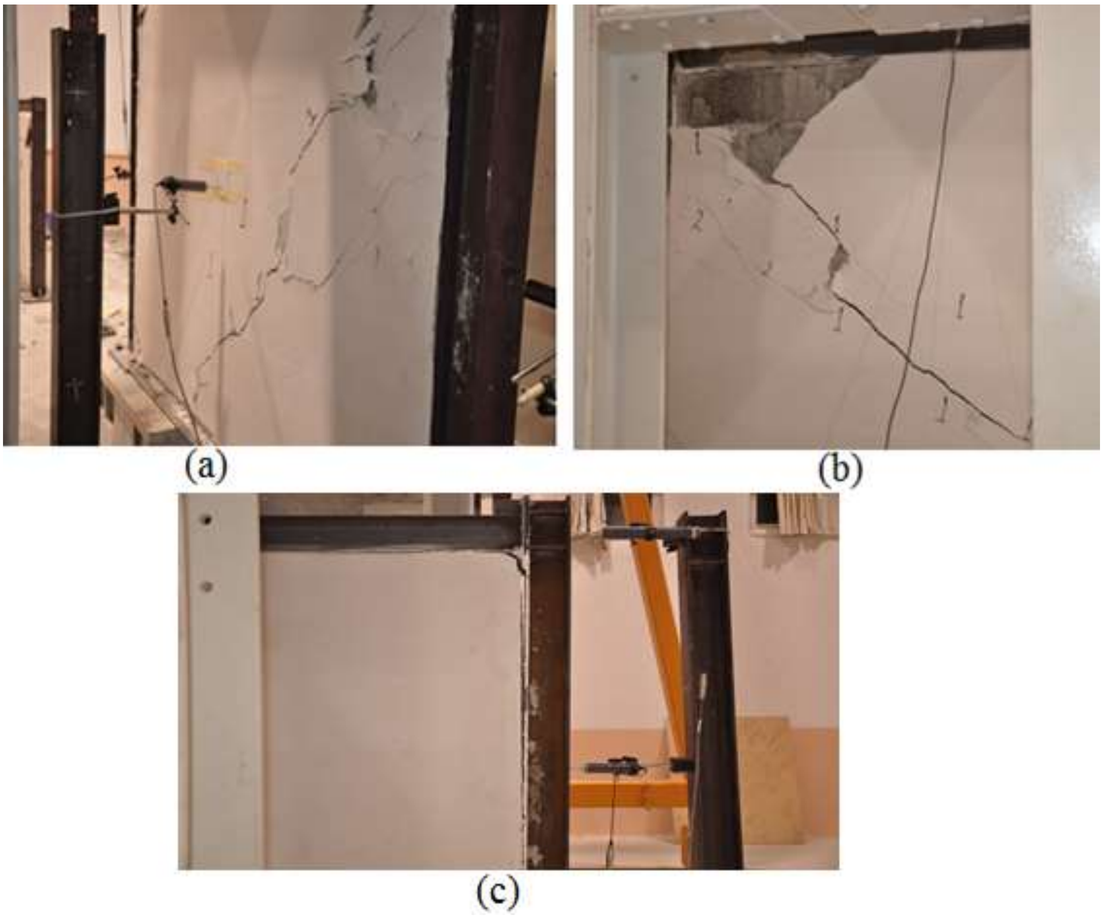


Figure 4.14: (a) Formation of the diagonal cracks followed by (b) compressive corner crushing of the infill wall and (b) damage due to infill wall-steel frame interaction for test frame MIN-MFIN6.

4.4.3 Braced Frame without Infill Wall (MAJ-BF7)

Lateral load was applied in the test setup by a hydraulic jack of 1000KN capacity. Figure 3.7 shows LVDTs locations and the hydraulic jack as well as readings from these measuring devices are given in Table 4.10. The results of LVDTs and rotation testing (MAJ-BF7) under gradually applied loading are shown in Table 4.10.

Table 4.10: Displacements and rotations measured at various locations for the braced frame without infill wall (MAJ-BF7).

Lateral Load P	Displacement (Δ) LVDT1	LVDT2	Drift ratio (α) $\times 10^{-2}$	Rotation (ɵ) $\times 10^{-3}$
kN	(mm)	(mm)	%	radians
0.00	0.00	0.00	0.00	0.00
3.17	0.07	0.25	1.67	0.25
8.84	0.24	0.66	4.40	0.57
15.34	0.46	1.18	7.87	0.96
21.84	0.71	1.86	12.40	1.53
28.84	1.15	3.18	21.20	2.71
34.34	1.31	3.49	23.27	2.91
40.17	1.58	4.22	28.13	3.53
45.84	1.78	4.81	32.07	4.05
50.34	1.92	5.16	34.40	4.33
55.18	2.04	5.52	36.80	4.64
60.35	2.17	5.84	38.93	4.89
70.18	2.39	6.38	42.53	5.33
75.85	2.50	6.63	44.20	5.51
80.18	2.57	6.79	45.27	5.63
85.68	2.64	6.95	46.33	5.75
91.02	2.76	7.14	47.60	5.85
97.02	2.84	7.36	49.07	6.03
91.18	2.69	6.92	46.13	5.65
118.52	3.04	7.48	49.87	5.93
125.19	3.08	7.60	50.67	6.03
130.86	3.14	7.78	51.87	6.19
135.53	3.21	7.96	53.07	6.34
140.36	3.27	8.10	54.00	6.45
145.69	3.32	8.18	54.53	6.49
150.03	3.36	8.26	55.07	6.53
155.19	3.42	8.34	55.60	6.56
160.70	3.50	8.43	56.20	6.58
165.53	3.58	8.58	57.20	6.67
170.53	3.70	8.78	58.53	6.78
180.20	3.87	9.05	60.33	6.91
178.70	3.83	8.90	59.33	6.76
175.20	3.76	8.68	57.87	6.57
170.53	3.69	8.50	56.67	6.42
166.70	3.62	8.29	55.27	6.23

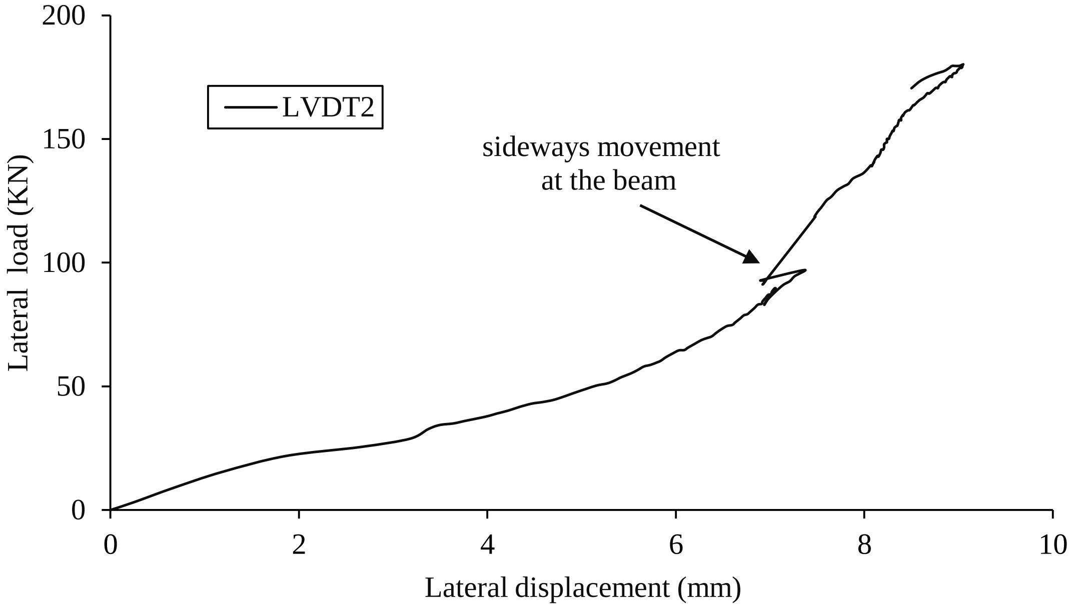


Figure 4.15 Lateral load versus lateral displacement test frame MAJ-BF7.

Fig. 4.16 (a,b,c) shows the test frame MAJ-BF7 under lateral loading tests. During the testing, in order to take some pictures of significant damage to the elements frame and its individual members, the displacements were paused at lateral load of 180.20 kN (Table 4.10) that mean the frame was failure. Figure 4.16 (a and c) shows the significant rotation of beam-to-column joints at a frame rotation of 6.91×10^{-3} rad and the lateral torsional buckling of beam, particularly close to the beam to column connection can be seen in Fig. 4.16 (b). The rotation of the beam-column joint was the main cause of the displacement and sideways movement of the frame members.

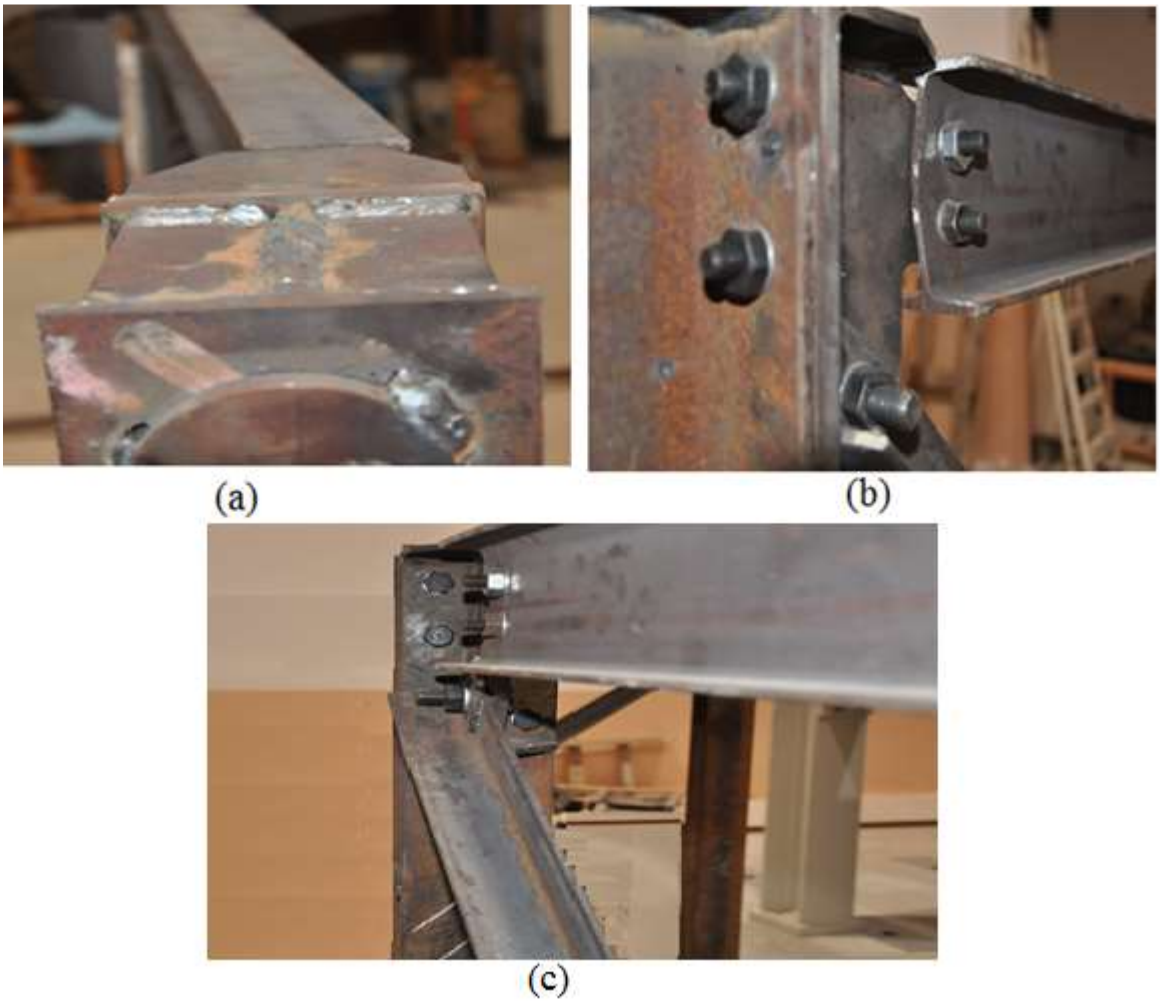


Figure 4.16: Various views of the test frame MAJ-BR7 showing the (a) damage of the lateral torsional buckling (b) damages at the beam-column connection (c) damages to bracing, beam, connection and torsion of the column members.

4.4.4 Braced Frame with Infill Wall (MAJ-BFIN8)

A hydraulic jack applied a lateral load to the test setup. The test setup along with locations of the LVDTs are shown in figure 3.7 as well as the hydraulic jack. Readings in table 4.11 are from these measuring devices and this table has results of LVDTs and rotating testing for (MAJ-BFIN8) under loading applied gradually.

Table 4.11: Displacements and rotations measured at various locations for the braced frame with infill wall (MAJ-BFIN8).

Lateral Load	Displacement (Δ)		Drift ratio (α)	Rotation (ϴ)
P	LVDT1	LVDT2	$\times 10^{-2}$	$\times 10^{-4}$
kN	(mm)	(mm)	%	radians
0.00	0.00	0.00	0.00	0.00
0.17	0.01	0.01	0.07	0.07
6.67	0.10	0.16	1.06	0.80
11.00	0.20	0.30	0.02	1.33
16.00	0.44	0.66	4.40	3.00
21.01	2.15	2.29	15.27	1.87
27.51	3.00	3.12	20.80	1.60
34.01	4.57	4.59	30.60	0.33
40.34	5.18	5.22	34.80	0.53
45.18	5.77	5.83	38.87	0.87
50.84	6.12	6.25	41.67	1.73
55.18	6.17	6.38	42.53	2.87
60.01	6.23	6.49	43.27	3.53
59.35	6.23	6.49	43.27	3.53
65.18	6.30	6.69	44.60	5.27
70.51	6.43	7.03	46.87	8.00
75.18	6.55	7.27	48.47	9.60
80.35	6.64	7.53	50.20	11.93
85.18	6.73	7.81	52.07	14.47
91.02	6.82	8.01	53.40	15.87
100.52	7.03	8.38	55.87	18.00
110.02	7.15	8.64	57.60	19.93
116.36	7.26	8.84	58.93	21.13
125.53	7.37	9.02	60.13	22.00
132.03	7.50	9.20	61.33	22.73
137.36	7.56	9.22	61.47	22.20
145.69	7.65	9.28	61.87	21.80
150.03	7.72	9.39	62.60	22.33
157.20	7.81	9.57	63.80	23.47
165.37	7.95	9.81	65.40	24.80
170.03	8.00	9.88	65.87	25.13
175.04	8.03	9.86	65.73	24.47
180.20	8.01	9.75	65.00	23.27
186.20	8.03	9.89	65.93	24.80
192.21	8.08	9.96	66.40	25.13
202.71	8.12	10.14	67.60	26.93
210.04	8.14	10.19	67.93	27.33
214.04	8.14	10.21	68.07	27.67
217.21	8.11	10.43	69.53	30.93
212.21	8.04	10.33	68.87	30.53
206.71	7.99	10.21	68.07	29.60

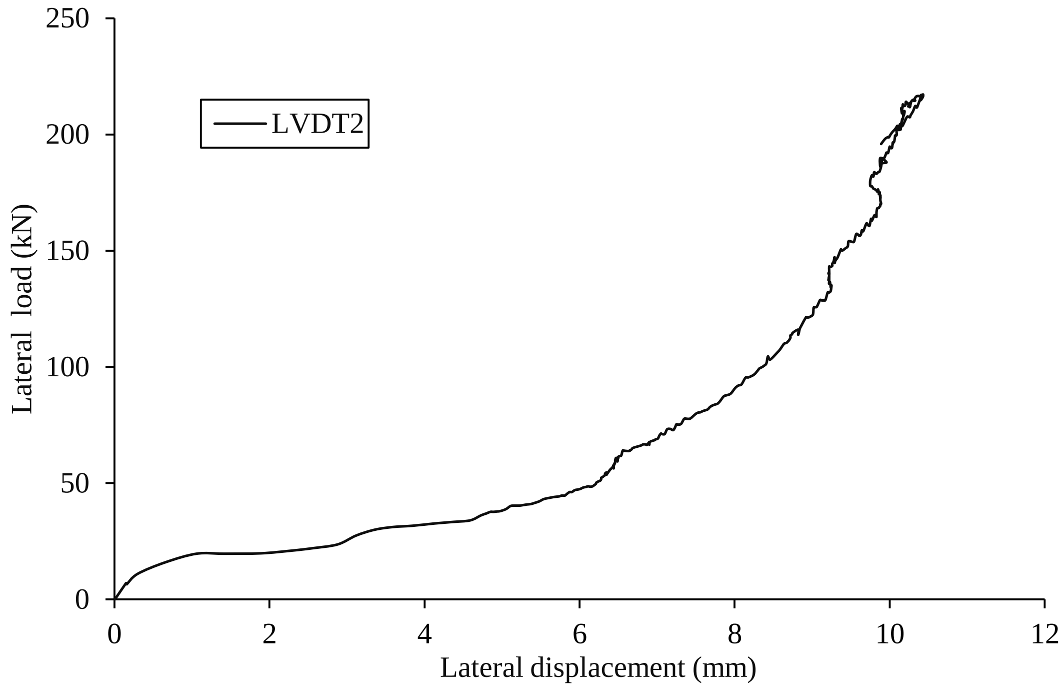


Figure 4.17: Lateral load versus lateral displacement test frame MAJ-BFIN8.

Fig 4.17 shows a relatively different load-displacement curve where instead of reaching a peak load and then decrease in load followed by failure it appears that the load achieved within the elastic part of the curve is about 7% of the ultimate load achieved. The test frame MAJ-BFIN8 presented elastic behaviour up to approximately 19.6 kN with a corresponding displacement of around 1.06mm. Then the test frame continued to resist the applied loading until it reached approximately 217.21 kN lateral load and 10.4 mm lateral displacement. After this point there was reduction in load and the frame failed to carry any more loading.

(a) (b)

Figure 4.18: Failure modes, (a) corner crushing due to compression (b) horizontal crack along the beam length of test frame MAJ-BFIN8 after testing.

4.5 Discussion of Test Results

The most important aim of this study is to examine the significance of masonry infill wall to the lateral strength and stiffness of moment and braced steel frame. The analyses of experimental results were carried out step by step to figure out the effect of various parameters on the behavior. These specifications are column orientation, test frames with and without infill wall. The discussions on test results are given in the following section.

4.5.1 Effect of Column Orientation on the Frame Action

Considering the frame to infill wall stiffness ratio results show that strong axis orientation of the steel columns resulted in an increase in the ultimate load carrying capacity of the test frame. Furthermore when the major axis test frame with infill wall is compared with the one having column orientation in the weak axis, the ultimate load capacity also higher.

4.5.1.1 Test Frames MAJ-MF5 and MIN-MF1

The effect of column orientation on the behaviour of test frame is further illustrated in Fig 4.19. As anticipated, the test frame with major axis column orientation exhibited markedly larger initial stiffness than those with minor axis column orientation but the latter attained greater ultimate displacement and ultimate lateral load, 20.79% and 38.18% higher, respectively. This indicates that the weaker test frame with lower ultimate lateral load obtaining greater ductility than the stronger test frame.

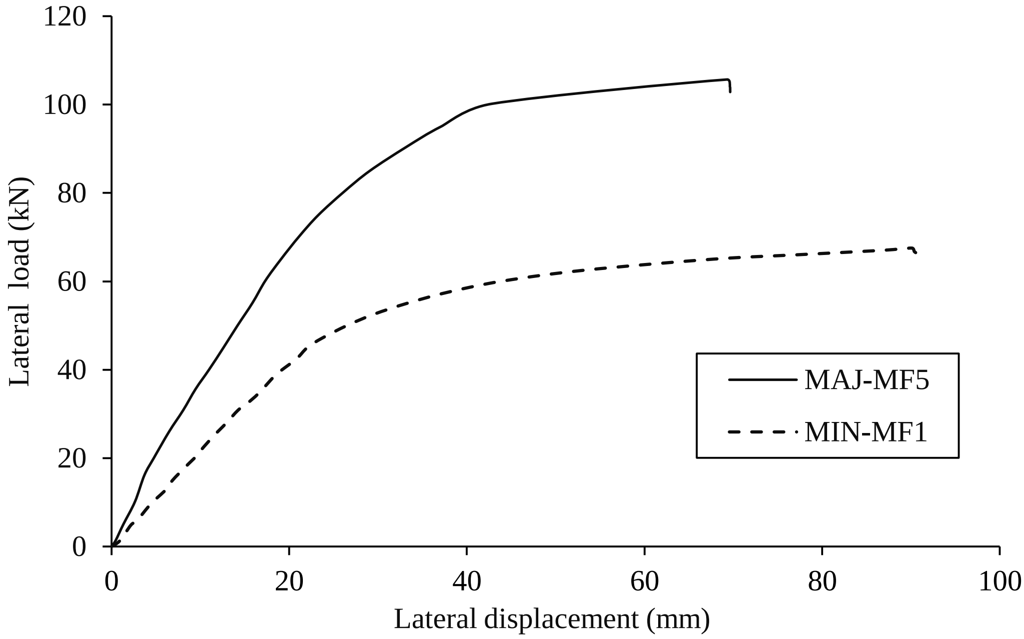


Figure 4.19: Comparison of load versus lateral displacement response for test frame MAJ-MF5 and MIN-MF1 with different column axis orientation.

4.5.1.2 Test Frames MAJ-MFIN6 and MIN-MFIN2

The test frames with minor axis column orientation had developed diagonal cracks before reaching the ultimate load. This can also be seen as a sudden drop on the load versus displacement curve and then followed by a continuing increase in the load until failure (Fig 4.20).

The effect of the column orientation is further illustrated in Fig 4.20. As expected, the test frame with infill wall and major axis column orientation exhibited notably larger initial stiffness than those with minor axis column orientation. Moreover, the latter attained 15.86% higher ultimate displacement and 49.68% higher ultimate lateral load.

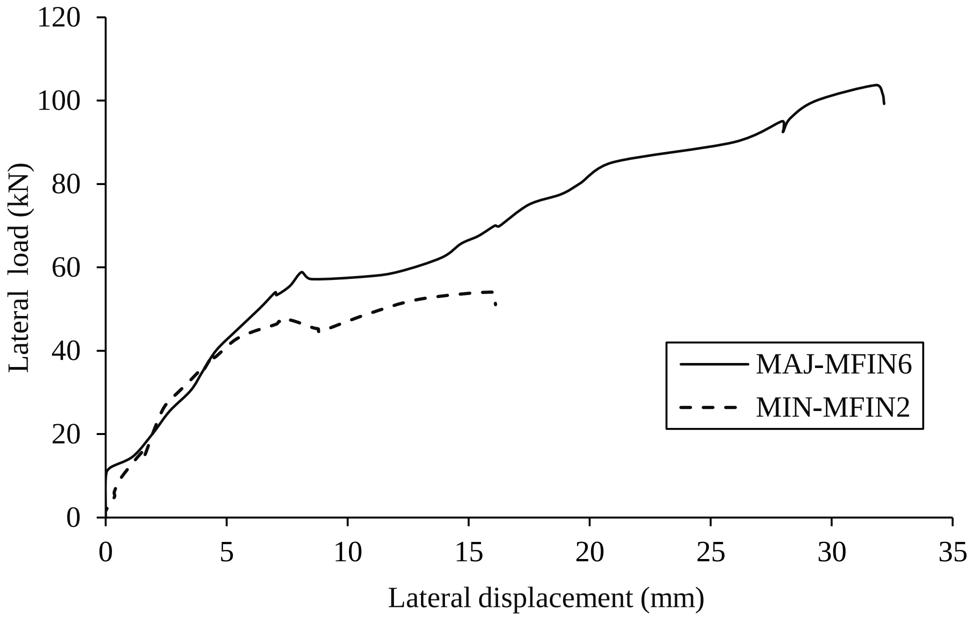


Figure 4.20: Comparison of load versus lateral displacement response for test frames MAJ-MFIN6 and MIN-MFIN2 with different column axis orientation.

4.5.1.3 Test Frames MIN-BR3 and MAJ-BF7

The effect of the column orientation is further illustrated in Fig 4.21. The test frame with major axis column orientation achieved higher initial stiffness than those with minor axis column orientation but the latter attained 13.41% higher ultimate displacement and 42.17% higher ultimate lateral load. This indicates that the weaker frame achieved less lateral load at ultimate level but on the other hand this was compensated by achieving higher ductility for the test frame.

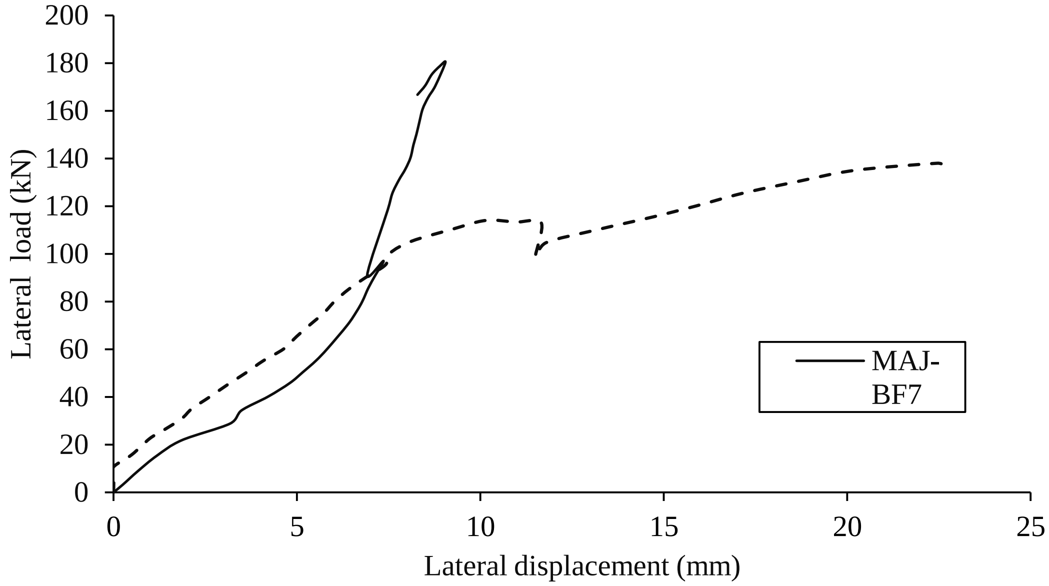


Figure 4.21: Comparison of load versus lateral displacement for the test frames MIN-BR3 and MAJ-BF7 with different column axis orientation.

4.5.1.4 Test Frame MIN-BRIN4 and MAJ-BFIN8

These test frames followed similar initial stiffness trend as in the other test frames in earlier sections (Fig. 4.22). Hence the frame with major axis column orientation attained 7.47% higher ultimate displacement and 18.17% higher ultimate lateral load

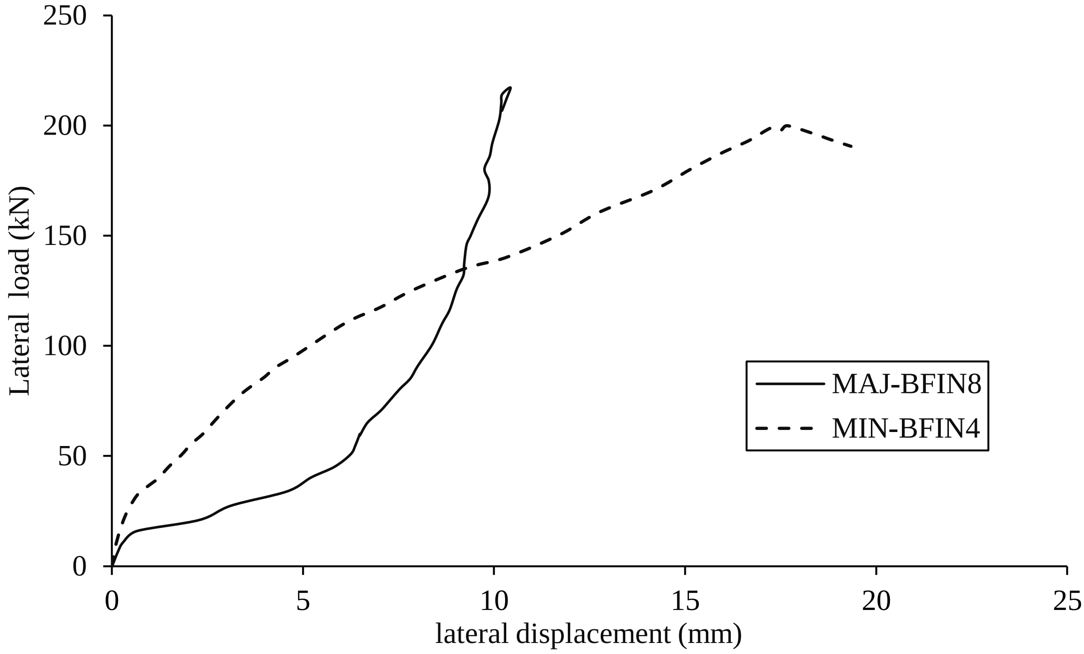


Figure 4.22: Comparison of load versus lateral displacement for test frames MIN-BRIN4 and MAJ-BFIN8 with different column axis orientation.

4.5.2 Effect of Infill Wall on the Frame Action

In the case of the frame-to-infill stiffness ratio as reflected in the frame systems with and without BIMs blocks, in this cases will be shown the contribution of BIMs block on steel frame.

Fig. 4.23 compares the lateral load versus lateral displacement curves of the moment frame with and without infill walls. It is very clear that the elastic stiffness of test frame MAJ-MF5 is considerably higher than the other three test frames. The normalized drift ratio at peak load of test frame MIN-MF1 is about 6.03% higher than the other three test frames (Table 4.4). The failure load of test frame MAJ-MFIN6 was 103.69 kN and this was higher than of the other three test frames. The contribution of infill wall to develop the stiffness of frame to resist the lateral load of the specimens of minor axis, moment frame tests and major axis frame tests is that when the infill wall is used its shows the highest stiffness that represented, MAJ-MFIN6, MAJ-MF5, MIN-MF1, and MIN-MFIN2 specimens as shown in Fig. 4.23.

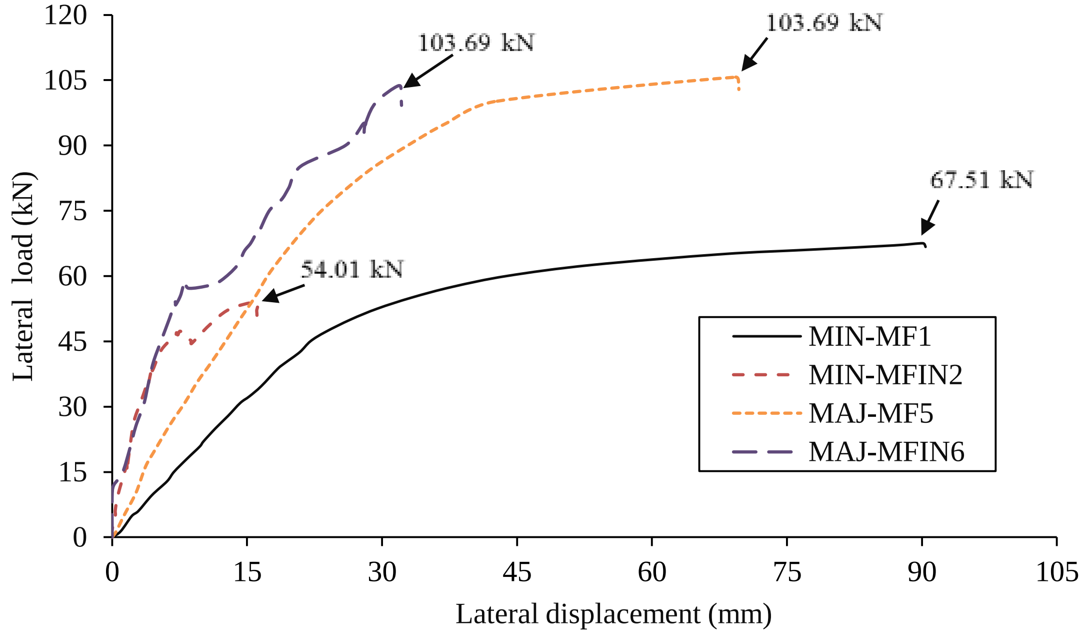


Figure 4.23: Comparison of the behavior (infill wall contribution, elastic stiffness, stiffness lateral load and stiffness displacement) of moment frame system with and without infill wall.

Figure 4.24 compares the braced frame with and without infill wall. When lateral load versus lateral displacement curve test frame is considered the elastic stiffness of test frame MAJ-MF5 was much higher than the other three test frames. The drift ratio at the peak load for test frame MIN-BR3 is higher when compared to the other three specimens, about 149.73×10^{-2} %, as listed in Table 4.8. The failure load of test frame MAJ-MFIN6 was 217.21 kN and this was higher than the other three test frames. When the stiffness of the frame to resist the lateral load is acting on it with minor axis frame tests and major axis frame tests it shows the highest stiffness of frames, MAJ-BRIN8, MIN-BRIN4, MAJ-BR7 and MIN-BR3. Finally, the contribution of infill wall with braced test frame achieved more lateral load stiffness and less lateral displacement stiffness of the behavior braced frame as shown in Fig. 4.24.

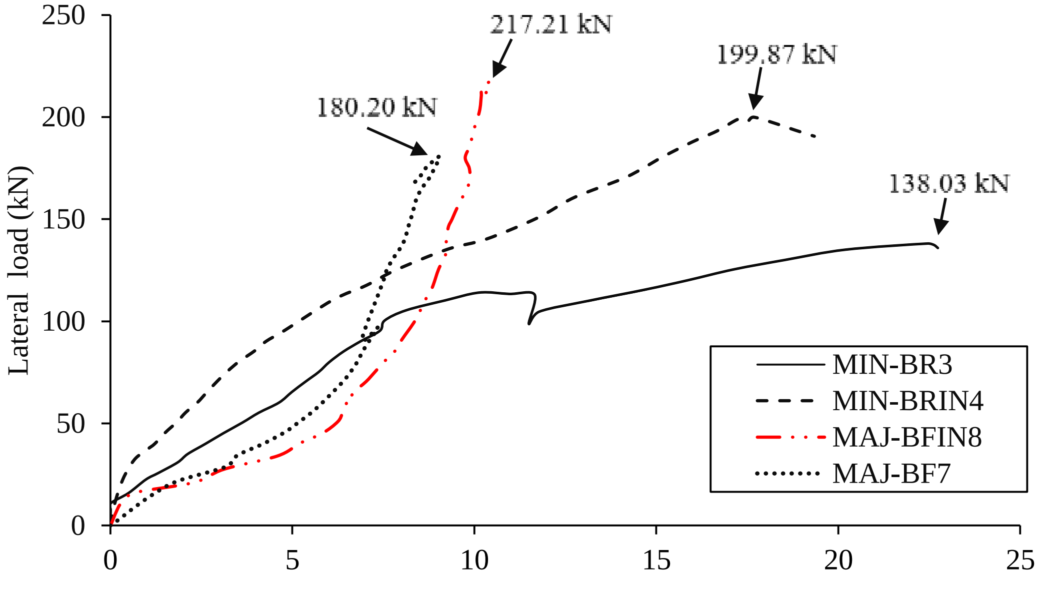


Figure 4.24: Comparison of behavior (infill wall contribution to elastic stiffness, lateral load and stiffness, stiffness displacement) of moment frame system with and without BIMs block.

Chapter 5

CONCLUSION AND RECOMMEDDATIONS

5.1 Conclusion

Four moment and four braced steel frames with and without BIMs block infilled walls were designed, fabricated, constructed and tested to investigate the lateral load resistance capacity. Another aim was to examine the impact of BIMs blocks infill wall to the lateral load resistance capacity of frames. Eight half-scale frames were constructed by using two steel columns and one beam member. Equal leg angles were used as cross bracing for the braced frames only. Parameters considered for discussing results and conclusions included lateral displacement stiffness, drift ratio, and the damaged shapes of steel members and the infill wall. The following are the conclusions drawn for this study.

- Infill walls constructed of BIMs blocks can be safely utilized to increase the stiffness of structural frames, resistance to lateral loads and to limit the frame deflections, especially for frames with major axis column.

- From the results of experimental test, it was observed that due to their high stiffness, the infilled frames experienced less damage than the moment and braced frame without infill wall.

- The lateral load versus lateral displacement curves for tested frames with infill walls showed weaving behavior while graphs of tested frames without infill walls (ductile systems) show softening or hardening behavior.

- The tested moment frames MIN-MFIN2 and MAJ-MFIN6 had diagonal compression cracks in the infill wall. Therefore, the masonry infill wall acted as an equivalent compression strut, like a diagonal compression brace to resist the lateral loading.

- Corner crushing due to compression was the dominant failure mode for braced test frames MIN-BRIN4 and MAJ-BFIN8 with infill wall. The use of infill wall with braced frame made the bracing more stiff and prolonged its buckling therefore increased the stiffness of the braced frame agains lateral load.

- Overall the bracing frame with infill wall behaves better than the moment frame with infill wall which is a result of the fact that the bracing frame with infill wall displays less damage of members and it shows more stiffness when subjected to lateral load.

- Specimens with major axis column orientation exhibited considerably larger initial stiffness and ultimate lateral load than those with minor axis column orientation but the latter attained greater ductility.

- Furthermore, considering the moment frame, for both major axis and minor axis without infill wall displayed higher ductility as can be seen on the graphs with high displacements. As for the braced frames, higher ductility was observed in the column minor axis braced frame without infill wall and the column major axis braced frame with infill wall.

- The lateral displacements of the frames are greatly reduced by the use of major axis column orientation when compared to minor axis column orientation. As a result, it can be concluded that use major axis column orientation leads to reduction in lateral frame displacements when compared to using minor axis column orientation.

- The lateral stiffness of the braced frame with and without infill wall increased and the lateral displacement decreased significantly when compared to moment frame with and without infill wall.

- The lateral displacements of the frames with and without infill walls are greatly reduced by the using bracing frame than using moment frame As a result, it can be concluded that bracing system has more influence on the lateral displacement. However, the elastic region of the moment frame more higher than the elastic region of bracing frame.

5.2 Recommendations

Based on the research presented in this thesis, the following are the recommendations for future work:

- In the research, the behavior of moment and braced frame with and without infilled wall subjected to lateral load has been studied. However, in practice, different types of bracing as V-bracing, Z-bracing, etc. can be used. The difference in types of bracing is expected to have influence on the overall frame beahviour. Therefore, similar work can be carried out by using different types of common bracing systems.

- Allowing some gap between the steel frmae and the infill wall is expected to give more ductile behavior and may be even more stiffness to the frame. This gap will allow more rotation for the column and beam for frames with infill walls. So this case can be further investigated to find out possible benefits for steel frames subjected to lateral loads.

- Use of the beam at the bottom as well will increase the stiffness in terms of behavior in resisting the lateral loading. In particular most construction use a beam and the top and also the bottom except at the ground level.

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APPENDICES

Appendix A: Details of Test Frames

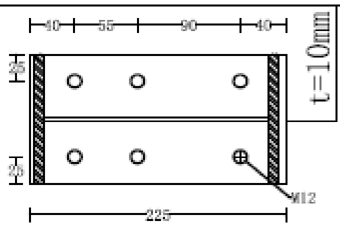
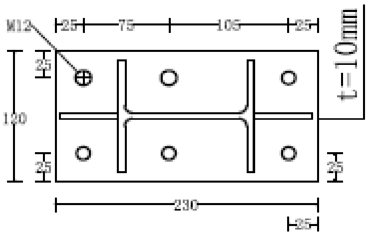
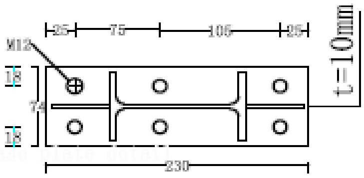
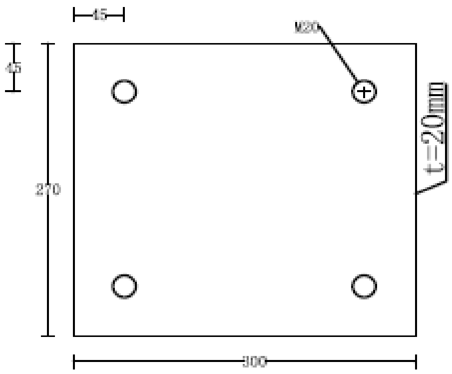
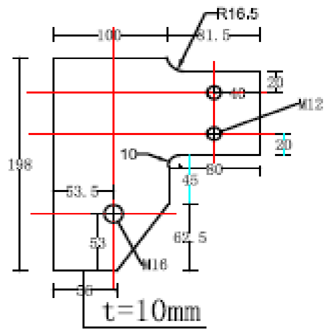
Serial No	No	Grade	Details
15	4	S 275	
16	4	S 275	
17	4	S 275	
18	16	S 275	
19	4	275	

Figure A.1: Stiffener, plate and base plate detail.

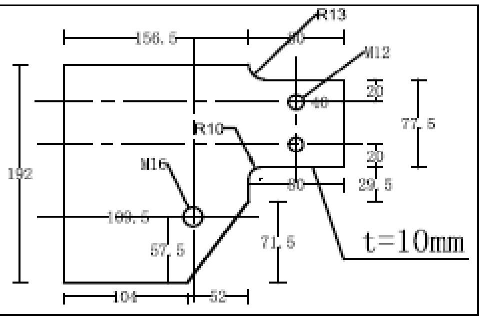
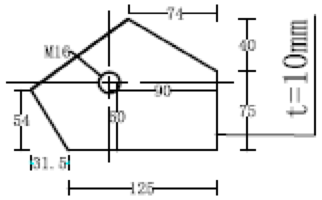
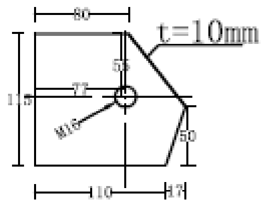
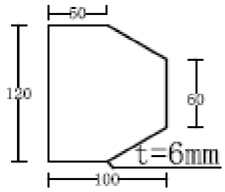
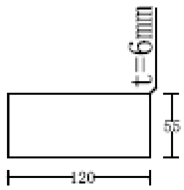
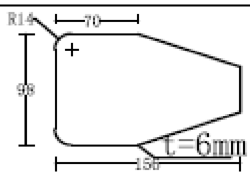
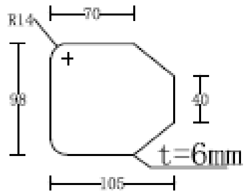
20	4	275	
21	4	275	
22	4	275	
23	4	S 275	
24	4	S 275	
25	4	S 275	
26	4	S 275	

Figure A.2: (Cont.)

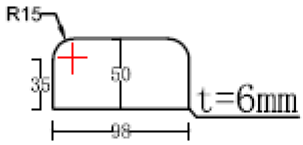
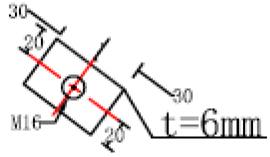
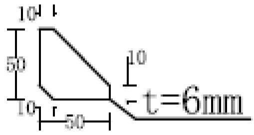
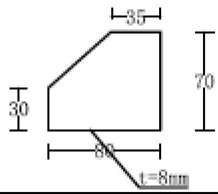
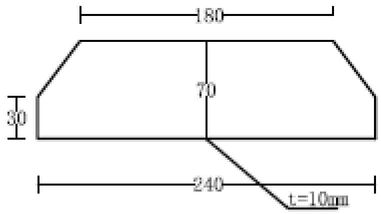
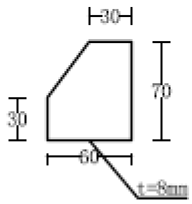
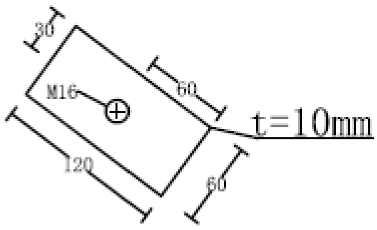
27	58	S 235	
28	16	S 235	
29	16	S 235	
30	12	S 235	
31	8	S 235	
32	16	S 235	
33	4	S 235	

Figure A.3: (Cont.)

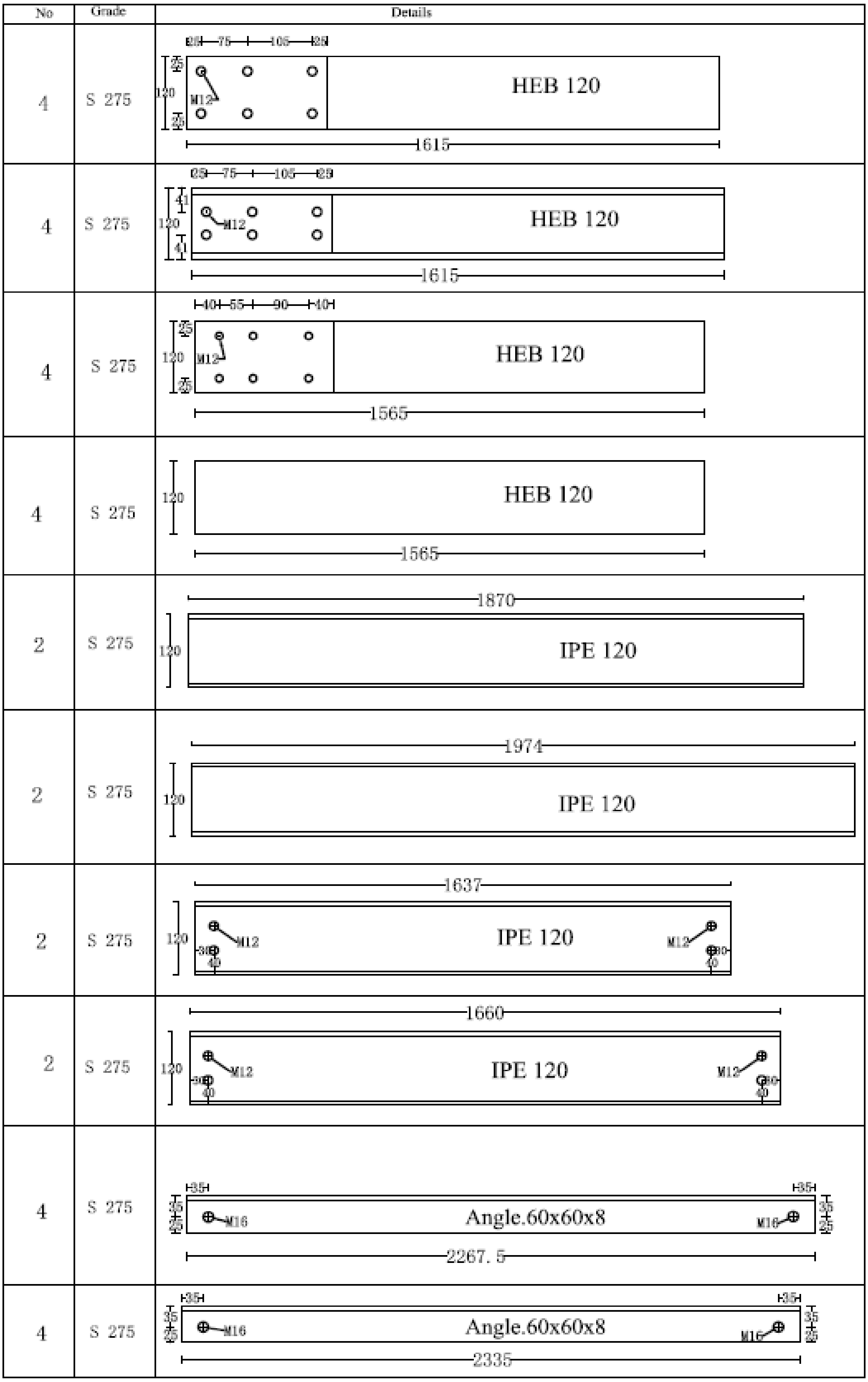


Figure A.4: Dimensions and holes detail of column, beam and bracing.

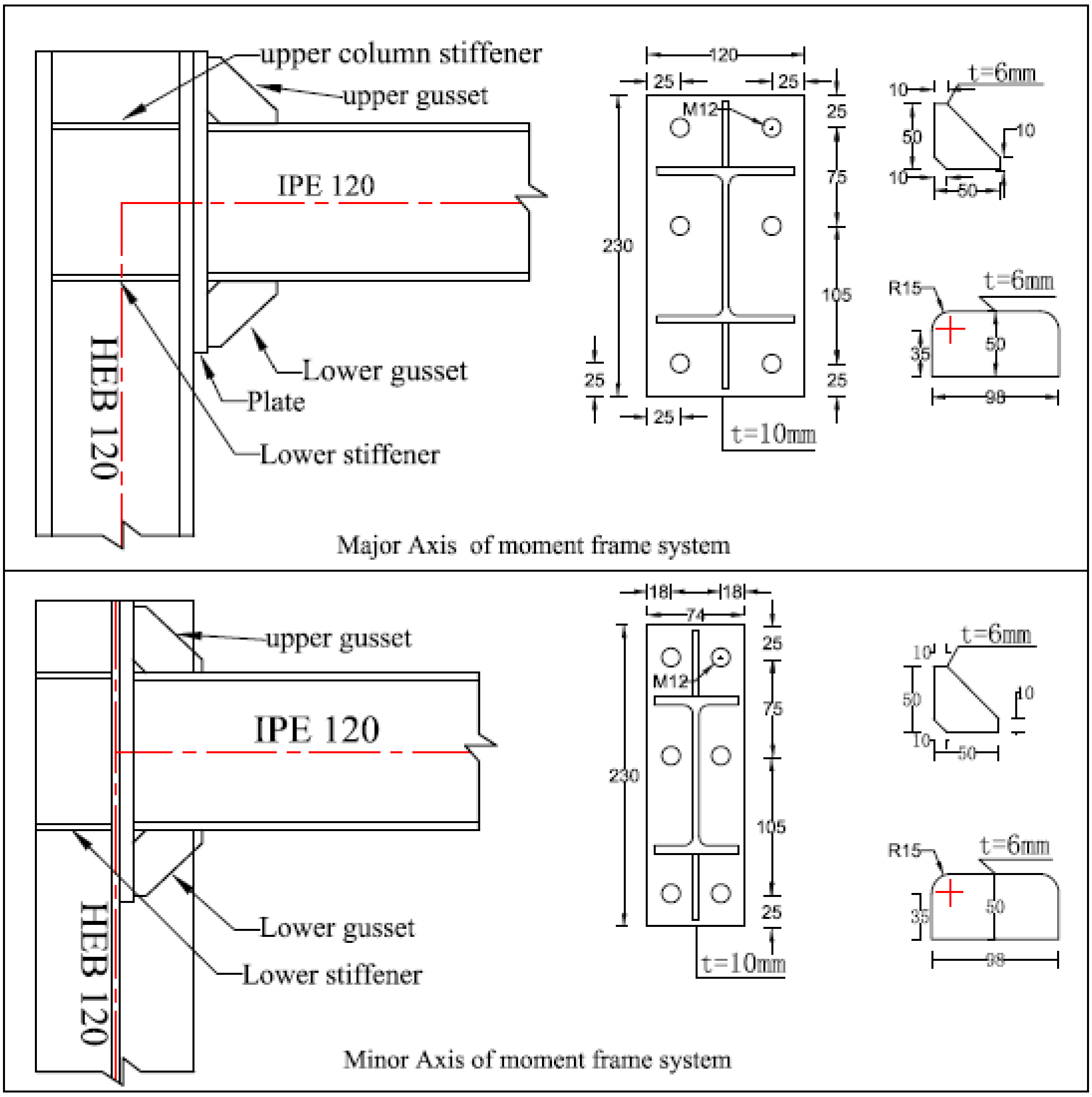


Figure A.5: Moment frame system, beam and column connection details.

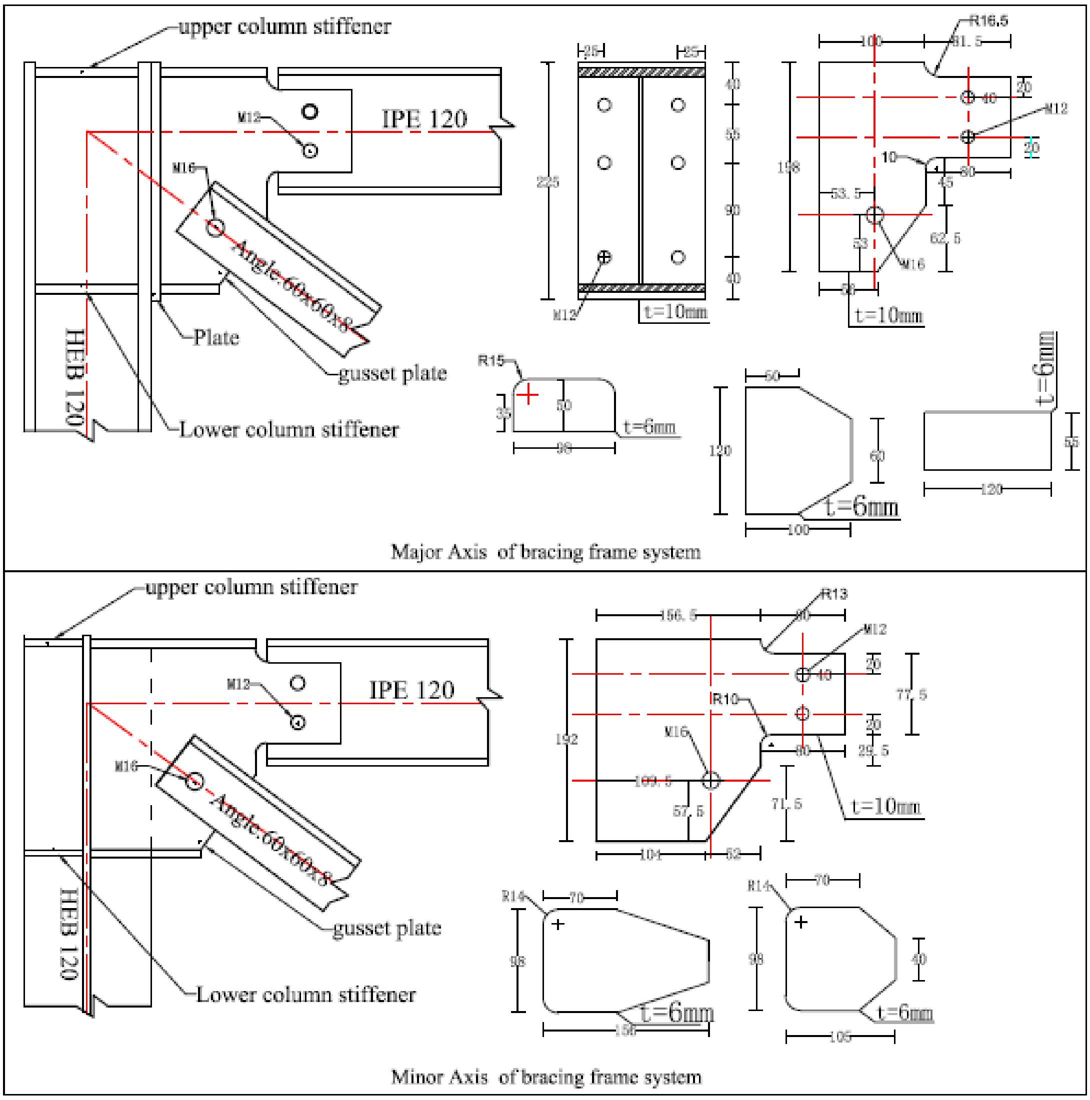


Figure A.6: Braced frame system, beam, column and bracing connection details.

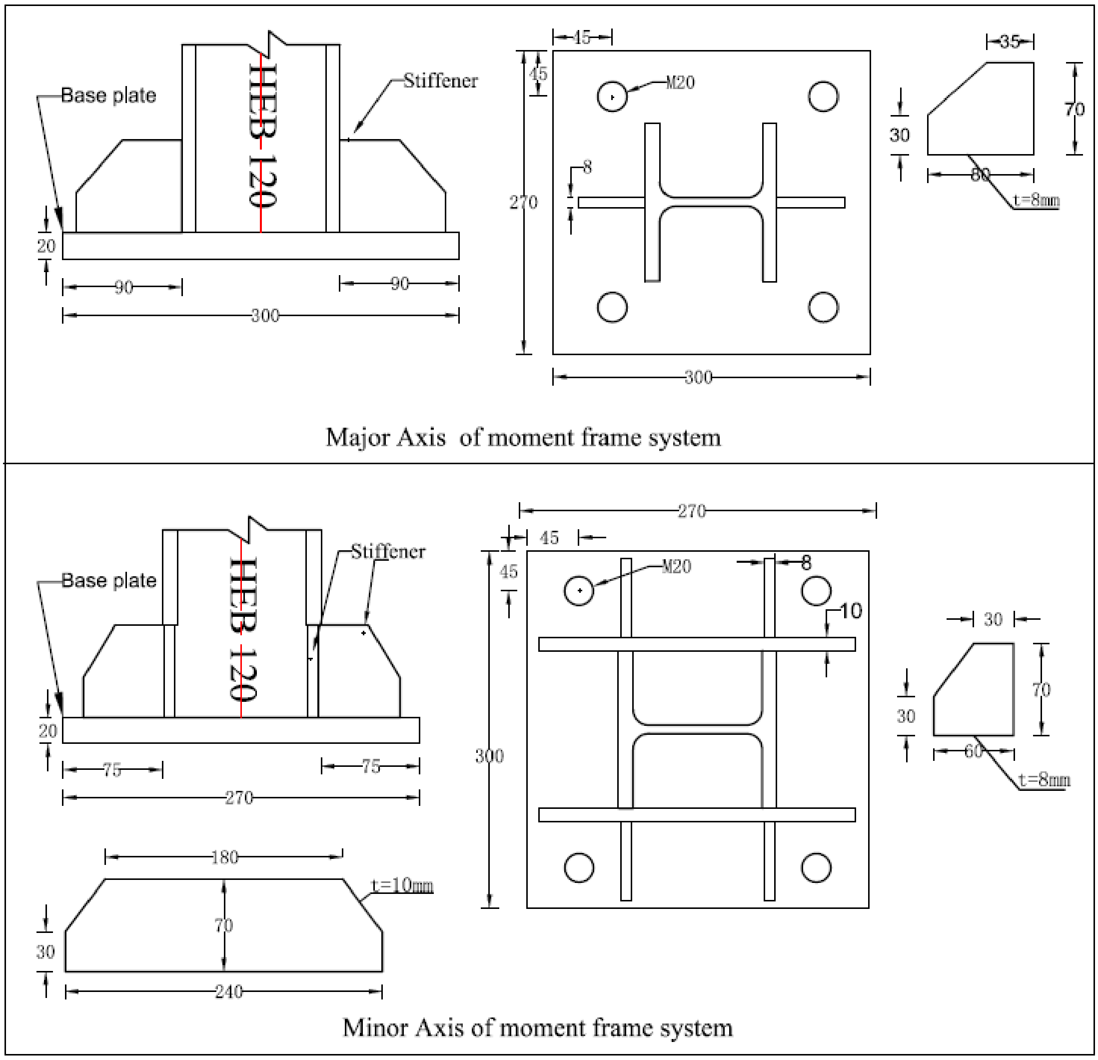


Figure A.7: Moment frame system, base plate and column connection details.

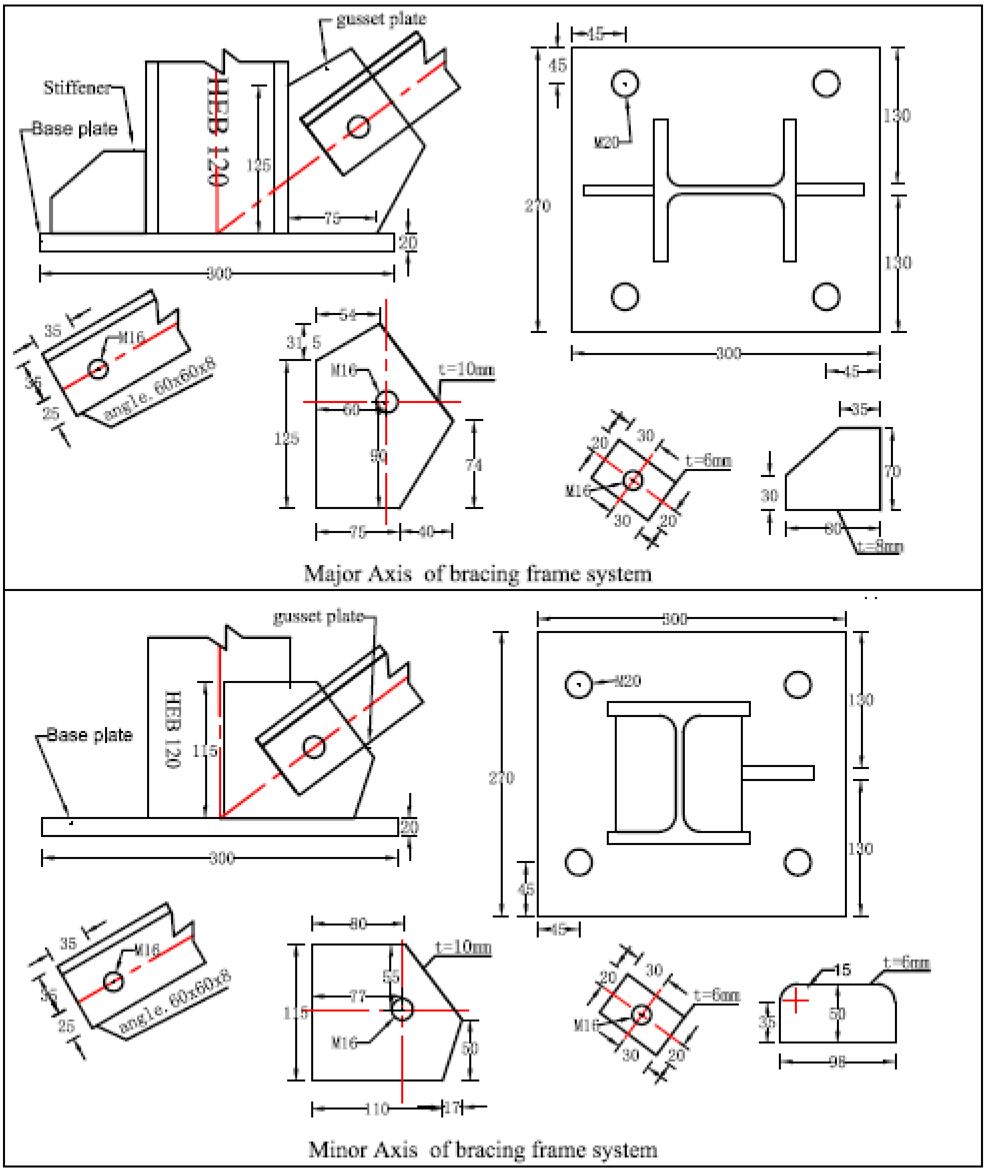


Figure A.8: Braced frame system, base plate and column connection details.

Appendix B: Frame Loading and the Support Frames

Figure B.9: Set up of hydraulic jack with a capacity of 1000 kN.

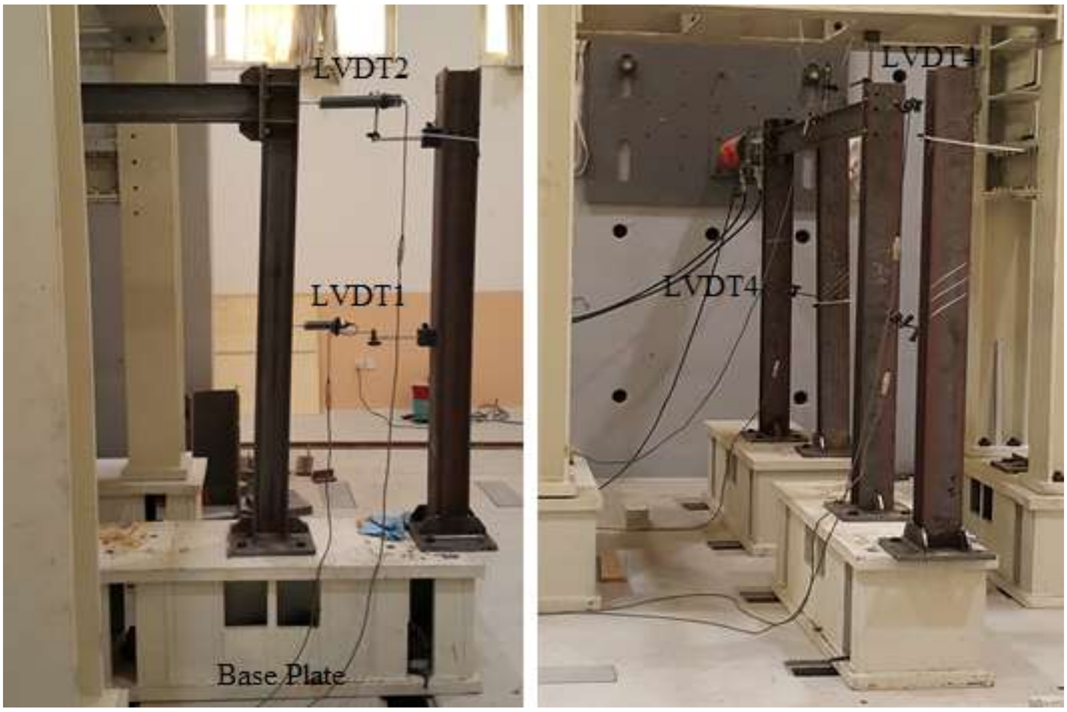


Figure B.10: Set up of test frame LVDTs.


Figure B.11 Set up of test frame MIN-MF1.


Figure B.12: Set up of test frame MIN-MFIN2.


Figure B. 13: Set up of test frame MAJ-MF5.

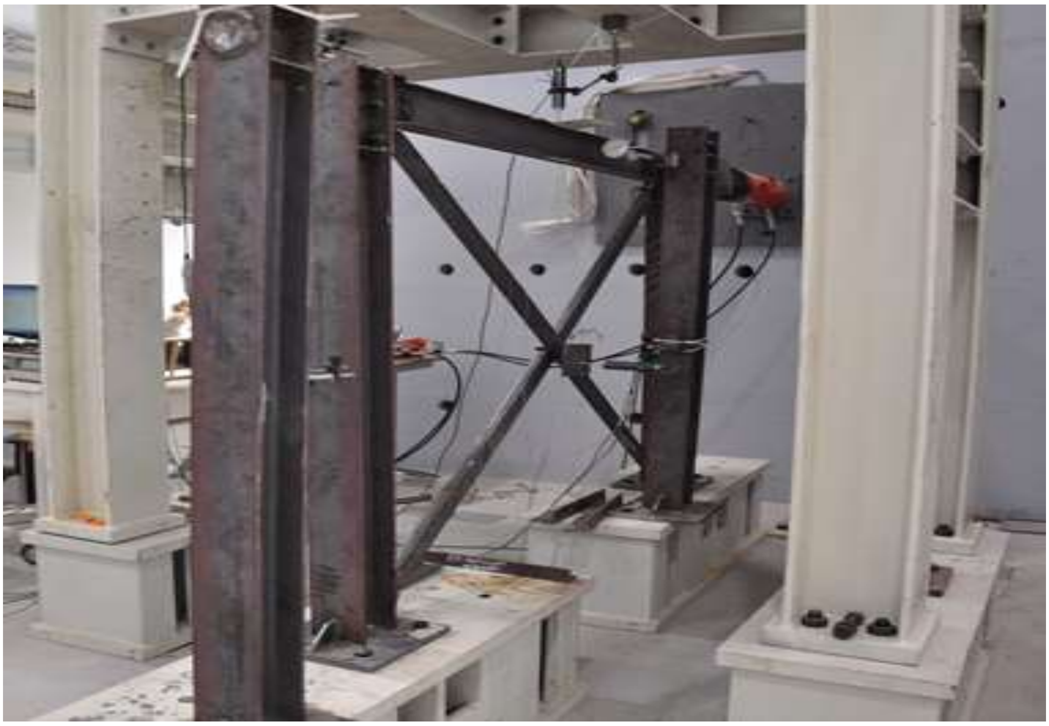


Figure B.14: Set up of test frame MAJ-BF7.


Figure B.15: Set up of test frame MIN-MFIN2.