EVALUATION OF LATERAL STIFFNESS OF DIFFERENT FORMS OF BRACINGS AND SHEAR WALLS AGAINST LATERAL LOADINGS FOR STEEL FRAMES

A THESIS SUBMITTED TO THE GRADUATE SCHOOL OF APPLIED SCIENCES OF NEAR EAST UNIVERSITY

By
KREKAR KADIR NABI

In Partial Fulfilment of the Requirements for the Degree of Master of Science in Civil Engineering

NICOSIA, 2018

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Approval of Director of Graduate School of Applied Sciences

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I hereby declare that all information in this document has been obtained and presented in accordance with academic rules and ethical conduct. I also declare that, as required by these rules and conduct, I have fully cited and referenced all material and results that are not original to this work.

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Date:

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ABSTRACT

One of the most significant properties of a building is the lateral stiffness which defines the resistance to displacement under seismic and wind loads, simultaneously the lateral stiffness has a great influence on the natural time period of the structure. In this study, pushover analysis is used to evaluate the elastic stiffness factor, natural time period, maximum base shear and pushover curves of 2D steel frames for different lateral load resisting systems. First, 720 2D steel models have been analyzed and designed using equivalent lateral force procedure. After that by using pushover analysis method, the results of all models have been analyzed, compared and evaluated. Then the effect of number of parameters such as different lateral load resisting systems, span length, number of stories, number of spans and story height on the elastic stiffness, natural time period, maximum base shear and pushover curves are considered. Based on the pushover analysis method in this study, by applying the effect of parameters considered in this study, the elastic stiffness factor, natural time period, maximum base shear and pushover curves of the structure with an acceptable result can be evaluated, and the obtained results show that, pushover analysis is an appropriate method to evaluate the performance of steel frames.

Keywords: Lateral load resisting systems; pushover analysis; elastic stiffness; natural time period; maximum base shear; pushover curves

ÖZET

Bir binanın en önemli özelliklerinden biri, sismik ve rüzgar yüklerinnnin altında yer değiştirmeyeolan direncini tanımlayan yanal rijitliktir, aynı zamanda yatay rijitliğin binanın ilk zaman döneminde büyük bir etkisi vardır. Bu çalışmada, yanal yüke dayanıklı sistemler için, rijitlik katsayısı, doğal periyot, maksimum taban kesme kuvveti ve itme eğrileri değerlerinin değerlendirilmesinde statik itme analizi kullanılmıştır. Öncelikle, 720 adet iki boyutlu çelik modeller analiz edilmiştir ve eşit yanal kuvvet prosedürü kullanılarak dizayn edilmiştir. Daha sonra statik itme analiz yöntemi kullanılarak tüm modellerin sonuçları analiz edilmiş, karşılaştırılmış ve değerlendirilmiştir. Farklı yanal rijitliğin, yük direnç sistemlerini, açıklık uzunluğu, kat sayısı, açıklık sayısı ve kat yükseliği gibi değişkenlerin rijitlik katsayısı , doğal periyot, maksimum taban kesme kuvveti ve itme eğrileri üzerindeki etkileri dikkate alınmıştır. Bu çalışmada statik itme anazliz yöntemine dayanarak, göz önüne alınan parametrelerin etkisi ile rijitlik faktörü, doğal periyot ve maksimum taban kesme kuvveti değerlerinin kabul edilebilir bir sonuca sahip olduğu gözlemlenmiştir. Elde edilen sonuçlara göre, statik itme analizi yönteminin çelik çerçevelerin performansını değerlendirmek için uygun olduğu görülmüştür.

Anahtar Kelimeler: Yanal yüke dayanıklı sistemler; statik itme analizi, elastik rijitlik, doğal periyod, maksimum taban kesme kuvveti

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SYMBOLS

ACI: American concrete institute

AISC: American institute for steel construction

ASCE: American society for civil engineering

EBF: Eccentrically braced frame

LLRS: Lateral load resisting systems

OCBF: Ordinary concentrically braced frame

OMRF: Ordinary moment resisting frame

SCOSW: Steel and composite ordinary shear walls

SDC: Seismic design category

NLPA: Non-linear pushover analysis

MRF: Moment resisting frame

CHAPTER 1
INTRODUCTION

1.1 Introduction

It is undoubtable that buildings are always subject to different types of loads which can either be lateral load or vertical load and in some cases a combination of both types of loads. Hence, it is always important to ensure that buildings have structural mechanisms that can handle the all the different types of loads. This can be evidenced by ideas which states that failure to cater for lateral load especially in double-storey buildings can pose serious problems. As a result, designers are strongly encouraged to come up with designs that can address this problem. This is important because it helps to improve the safety of the building. As such, so many different types of load resisting structures which are capable of sustaining different types of loads were developed. Such developments have made it possible to develop stiff structure such as lateral force resisting mechanisms which are capable of handling lateral forces. This is so important especially in areas which are prone to earthquakes because such structures are earthquake resistant. In most cases, an earthquake can produce severe horizontal forces which can weaken the structural parts of the building and thereby causing the entire structure to collapse. Thus, it is encouraged to have structural systems that can resist wind and seismic forces, and other types of horizontal forces. On the other hand, structures can fail as a result of being exposed to sway movement and severe stress produced by lateral forces. It is in this regard that that suggestions are made to develop stiff and strong structures that can withstand both lateral and vertical loads. Consequently, this justifies the importance of studies that examine the performance of lateral force resisting structures when subjected to seismic forces. Thus, this study concentrates on analyzing the effect of lateral force resistant mechanisms such as steel bracing and shear wall on building structures. (H.M. Somasekharaiah et al. 2016).

Over the past ten years, the construction industry has capitalized on the use of steel so as to enhance the structural performance of a building structure when subjected to seismic load. This has included the introduction of lateral resistant systems which help to improve shear capacity of the building structure. These include eccentrically braced frames, concentrically braced and shear walls. However, care must be given when choosing between different lateral resistant mechanisms and it is important to ensure that the desired mechanism possess the required stiffness capable of withstanding seismic forces. As such, this study used push over analysis to determine among others, the elastic stiffness factor and natural time period , pushover curves and maximum base shear properties of the lateral resistant system considering different parameters. (Padmakar, 2013)

1.2 Steel

The steel industry is one of the key sectors of the economy and the produced steel is used in quite a number of construction activities. This is mainly because steel has better structural properties such as strength. For instance, the strength of steel is tenfold better than that of concrete. The structural properties of steel which make it an ideal construction material are not limited to strength but also include demount ability, prefabrication and speed of erection. Steel is used in buildings for a lot of things such as space frames, bridges, in trusses and load-bearing frames. But its uses require that that it be protected against corrosion and fire and in most cases, it is supported by the use of concrete foundation, masonry materials and claddings. In some cases, it is also used with a combination of shear and frame wall construction. One of the notable advantages of using steel is that it has a better life span. This is because it has high strength in relation to its weight. In addition, steel is a bit affordable as compared to other building materials such as concrete. Moreover, steel structures do not take much time to construct and this makes it easy to speed up the construction process. More importantly is the idea that steel results in light construction projects, has a high tensile strength and a better compressive ability.

As noted, the effectiveness of steel requires that it be protected against corrosion and fire and most importantly, it ought to be structured in a way that promotes erection and fabrication. On the hand, sound quality control is always needed when fitting steel structures together. Such considerations must also take into account of changes in temperatures. However, this does not discount the fact that steel can hold off the effects of an earthquake, is robust and ductile. But this is only guaranteed when all the weds have been properly designed and designers are encouraged to have full knowledge and understanding of the best available designs. This helps to avoid the problem of fatigue which might occur as result of the development of cracks. However, steel has a better capacity to allow for retrofitting to sustain huge loads and easy repairs. When it comes to environmental sustainability, one can contend that steel is totally recyclable and environmentally friendly. Moreover, its production is done in an environment characterized by high quality control measures and this makes it one of the safest and reliable construction materials. (Padmakar, 2013)

1.3 Lateral Load Resisting Systems (LLRS)

Structural systems are mainly designed to promote effective distribution of gravity in building structures. Gravity is usually associated with three distinct types of loads and these are snow load, live load and dead load. Apart from gravity, earthquakes, blasting and wind can also cause lateral load. The challenge is that vibration, sway movement and high can occur when a building is exposed to lateral load. Hence, it is of high importance to ensure that the building structure are very stiff and strong so that they will be able to withstand vertical loads. In earthquake engineering, one of the ways that can be used to determine the capacity of a building to determine the stiffness and strength of a building is seismic analysis. This approach involves exposing the building to seismic excitations. In the past, much of the focus was centered on testing for gravity, but modern developments now include structural analysis during an earthquake, in particular seismic analysis. This has led to the development of lateral load resistant mechanisms that are capable of withstanding gravity and eccentric loads, wind and seismic forces. Lateral load tends to vary with the height of the building and this is notable in tall buildings. This is why it is important to design stable, rigid and strong structures but the challenge is that this is associated with high structural costs. This problem is notable in two storey buildings and this requires that systems that are capable of withstanding lateral load. Such systems can be listed as follows (Thorat, S. R., & Salunke, P. J.,2014):

I. Moment Resisting Frames
II. Shear walls
III. Concentrically and eccentrically braced frames

1.3.1 Moment resisting frames

Moment frames are made up of horizontal (beams) and vertical (columns) members as depicted in figure 1.1. Moment frames are capable of holding shear force and bending moment generated in columns and beams through the use of axial forces. But capacity design procedures should be used to ensure that the design of the columns and beams are able to prevent brittle shear failure and undergo ductile behavior (Baikerikar, A., & Kanagali, K., 2014).

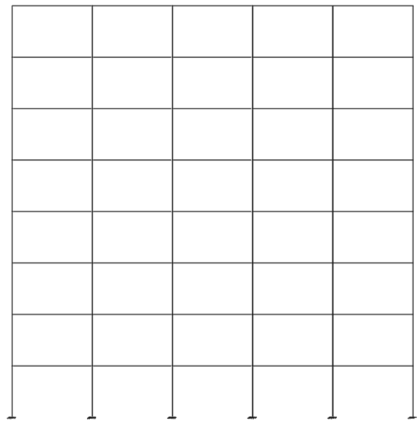


Figure 1.1: Moment resisting frame

1.3.2 Shear walls

It can be noted that buildings are bound to shake during an earthquake and hence it is of important to ensure that the buildings have earthquake resistant structures that meet the required stiffness levels. This will help to prevent the building from shaKng a lot during an earthquake. This is one of the challenges of using moment frames and ideas assert that moment frames may not be able to address this issue. Shear walls (structural walls) can be used to prevent shifting of the entire building especially in buildings that have moment frames which are subjected to a lot of lateral displacement. This is made possible because they have built in planes that are strong and stiff. Thus, each area which has structural walls will be having moment frames with specific bays. In this way, structural walls are characterized by combined axial-flexure-shear action which makes it capable of withholding lateral forces. Using a combination of lateral load resistant system and moment frames will aid in reducing moment and shear pressure on the columns and beams of the building. In order to ensure that the building will perform way much better during an earthquake, it is important to ensure that the entire building has structural walls. The performance of the building can also be enhanced by maKng sure that the building is constructed on hard soil strata. However, using structural walls alone is not sufficient to resist lateral loads. This is because the position of the structural walls also plays an important role in improving the load resisting capacity of the building. Overall, structural walls help to deal with natural periods of oscillation and the problem of lateral displacement (Baikerikar, A., & Kanagali, K., 2014).

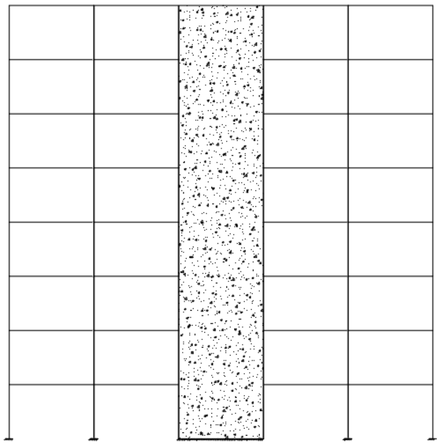


Figure 1.2: Shear walls

1.3.3 Concentrically and eccentrically braced frames

Bracings are a structural system which is designed primarily to resist wind and earthquake forces. Members in a braced frame are intended to work in compression and tension alike a truss. Braces assist in lowering shear force demands and lessening bending moment on beams and columns in buildings and in lessening the entire lateral displacement of buildings.

The earthquake force is shifted as an axial force in the brace members. It is possible to use several Knds of an eccentrically braced frame like K shaped bracings and this includes global bracing along the building height. It is also possible to use concentrically frames such as X, Z, V and IV shaped, Braced frames are easy to raise on site, and bracing elements can be changed to allow horizontal movement across the floor plate. Although braced frame systems can be included inside concrete framed fabrications, they are properly suitable for use in steel framed buildings with eccentrically braced frames and/or diagonal bracing (Baikerikar, A., & Kanagali, K., 2014).

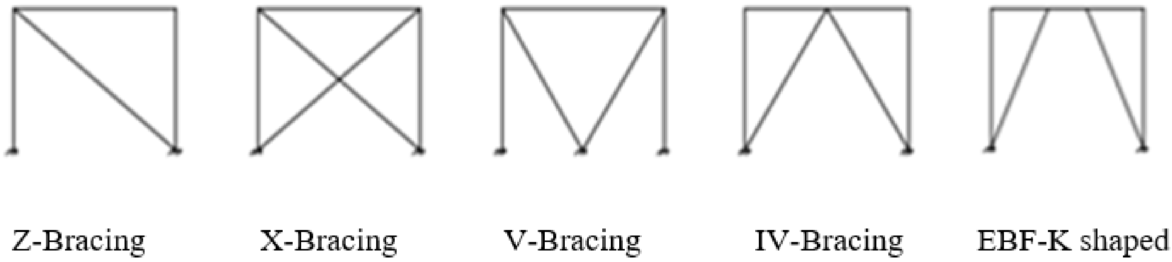


Figure 1.3: Concentrically and eccentrically braced frames

1.4 Stiffness

In simple terms, stiffness is simply an indication of how rigid an object is. That is, the ability of an object not to deform when subjected to a load. The greater the ability not to deform, the stiffer the object will be. Despite the existence of so many definitions about stiffness,

Hook's law consider it as an ability to displace an equally proportional force to the subjected force on solid objects. This is often captured by what is known as the coefficient of stiffness and can be determined using the following expression;

$$[K] = \frac{\{F\}}{\{D\}} \qquad (1.1)$$

The object's stiffness is represented by K, the produced displacement by D and the applied force by F. Equation (1.1) thus illustrates that there is an indirect relationship that exists between lateral displacements and the structure's stiffness. This entails that the stiffness of an objectives has significant effect on displacement. Thus, it is essential to determine how changes in stiffness influence the object's ability to displace a load so as to effective chose the best material or object to use in building structures. However, though stiffness is a good feature, the use of stiff materials can affect the design of building standards and structures. Thus, the ability to solve structures analysis equations and problems relies on the ability to know the stiffness matrices and values (Rokhgar, N., 2014).

1.5 Natural Time Period

This is the period which indirectly related to the building frequency when its harmonic is at its lowest level and measures the extent to which a structure moves back and forth. This period does not vary with the load applied but is determined by the stiffness and mass of the object as shown below;

$$T = 2\pi\sqrt{\frac{m}{k}} \tag{1.2}$$

The equation shows that a structure's natural period significantly changes in response to the stiffness of the object. Usually the natural period is short when the object is stiffer. On the other hand, modal periods are of huge importance in building and have implications on the examination of a structure. The other emphasis of this study is placed on the need to examine the effects of changes in lateral resistant systems parameters on natural period (Rokhgar, N., 2014).

1.6 Objective and Scope

1. The main emphasis of this study is to contrast and assess the natural time period and elastic stiffness factor of various types of shear walls and bracing systems of 2D steel frames.
2. To assess the impact of various coefficients on the elastic stiffness factor and time period of 2D steel frames for various forms of shear walls and bracings.
3. To choose the best possible earthquake lateral load resistant shear walls and bracing forms which can offer the best stiffness.
4. To examine the seismic response of 2D steel frames by conducting non- linear and linear static examinations

1.7 Significance of the Study

1. This study offers a quick method for determining the lateral stiffness of building structures, including braced frames as well as frames with shear walls, which can be used for preparatory examination, seismic assessment of old and present buildings.
2. The method can be used to estimate the displacement of the building at separate stories which are subjected to lateral loads so as to improve the contribution of various lateral resistant systems in maintaining the lateral loads.
3. Analyzing the various kinds of bracings and shear walls helps to explain the structural response of an object under seismic action. This can act as a guideline to view and examine the potential lateral load resisting systems throughout the design phase and choose the suitable lateral load resisting systems based on the analyzed results.

1.8 Organization of the Thesis

The study consists of five chapters. The first chapter provides an introduction to the study and the aim of the study is clarified in this chapter, it also delivers a brief explanation to the lateral load resisting systems used in this study.

The previous studies related to the thesis are shown in the second chapter, the literature reviews are divided into two parts, the first part evaluates and compares different lateral load resisting systems and the second part describes the pushover analysis used in the previous studies.

The third chapter covers the theory and formulation which includes the details about the material used, the process of simulation of the structure, base shear calculation and pushover analysis carried out for the same.

The fourth chapter contains results and discussions of the models.

The fifth chapter lists the conclusions and recommendation which are drawn from the work.

CHAPTER 2
LITERATURE RVIEW

2.1 General

This chapter describes previous researches related to different lateral load resisting systems. Similarly, this chapter also introduces previous studies on the pushover analysis method used for evaluation of seismic performance of new and existing buildings.

2.2 Literature Review on Lateral Load Resisting Systems

Baikerikar and Kanagali (2014) Used a regular model having 4 spans in each direction with a length of 5 m for each span, ETABS 9.7.0 software computer program is used in this study to evaluate and compare the effect of lateral load resisting systems including shear wall and bracings for varied heights, for the present study maximum height considered is 75 m. After modeling, all the buildings are evaluated to find the influence of lateral load resisting systems with different heights based on lateral displacement, lateral drift base shear and time period. The seismic zone V is selected for the study and the type of the soil is selected as specified in IS 1893-2002. From the analytical results, it is determined that lateral displacement and drift increases as the height of the buildings increases. MRF produces larger displacement and drift compared to shear wall and bracings. It is also observed, after placing lateral load resisting systems into the building, lateral displacement of the building significantly decreases. From the study it is found that the time period of the building increases with increasing the height of the building because the stiffness of buildings decreases and the overall mass of the building increases at the same time. After placing lateral load resisting systems, time period has significantly decreased because the stiffness of the building increases.

Kevadkar and Kodag (2013) did a 3-phase analysis of a modeled R.C.C. building in which the first phase did not have shear walls and bracings, the second had various shear walls and the third had also various bracings. The objective was to determine which lateral load mechanism would effectively sustain a load in an environment of severe seismic force and the analysis was done using E-TABS. The building's performance was evaluated in terms of demand, base shear, storey drifts, storey shear and lateral displacement capacity. It was established that shear wall systems did not contribute much towards reducing the demand capacity, lateral displacement and enhancing the stiffness of R.C.C building as compared to steel bracing systems of an X type.

Choudhari and Nagaraj (2015) did a pushover analyses that used SAP2000 to analyze the effects of using knee, inverted V, V and X bracings to model a G+4 steel bare frame. The results were compared together based on their performance points, storey drift, time period, roof displacement and base shears. The findings were similar to what was established by Kevadkar and Kodag (2013) and it was concluded that steel bracing systems of an X type are effective in reducing maximum interstate drift and contributing towards enhancing a steel building's structural stiffness.

Esmaeili et al. (2011) studied the difference between the effects of using concentric braced frames and concrete shear walls to reinforce concrete moment-resisting frames affect the responsiveness of a building's structural system. This was based on the use of a pushover analysis approach aimed at examining how the structural system of a 30-storey building would respond when exposed to seismic conditions. The analysis was conducted based on how the structures behaved in terms of response modification, over-strength and ductility ability. It was noted that the structural systems behaved in an inelastic nonlinear manner that caused them to withstand and displace the entire seismic force. In addition, it was considered that response modification and ductility are high when the RCSWA are used together with SMRF. That is, it has a better capacity to handle seismic forces.

Tafheem and Khusru (2013) focused on analyzing how live, dead and wind loading, and lateral earthquake affects the structural performance of a building using a 6-storey building model. The performance of the building was evaluated based on how the building responded when braced with HSS sections, V-type and crossed X bracings in relation to bending moment, axial and drift force, and storey displacement. It was noted that structures with X-bracings were relatively stiffer and had a better capacity to displace more lateral load.

Dharanya, Gayathri and Deepika (2017) examined the role of shear walls and bracing in G+4 storey residential RC building using ETABS and this was done in accordance IS 1893:2002 guidelines. Focus was placed on looking at how the time period, shear and axial force, storey drift, base shear and lateral displacement change in the event of an exposure to seismic effects. They established that the presence of an earthquakes exposes all the areas to seismic forces and that effects are high in tall buildings. As a result, they outlined that such buildings tend to be highly responsive to oscillatory movements caused by torsional or lateral deflections. This is why it is important to make sure that all building structures have the required stiffness capacity enough to withstand seismic effects. This can be done by using cross bracings and shear walls. Discoveries were made that placing shear wall in the building has an effect of reducing the natural period as compared to using bracings. Hence, shear walls were considered as having a high capacity to enhance the stability of multi-storey buildings during seismic events.

Kumar, Naveen and Shetty (2015) concentrated on examining variations in performance of building structures situated in areas considered by the IS-1893-2002 as Zone 5. The motive was to determine the best structural behaviour of buildings fitted with braces in handling lateral loads triggered by seismic effects. It was confirmed that braces have a positive contribution towards improving the stiffness of the buildings in high seismic zones. The natural period and the natural frequency of the structures was discovered to be bilaterally and unilaterally related to stiffness. However, it was further concluded that the natural period continuously increases in tall buildings even as high as 9-storeys whereas lack of stiffness causes the natural frequency to decline. These results strongly show that there is a positive association between natural period and the height of a building. But the structures must be braced to enhance the stiffness of the entire structure.

Viswanath, Prakash and Desai (2010) did a similar study as to the one by Kumar, Naveen and Shetty (2015) and based their efforts on IS-1893-2002 as Zone 5 but this focus was based on 4-storey buildings. Their study was aimed at evaluating the performance of building structures in relation to story and global drifts of structure that are braced with steel braces of an X-type. The argument was that steel braces of an X-type are effective in improving the stiffness of a building structure during seismic activities. The findings went on to establish that bracing a structure with steel bracings of an X-type are way effective in enhancing the stiffness of a building structure. The study went on to establish that steel bracings have a high potential to enhance the stiffness of a structure and flexible to suit the design of any structure. More so, they were considered to be economical that other type of reinforcements and bracings. The use of X-type steel bracings was still considered as the best way of improving the strength of a building structure especially those found in high seismic zones. As a result, the maximum drift of a structure was noted to be low in structures that have X-type steel bracings. Moreover, the effectiveness of X-type steel bracings was believed to be high even from 4-storey to 12-storey buildings. Hence, it can eb said that the use of steel bracings helps to minimize shear and flexure demands and can displace a huge amount of load and this is because they have a low level of bending moments.

Venkatesh, Sharada and Divya (2013) based arguments of their study on the idea that earthquakes have an inclination to destroy any building structure especially those that are not created to withstand lateral loads. Hence, they reiterated the importance of having load resisting systems such as steel bracings, infill frames and shear walls. In an attempt to prove their argument, they used 2-bay and 3-bay 3D 10-storey building models that are reinforced with steel bracings to test their ability to handle lateral loads in India's Seismic Zone 5. The models were subjected to linear dynamic analysis to determine the beam force, support reaction and joint displacement values of the three models having internal and external steel bracings, and a moment resisting RC frame. The findings outlined that steel bracings have a high potential to improve a structure's ability to handle lateral loads. Considerations were also made that bother internal and external bracings be used for an improved maximum total load resistant ability. However, the use of internal and external bracings requires that the structures be properly connected whether it is retrofitting or an upgrade.

Azam and Vinod Hosur (2013) did an examined how a combination of reinforcements can be used to improve the performance of a building structure. Their examination was based on the use of concrete shear walls and special moment resisting frames. As a result, they compared the performance of the structures based on their damping, stiffness and strength by changing the position of the structure's frames. The observations were analyzed using static pushover and response spectrum analysis. It was published that changing the position of the structural frames has an important implication on a building structure's damping, stiffness and strength. Most importantly, the symmetrical positioning of shear walls next to the moment resisting frames was observed to offer the best seismic resistance capacity.

This led to the conclusion that shear walls (RC) can withstand severe seismic effects during an earthquake or any wind subjected load. Hence, the also supported the idea of combining different types of load resisting mechanisms.

Chandiwala (2012) observed that there is a growing demand for secure buildings that can withstand an earthquake. This was in turn, thought to have resulted in a rise in demand for moment holding systems. But Chandiwala stressed out the importance of minimizing costs and need to ensure optimality in the use of steel as well as having acceptable concrete walls of the right size. It was discovered that the outer parts of a flange always sway a lot during seismic activities and that having an "L" section wall with an F-shear wall will help enhance the performance of a structure.

Venkatesh and Bai (2001) assert that buildings must be capable of withstanding seismic effects of any magnitude. With this in mind, they reiterated the importance of knowing the responsiveness of a structure to seismic activities when subjected to a lateral load. They used two different shear walls in three 3D single 3-bays in India's IS 1893 seismic Zone 5 using 15 models. The models were evaluated in terms of their ability to handle seismic, live and dead loads. Of the respective models, two models had moment resisting frames of different columns and sizes and one had 3 bare frames of different sizes. Both the internal and external walls comprised of varying width. The models' beam and column forces, support reactions and joint displacement values were determined using linear static analysis. It was discovered that structures with squares walls have a high lateral load resistance capacity. In addition, the use of internal and external shear walls was also established as capable of reducing the displacement of the frames' large joints. The findings however, rejected the idea that the thickness of the walls plays an important role towards enhancing the stiffness of a structure. On the other hand, the performance of rectangular columns was considered to be lower than that of square columns when both are subjected to lateral loads. Also, a combined use of internal and external loads was established as capable of lowering individual forces and support reactions. However, the use of external walls was established to be performing poorly that a structure with internal shear walls. The challenge is that such a method may result in an increase in torsion moment and shear force in the beams and columns. Hence, case like retrofitting which might not be possible to do when external shear walls are used, often work best when internal shear walls are used. Venkatesh and Bai further concluded that any need to determine the best structure to use, must consider both the seismic and gravity loads.

2.3 Literature Review on Pushover Analysis

Balaji et al. (2012) used ETABS and SAP-2000 to analyze the performance of structures with different symmetrical features inclined at a 30-degree angle. The structures when then subjected to loads of different sizes. The push over results showed that unequal vertical structures are more prone to fractures caused by seismic effects. Balaji contends that nonlinear analysis rather than ATC 40, be done to examine nonlinear behaviour in buildings induced by seismic effects. The pushover analysis first involved displacing the building and then to earthquake excitation was done up to a level where the target displacement equals the top displacement. Nonlinear static analysis in asymmetric buildings was also used to determine the torsion effects up from the onset up to their point of failure. The study was done in line with recommendations by Shakeri (2012) to use a displacement based adaptive pushover throughout the entire analysis (Chintanapakde, 2004).

Kadid and BoumrKk (2008) looked at how vulnerable structures developed in accordance to Algerian standards would act when displaced. The study was done using a pushover analysis and capacity curves were developed for each building structure and this made it possible to determine each building's target displacement. The study was also done under the assumption that the actual damage that will occur to the building during the earthquake. Conclusions were made that reinforced structures have an inelastic response to the effects of an earthquake. However, they considered that the accuracy of pushover analysis is subjective and determined by the extent to which other analysis methods are able to record the impact of the seismic activities.

Faella et al. (2002) suggested that methods be developed to capture both the demand and displacement capacity of the structures. Their aim was to develop methods that easily be applied and used to determine the stiffness of a structure during seismic activities and its degree of vulnerability. The results pointed out that subsoils are not stiffer enough to withstand seismic effects and hence make the structure more vulnerable to the impact of seismic effects. Efforts to determine the bracing mechanism with the best mechanical feature to use in retrofitting in accordance to Eurocode 8-Pt.3 safety standards. Recommendations were made that having using displacement demand is not an effective way of assessing the displacement capacity of a structure as well as the type of bracing to use in a structure rather its lateral stiffness.

Monavari, Massumi and Kazem (2012) used NLSA to a building's determine the seismic demand, failure criteria and overall yields in Iran using 13 structures with 2 to 20 storeys that are reinforced with concrete frames. The modelling process was done using modeled by IDARC in line with the ACI318-99 Building Code and the 2005 Iranian Seismic Code. They considered that there is an unresolved issue over the following effects of an earthquake and its ability to cause overall failure in a structure. The experimental findings revealed that some structures started failing as the structures were losing their stiffness. The failure of the structures varied and some structures experienced total failure while others experienced minor effects.

Sattar and Liel (2010) made an attempt to determine the effectiveness of masonry infill walls in reducing the risk of nonlinear building models collapsing when subjected to seismic effects. The performance of the bare frames was discovered to be lower than that of the infilled frames in relation to both the amount of energy displaced, stiffness and initial strength irrespective of the walls failing. Findings made from the dynamic analysis showed that the impact of an earthquake is high in a structure that are fitted with bare frames. This is because their have a lower capacity to dissipate energy and are of low strength.

Shah et al. (2011) posit that it is difficult to solve nonlinear static analysis because of its natural procedure. As a result, they recommend that software such as ADINA, SAP and ETABS be used to deal with any situation involving NLSA. This is because they can handle any geometrical situation irrespective of its complexity. Moreover, they have ASCE41-13, FEMA 273 and ATC-40 features that enable them to assess any structure's ultimate deformation. The use of ETABS 9.7 is done in respect of the following stages;

- Modelling,
- Static analysis
- Designing
- Pushover analysis

In addition, it was pointed out that this is also due to the idea that it strongly revolves around the final displacement of the structure and this makes the process more difficult especially at the final load. They further concluded that activities of instability will have an effective of producing a negative stiffness matrix.

Sofyan (2013) did an analysis of the impact of using concrete frames that are reinforced with 5-bays in 10-storeys buildings in Mosul, Iraq using NLSA. The performance of the buildings was determined based on their ability to withstand seismic load taking into account of the buildings' nonlinear response to lateral static load. The study proved that reinforcing structures with concrete frames helps to reduce the seismic effects. The building was discovered to be structurally stable and strong to withstand seismic effects because its maximum total drift remained inelastic to changes in seismic force. It was discovered that beams faced a problem of plastic hinge formation in each of the individual frame at collapse prevention performance level. As a result, there is always a need to improve the beams' strength.

Dhileep et al. (2011) based their focus on nonlinear seismic aspects of high modal frequency and their responsiveness capacity using NLSPA. The use of pushover analysis was considered to offer the best results even though there are ideas which suggests that it can be associated with a lot of inexactness about the responsiveness of higher modes. As a result, it is considered that a small number of lower order modes be used to assess the overall responsive capacity so as to obtain a high level of reliability. Hence, it is always best to account for the impact of nonlinear effects and frequency modes. It was reported that high frequency modes are a common feature in irregular or stiff structures. It was also discovered that the effectiveness of NLPA depends on the presence of rigid content of higher modes.

CHAPTER 3
METHOD OF ANALYSIS

The analysis, design and evaluation process of all models used in this study are explained through this chapter. Equivalent lateral force procedure used for the analysis and design of the models and then all the models are evaluated using pushover analysis and their procedure can be found throughout this chapter. For the analysis, design and evaluation of the structures and their execution assessment numerical model is required. So in the present study, ETABS 2016 computer program is used to build the models and performing equivalent lateral force procedure and pushover analysis.

3.1 Frame Types

Distinctive kinds of 2D steel frames are thought about and exposed to the analysis and designing. Eight lateral load resisting systems are used including, ordinary moment resisting frame (OMRF), Steel ordinary concentrically braced frames (OCBF) with (X, Z, V and IV shaped bracings), Steel eccentrically braced frames (EBF) with (K-shaped) and Steel and concrete composite ordinary shear walls (SCOSW) with (two compressive strengths 25 and 30 N/mm^2) are used. There are other parameters that have been changed for the above structural systems, the span length (L) of 4.5, 5, 5.5, 6 and 6.5 m as well as the number of stories (S) 1 (Low), 5 (Medium) and 8 (High) have been considered, and with the variation of number of bays (N) 1, 3, and 5 bays. For the height of stories (H), the values of 3.2 and 3.4 meters are applied. The lateral load resisting systems are placed in the middle of spans. As a result, the database of this research contains 720 models of buildings using different steel framing systems.

3.2 Illustration of Frame Types with Figures

a) Lateral load resisting systems (B)

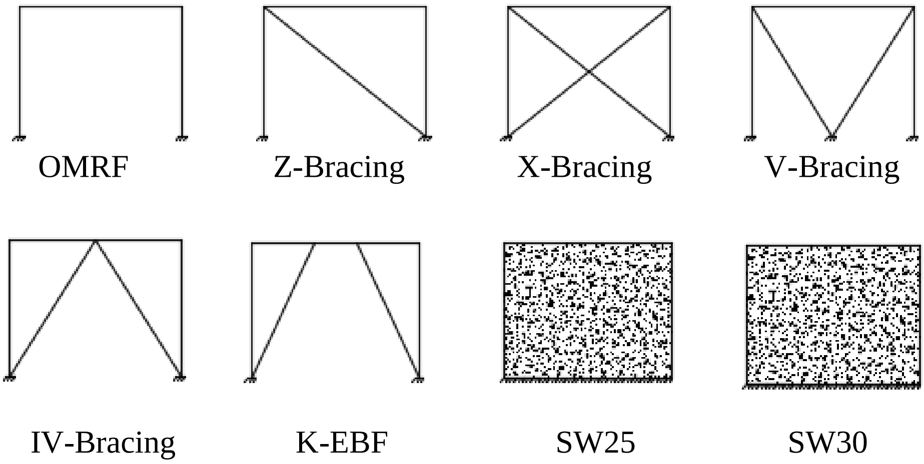


Figure 3.1: Different lateral resisting systems

b) Span length (L)

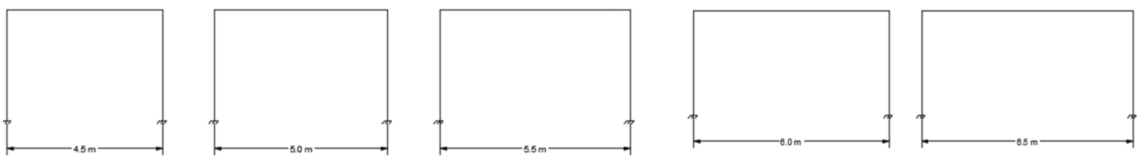


Figure 3.2: Span length change

c) Number of stories (S)

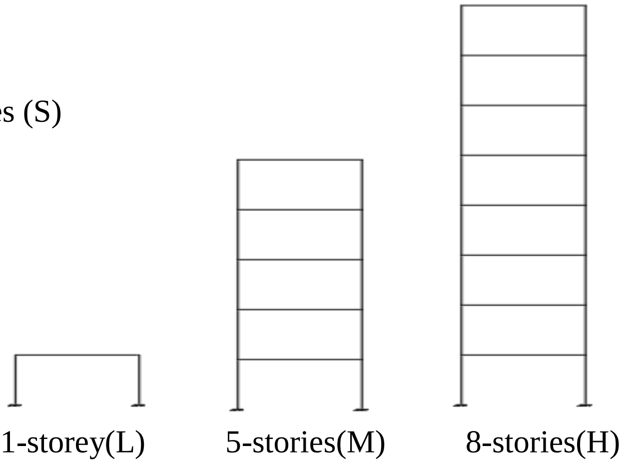


Figure 3.3: Number of stories

d) Number of spans (N)

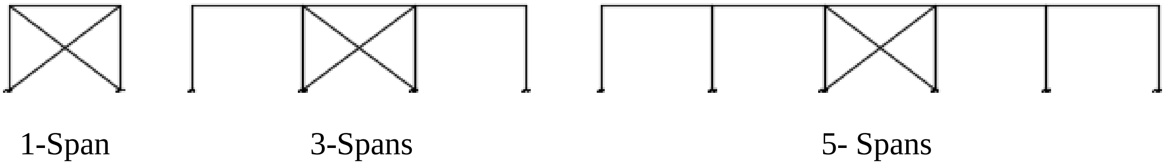


Figure 3.4: Number of spans

e) Story height (H)

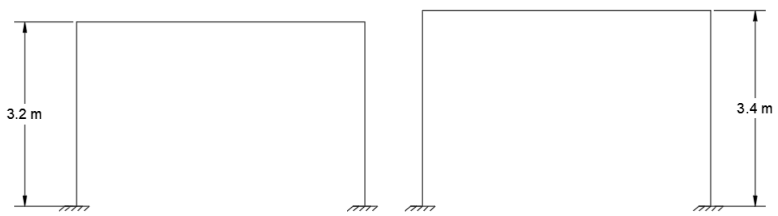


Figure 3.5: Story height change

3.3 Material Properties

Two types of material are used in this study, they are steel and concrete, their properties are explained in the below table.

Table 3.1: Material properties of models

Materials	**properties**
Fy of steel sections	240 N/mm^2
Fu of steel sections	448 N/mm^2
F'c for shear walls	250 and 300 N/mm^2
Steel modulus of elasticity	200000 N/mm^2
Concrete modulus of elasticity	23500 and 25743 N/mm^2
Fy of reinforcement steel	420 N/mm^2
Unit weight of concrete	24 kN/m^3

The material properties of steel sections are used for the steel frames, concentrically and eccentrically braced frames, the two compressive strength and yield strength of reinforcement steel are utilized for the shear walls in combination with steel frames.

3.4 Gravity Loads

In all models, dead load, super dead load and live loads are fixed and considered to be the same for all models. The gravity loads considered in this thesis are live load, super dead load and dead load (self-weight of the structure)

The program automatically calculates the self-weight of the structure. But live load and super dead load are defined and assigned to the program as follows. The live load is 25 kN/m and super dead load 20 kN/m are considered and assigned to the frames

3.5 Seismic Analysis Methods

Every structure should be designed in such a way to resist lateral loadings including earthquakes. In this study, the seismic loadings are determined according to ASCE 7-10 provisions. There are four types of seismic analysis, the seismic analysis type that should be used to analyze the structure depends on dynamic properties, the structure's seismic design category, regularity and structural system.

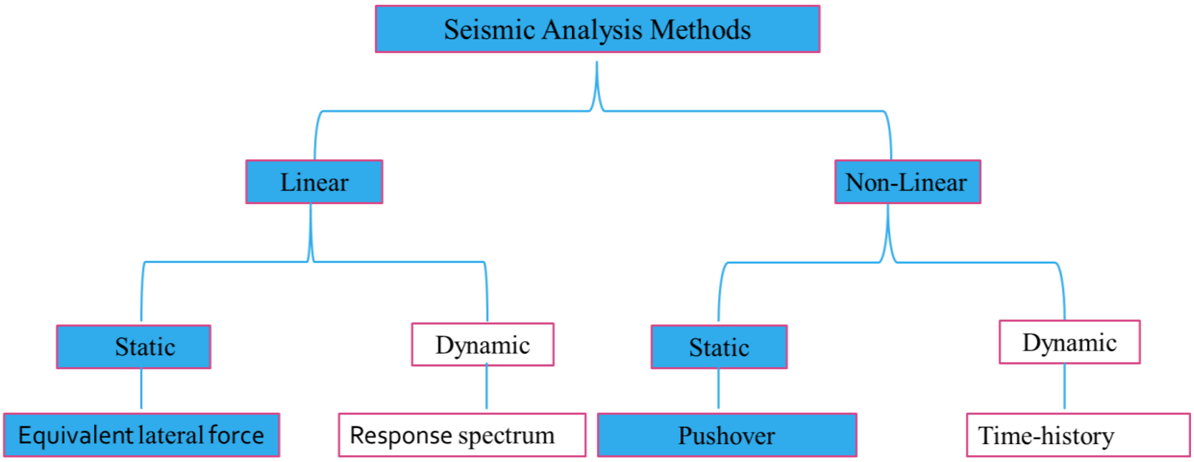


Figure 3.6: Seismic analysis methods

The seismic design category (SDC) of all the models is category C as calculated in the SDC section 3.6.2. After finding the SDC of all models, equivalent lateral force procedure is selected for the analysis and designing of all models based on ASCE Table 12.6-1. Therefore, after designing the models, all the models are evaluated using non-linear static analysis (pushover analysis). All the models are evaluated using ETABS 2016.

3.6 Seismic Design Category (SDC)

Structures are assigned to an SDC based on the severity of the design earthquake ground motion at the site and its occupancy. Section 3.6.1 illustrates the procedure to find SDC of a structure.

3.6.1 Procedure for calculation of SDC according to ASCE 7-10. (ASCE/SEI 7–10)

1- Determine risk category in Table 1.5-1 in ASCE 7-10, in this study risk category I is used since the frames are considered to be designed for residential building so importance factor is 1 according to Table 1.5-2 ASCE 7-10.

2- The mapped MCE_R spectral response acceleration parameter for short periods (Ss) and mapped MCE_R spectral response acceleration parameter at a period of 1 second (S_1) are determined based on the location of the building. In this thesis Ss and S_1 values are taken from Kurdistan Region of Iraq (Erbil city) which are 0.52 g and 0.13 g respectively.

3- From the properties of the soil and the soil profile name, the site class is determined. In this study site class D is used since it is permitted to be used by ASCE 7-10 when the location is unknown.

4- Then the MCE_R spectral response acceleration parameter for short periods (S_{MS}) and at 1 second (S_{M1}) are adjusted for Site Class effects (equation 3.1 and 3.2) according to ASCE 7-10 section 11.4.3

$$S_{MS} = F_a S_S \tag{3.1}$$

$$S_{M1} = F_v S_1 \tag{3.2}$$

ASCE 7-10, Tables 11.4-1 and 11.4-2 defines site coefficients F_a and F_v and these tables are demonstrated in this thesis in Table 3.2 and 3.3, respectively.

Table 3.2: Site coefficient F_a

Site Class	Mapped MCE_R spectral response acceleration parameter at short periods				
	$S_S \leq 0.25$	$S_S = 0.5$	$S_S = 0.75$	$S_S = 1.0$	$S_S \geq 1.25$
A	0.8	0.8	0.8	0.8	0.8
B	1	1	1	1	1
C	1.2	1.2	1.1	1	1
D	1.6	1.4	1.2	1.1	1
E	2.5	1.7	1.2	0.9	0.9
F	See section 11.4.7 of ASCE				

Note: Use straight-line interpolation for intermediate values of S_s

Table 3.3: Site coefficient F_v

Site Class	Mapped MCE_R spectral response acceleration parameter at 1-s period				
	$S_1 \leq 0.1$	$S_1 = 0.2$	$S_1 = 0.3$	$S_1 = 0.4$	$S_1 \geq 0.5$
A	0.8	0.8	0.8	0.8	0.8
B	1	1	1	1	1
C	1.7	1.6	1.5	1.4	1.3
D	2.4	2	1.8	1.6	1.5
E	3.5	3.2	2.8	2.4	2.4
F	See section 11.4.7 of ASCE				

Note: Use straight-line interpolation for intermediate values of S_s

5- S_{DS} at short period and at 1 second period S_{D1}, design earthquake spectral response acceleration parameters are determined from equation 3.3 and 3.4 respectively.

$$S_{DS} = 2/3\ S_{MS} \tag{3.3}$$

$$S_{D1} = 2/3\ S_{M1} \tag{3.4}$$

6- Determine SDC according to Table (11.6-1) and (11.6-2) in ASCE7-10 and Table 3.4 and 3.5 in this thesis.

Table 3.4: SDC based on short period response acceleration parameter

	Risk Category	
Values of S_{DS}	**I or II or III**	**IV**
$S_{DS} < 0.167$	A	A
$0.167 \leq S_{DS} < 0.33$	B	C
$0.33 \leq S_{DS} < 0.50$	C	D
$0.5 \leq S_{DS}$	D	D

Table 3.5: SDC based on 1-S period response acceleration parameter

	Risk Category	
Values of S_{D1}	**I or II or III**	**IV**
$S_{DS} < 0.067$	A	A
$0.067 \leq S_{DS} < 0.133$	B	C
$0.133 \leq S_{DS} < 0.2$	C	D
$0.2 \leq S_{DS}$	D	D

3.6.2 Determination of SDC for all models.

1- Risk category = I and I_e = 1
2- S_s = 0.52g, S_1 = 0.13g
3- Site class = D
4- F_a = 1.375 and F_v = 2.28 from Table (11.4-1) and (11.4-2) in ASCE 7-10
5- S_{MS} = 1.375* 0.52g = 0.715g

   S_{M1} = 2.28 * 0.13g = 0.2964g
6- $S_{DS} = 2/3 * 0.715 = 0.476g$

   $S_{D1} = 2/3 * 0.0.2964 = 0.197g$
7- According to S_{DS} and S_{D1} values, SDC is found based on Table (11.6-1) and (11.6-2) in ASCE7-10 and Table 3.4 and 3.5 in this thesis. Depending on the tables, SDC of all models is category C.

As it is found above, the SDC for all the models in this thesis is category C. by knowing the SDC, it can be decided that equivalent lateral force method can be performed to analyze and design of all the models. After assigning the SDC, the specific requirements for steel and reinforced concrete frames are delivered in Table 12.2-1 ASCE7-10, such as limitations on structural height and lateral load resisting and the table is shown in the appendix 3. According to Table 12.2-1 in ASCE7-10 steel ordinary moment-resisting frames OMRF, Steel ordinary concentrically braced frames (OCBF), Steel eccentrically braced frames (EBF), Steel and concrete composite ordinary shear walls (SCOSW) are used as structural systems in this thesis when the SDC is category C and the height of the buildings is within the limit. The supports of all models are assumed to be fixed and the connections between columns and beams are fixed as well, but the connection of bracing with the frames are stated as hinge connections.

Further information is required to define earthquake forces and designing the models ELF, the more required information to carry out earthquake forces in ETABS 2016 is demonstrated in Table 3.7 which have been selected in Table of 12.2-1 and 12.8-1 ASCE7-10

Table 3.6: Design coefficients and factors for seismic force-resisting systems and values of approximate period parameters Ct and x

Bracing pattern	**Response modification factor**	**Overstrength factor**	**Deflection implication factor**	**Ct**	**X**
OMRF	3.5	3	3	0.028	0.8
OCBF	3.25	2	3.25	0.02	0.75
EBF	8	2	4	0.03	0.75
SCOSW	5	2.5	4.5	0.02	0.75

3.7 Some Modeling Samples in ETABS for 2D Steel Frames and Combination with Shear Walls and Bracings.

Some of the models are shown in the figures below for further illustration

A- Ordinary moment resisting frames (OMRF)

Figure 3.7: Low rise building of OMRF

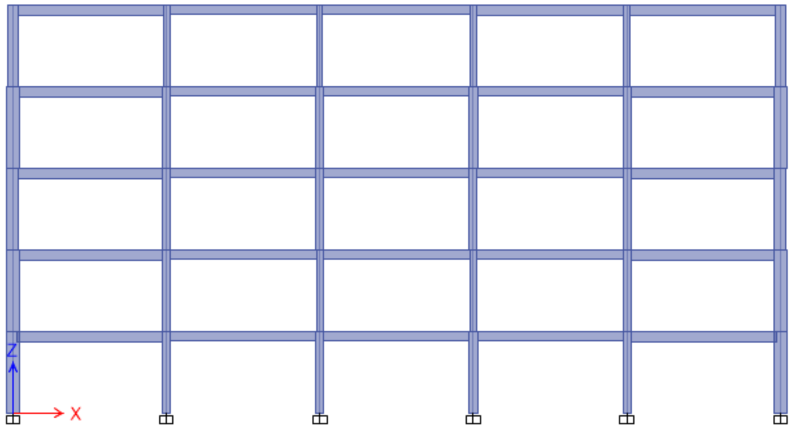


Figure 3.8: Medium rise building of OMRF

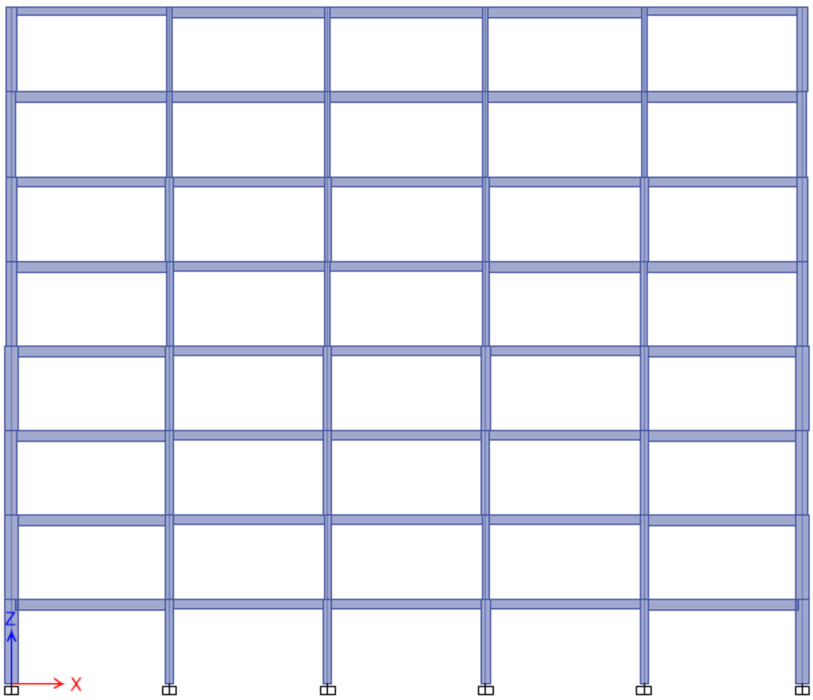


Figure 3.9: High rise building of OMRF

B- Ordinary concentrically braced frames (OCBF)

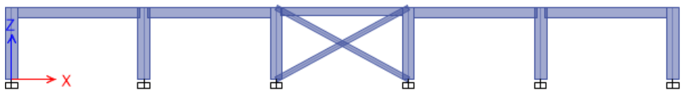


Figure 3.10: Low rise building of OCBF

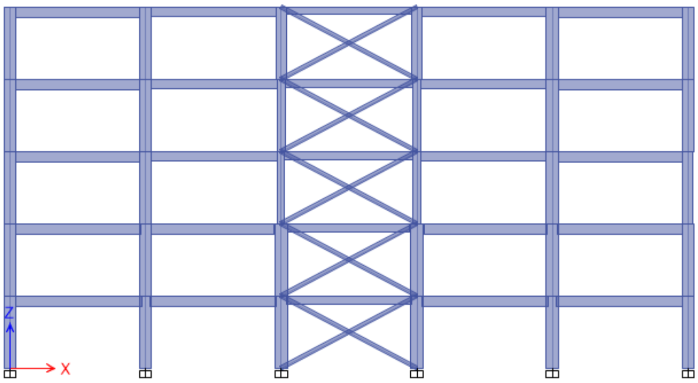


Figure 3.11: Medium rise building of OCBF

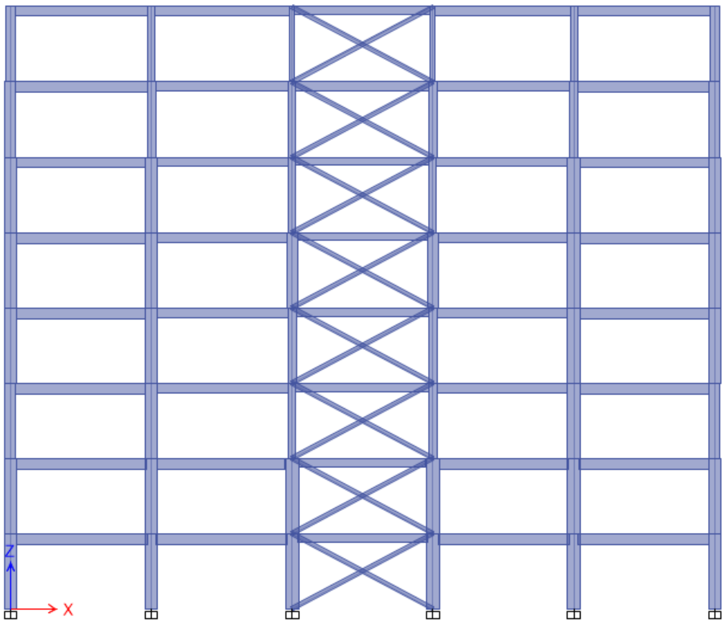


Figure 3.12: Medium rise building of OCBF

C- Steel and concrete composite ordinary shear walls (SCOSW)

Figure 3.13: Low rise building of SCOSW

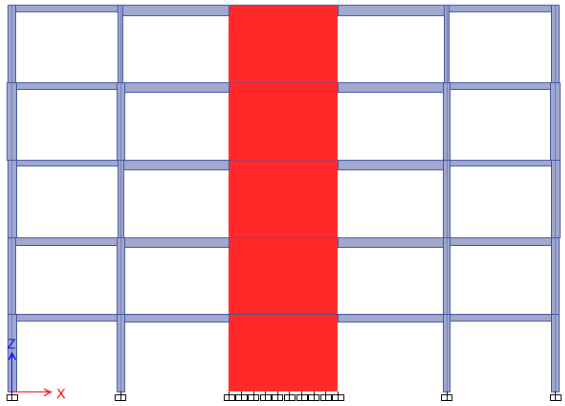


Figure 3.14: Medium rise building of SCOSW

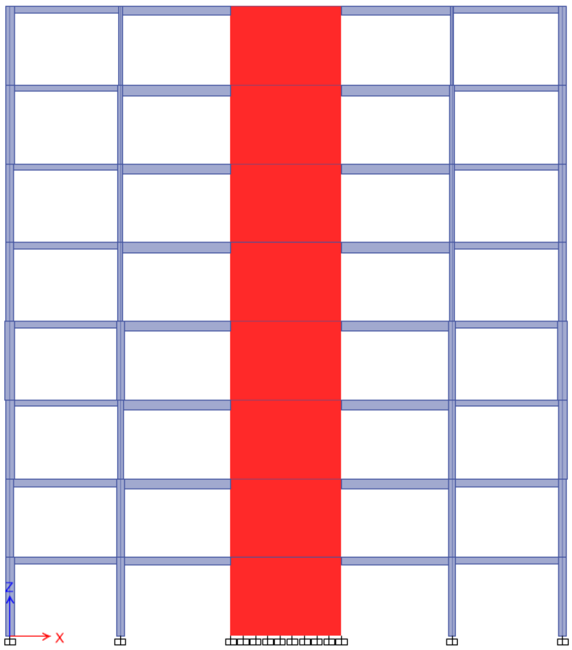


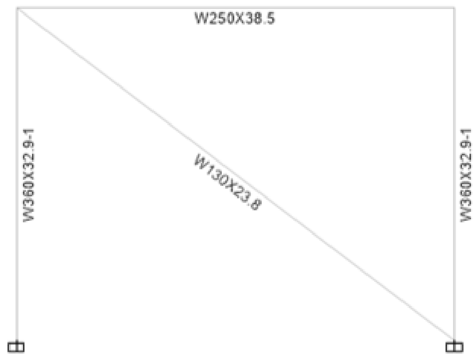
Figure 3.15: High rise building of SCOSW

3.8 Designed Sections of Steel Frames Considering Different Parameters (some Design Results)

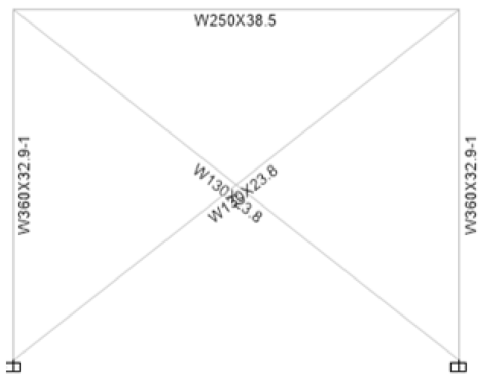
After loading, the steel models are designed based on the AISC360-10 code, applying LRFD method AISC360-10. The models containing shear walls are designed based on ACI 318-14. To analyze and design the models ETABS 2016 software program is employed. In the design processes of all models the American standard profile of type AISC W sections have been used for all models of steel. In the following figures the effect of some parameters are shown on the designed sections of the frames.

a) Different types of bracings

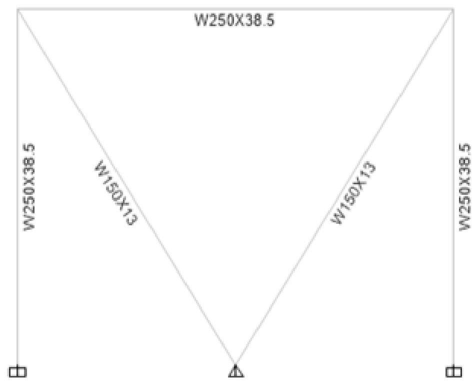
OMRF



Z-Bracing



X-Bracing



V-Bracing

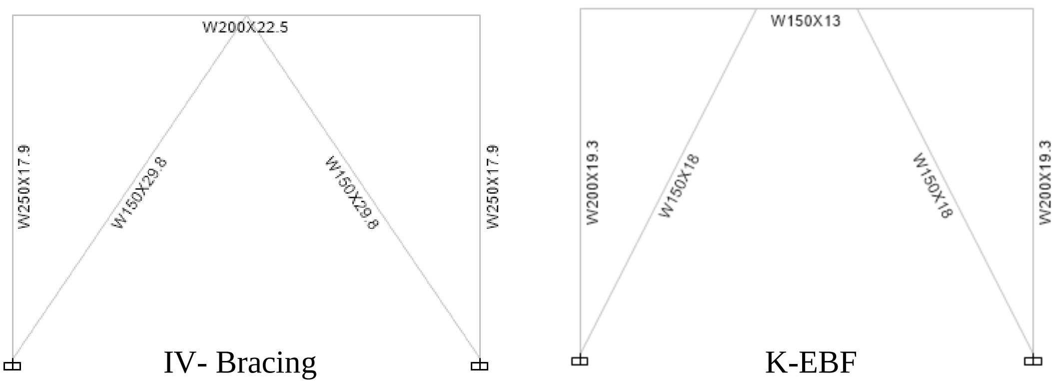
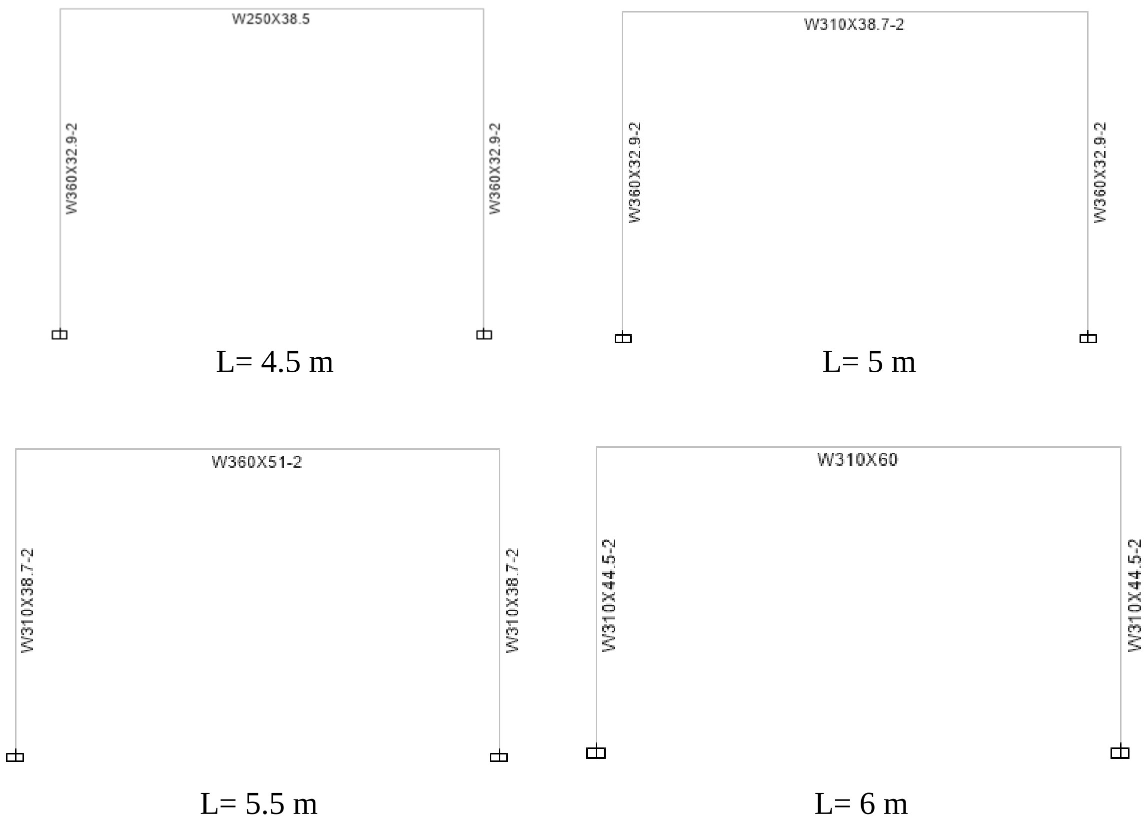


Figure 3.16: Different types of bracings

b) Span length (L)



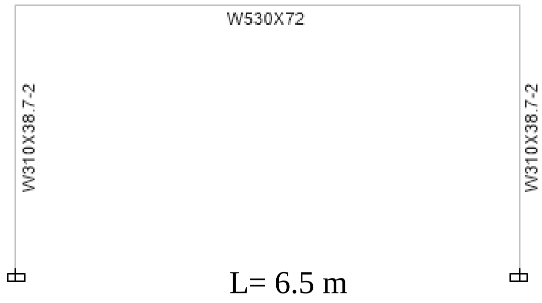


Figure 3.17: Effect of span length change on the designed sections of steel frames

c) Number of stories (S)

Fixed parameters N= 1, L= 6.5 m, H= 3.4 m and OMRF

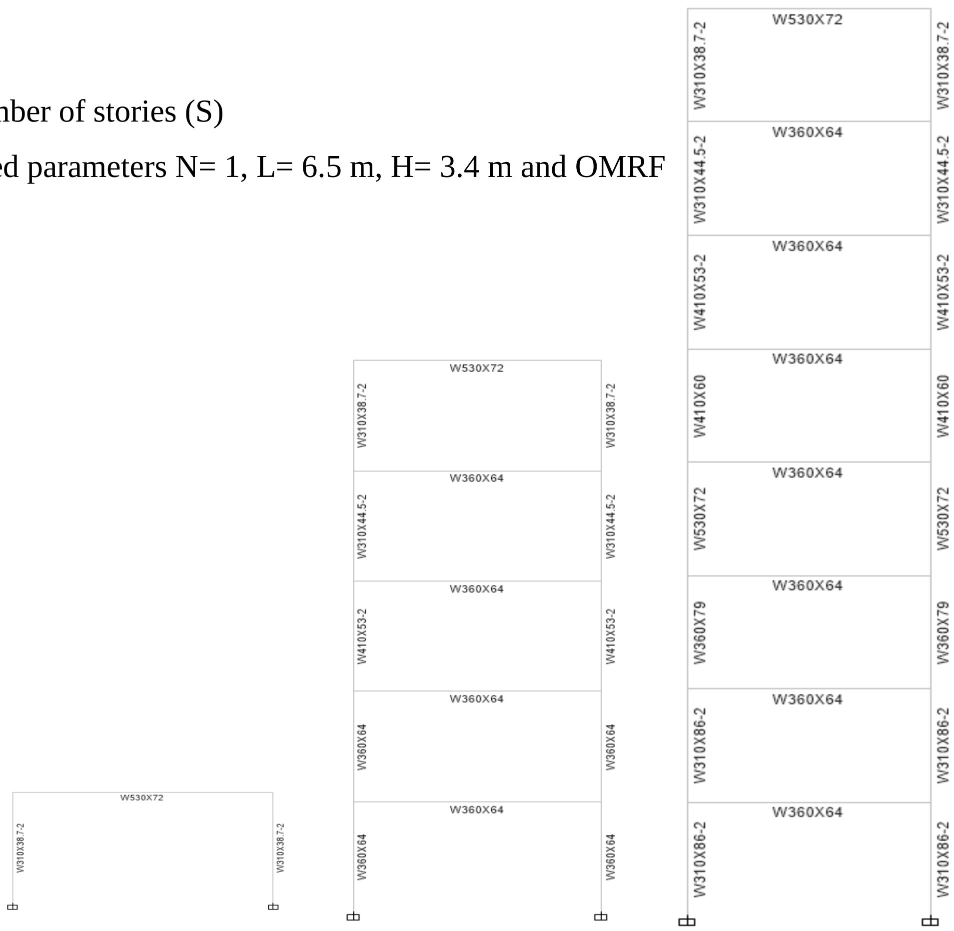


Figure 3.18: Number of stories

d) Number of spans (N): Fixed parameters S= L, H= 3.2, L= 4.5 m and OMRF

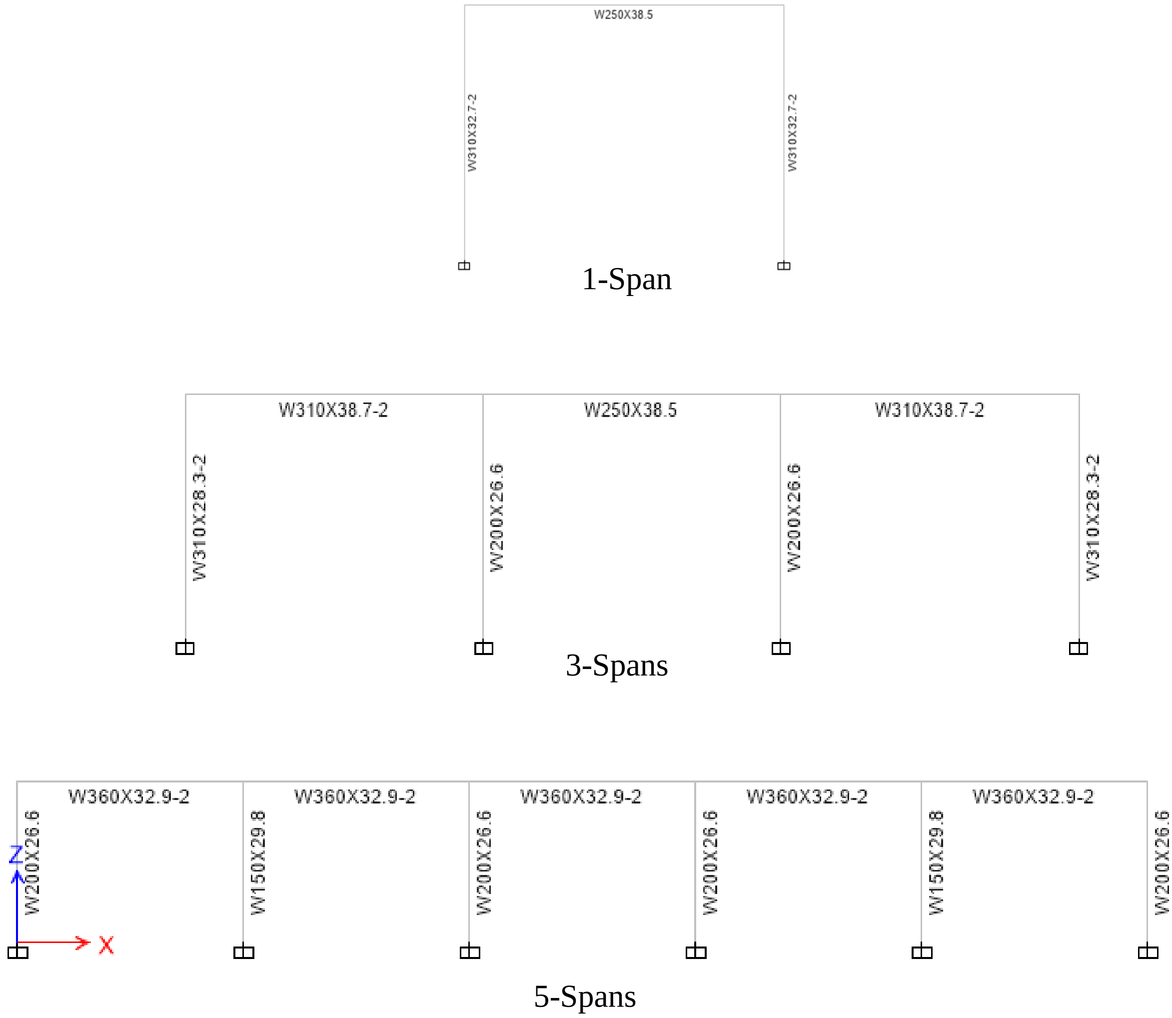


Figure 3.19: Number of spans

e) Story height (H): Fixed parameters N= 1, L= 4.5 m S= L and OMRF

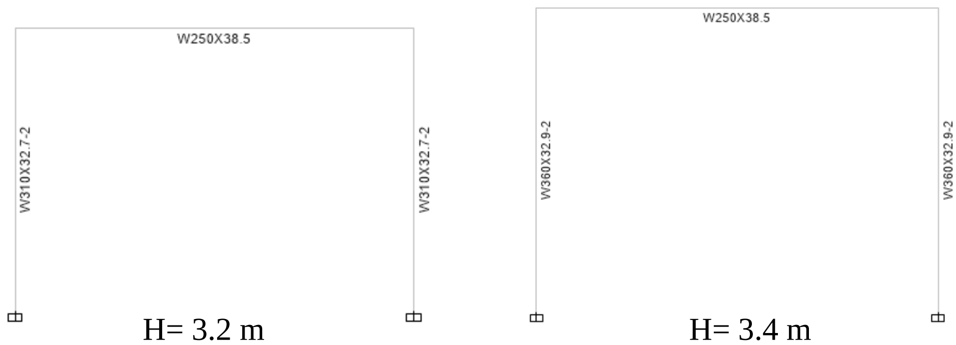


Figure 3.20: Story height change

After designing the models, pushover analysis is performed for all the models to evaluate the elastic stiffness factor, time period, maximum base shear and pushover curves of different types bracings and shear walls with changing parameters.

3.9 Pushover Analysis

Pushover analysis is one of the seismic analysis methods in which the structure is subjected to a lateral load and the lateral load on the structure is gradually increased and the structure undergoes non-linear behavior until a target displacement is achieved. The capacity and performance can be studied throughout pushover analysis, and the seismic demands of the building can be investigated. From pushover analysis a curve is drawn as shown in Figure 3.21.

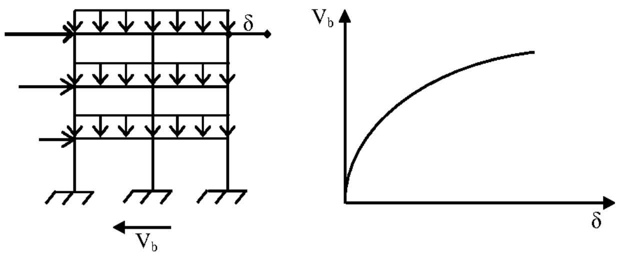


Figure 3.21: pushover curve (Padmakar Maddala, 2013)

3.10 Pushover Analysis Procedure

In this thesis displacement-controlled method of pushover analysis is used, and all the models are pushed up to rupture displacement at the controlled joint. The procedure of pushover analysis used in this study is illustrated below to find the elastic stiffness factor, natural time period, pushover curves and maximum base shear of all the frames.

a) two dimensional mathematical models of the steel frames are first created and designed using ELF
b) Hinges are assigned to the frames, bracing and shear walls.
c) 25% of live load, dead load and super dead load are initially applied to the 2D steel frames.
d) Then pushover analysis is defined and the load patterns of pushover analysis are assigned to a direction. The lateral load pattern considered in this study is the acceleration pattern, in the acceleration pattern the lateral load is increased

incrementally till the structure reaches the full capacity of the system which means drawing pushover curve up to failure of the structure.

e) After pushover analysis, a pushover curve is drawn which represents base shear and lateral displacement of the structure.

f) The values of elastic stiffness factor, natural time period is calculated using ASCE41-13 in the program, the procedure of calculation of these two parameters are explained with the help of a pushover curve in a sample below.

After drawing pushover curve Elastic stiffness factor and natural time period are calculated and the maximum base shear strength is extracted from pushover curve as follows:

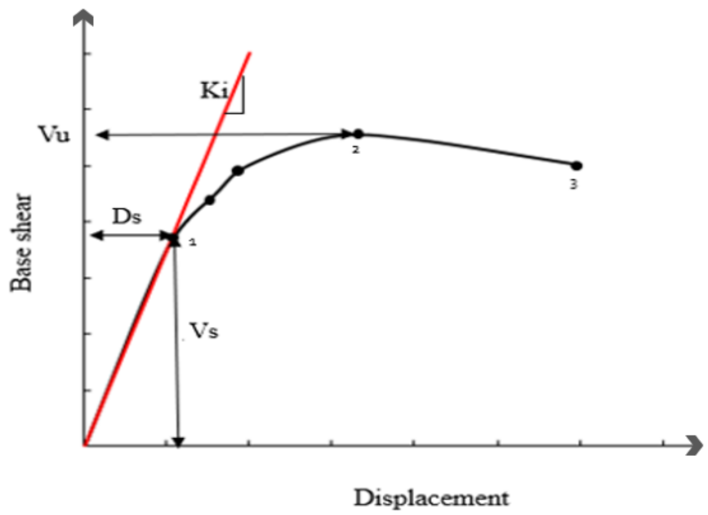


Figure 3.22: States of pushover curve

Number (1) denotes first plastic hinge formation of the structure where the elastic stiffness factor and natural time period are found for all models.

Number (2) represents maximum base shear (Vu)

Number (3) shows the maximum displacement the structure can endure (displacement at rupture)

$$K = Vs/ Ds \tag{3.5}$$

Where:

Vs = First significance yield strength (first hinge formation)

Ds = Displacement at first plastic hinge formation

K = Elastic stiffness factor

$$T = 2\pi\sqrt{\frac{m}{k}} \tag{3.6}$$

Where:

m = Gravity loads composed of dead loads and a specified portion of 25% live loads

K = Elastic stiffness factor

T = Natural time period

CHAPTER 4
RESULTS AND DISCUSSION

In this chapter analysis results of 2D steel frames are compared and discussed in graphs and tables for different parameters including different lateral load resisting systems, span length, number of spans, storey height change and number of stores, the comparison and evaluation of the steel frames is based on the elastic stiffness factor, time period, maximum base shear and pushover curves of the steel frames. This chapter is divided into four sections, the first section deals with the elastic stiffness factor of steel frames considering different parameters. The second section describes the effect of different parameters on the time period of the steel frames and the results are discussed in each section. In the third section, the results of maximum base shear for the steel frames have been shown and discussed. In the last section push over curve of the steel frames are demonstrated in figures and discussed.

In order to know the effect of one parameter on the elastic stiffness factor, time period, maximum base shear and pushover curve of the steel frames, other parameters are fixed. For better understanding, the symbols used in the graphs and tables are explained below:

S is the type of the building according to its height, low (one storey), medium (5 storey) and high-rise building (8 storey).
Number of spans is symbolled as N
The span length is symbolled as L
H is the height of the building
SW30 is the shear wall with compressive strength of concrete of 30 MPa
SW25 is the shear wall with compressive strength of 25 MPa

4.1 Elastic Stiffness Factor

Several other parameters are affected by elastic stiffness of a structure, and elastic stiffness is a function of some parameters which have been discussed through this section. The purpose of this section is to evaluate and compare the elastic stiffness of 2D steel frames for different types of bracings and shear walls considering different parameters such as span length, number of spans, number of stories and story height.

4.1.1 The effect of span length on the elastic stiffness of the 2D steel frames for varied types of concentrically and eccentrically bracing and shear walls

Changes in span length is an appropriate expression which effects the seismic behavior and stiffness of the frames. Apparently, changes in span length could have an important effect on the weight and designed sections of the frames so that any change in the span length will affect the elastic stiffness of the steel frames. From Figure 4.1 and Figure A.1.1 to A.1.8 show the elastic stiffness factor of the steel frames versus span length for different types of bracings and shear walls considering different parameters. From the below figure and Table 4.1 it is observed that with increasing span length for all types of bracings and shear walls the elastic stiffness factor of the steel frames is increased. From the figures and table 4.1 it is seen that after placing lateral load resisting systems in 2D steel frames the stiffness of the frames is increased, shear walls with compressive strength of 30 MPa has the highest elastic stiffness factor. Among the bracings, X type bracing has the maximum stiffness which is below the shear walls and OMRF has the lowest value of elastic stiffness factors. Figure 4.1, A.1.3 and A.1.6 demonstrates that when the storey numbers is increases, difference between elastic stiffness factor of shear walls and bracings decreases.

The parameters fixed for this figure and table are N = 1, S = L, H = 3.2 m

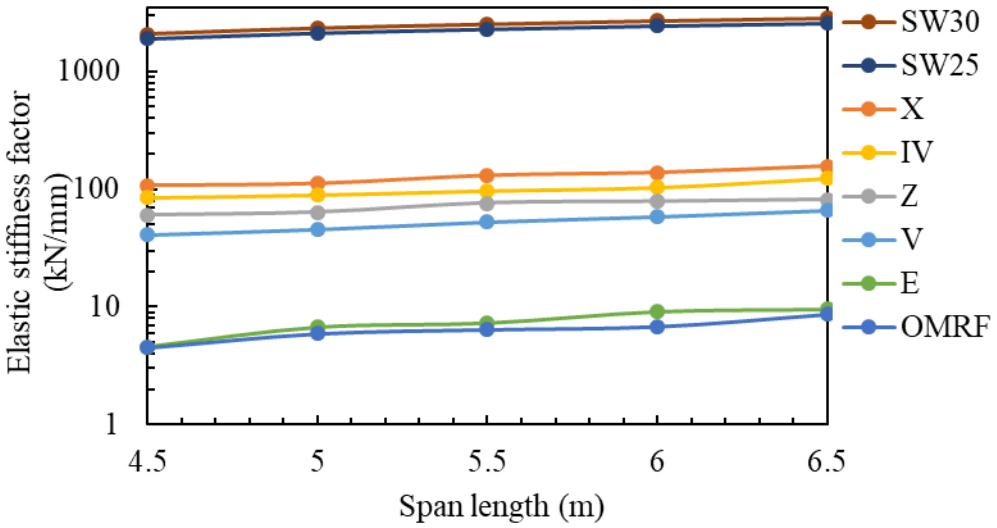


Figure 4.1: The elastic stiffness factor of the frames versus span length for different types of bracings and shear walls

Table 4.1: Results of elastic stiffness factor of different forms of bracings and shear walls as span length changes

Span length (m)	Elastic stiffness factor (kN/m)							
	OMRF	X	Z	IV	V	E	SW25	SW30
4.5	4.509	107.401	60.304	84.524	40.987	4.539	1892.303	2071.39
5	5.921	111.417	63.656	89.136	45.676	6.718	2125.141	2326.591
5.5	6.410	130.597	76.081	96.623	52.734	7.298	2294.76	2512.52
6	6.807	138.043	78.334	102.953	58.552	9.184	2450.195	2682.858
6.5	8.639	156.603	81.111	122.631	66.132	9.621	2579.658	2824.643

4.1.2 The effect of number of stories on the elastic stiffness factor of the steel frames for different types of bracings and shear walls

Figures 4.2 and A.1.9 through A.1.22 demonstrate the changes in the elastic stiffness factor of steel frames versus number of stories for different types of bracings and shear walls. From the figures and Table 4.2 it is seen that after placing lateral load resisting systems in 2D steel frames the elastic stiffness of the frames is increased, shear walls with compressive strength of 30 MPa has the highest elastic stiffness factor. Among the bracings, X type bracing has the maximum stiffness which is below the shear walls and OMRF has the lowest value of elastic stiffness factors, Figures 4.2 and A.1.9 through A.1.22 and Table 4.2 demonstrate that with increasing number of stories the elastic stiffness factor decreases for all types of bracings and shear walls, as it is seen from Figure 4.2 and A.1.9 to A.1.12, the decrease is the same when the span length is changed as well. From figure 4.2, A.1.13 and A.1.18 it is observed that when the number of spans changed from 1 to 3 and then to 5, the percentage of decreasing in elastic stiffness factor decreases while the number of stories changed.

Assuming fixed parameters for the figure and table N = 1, L = 4.5 m and H = 3.2 m

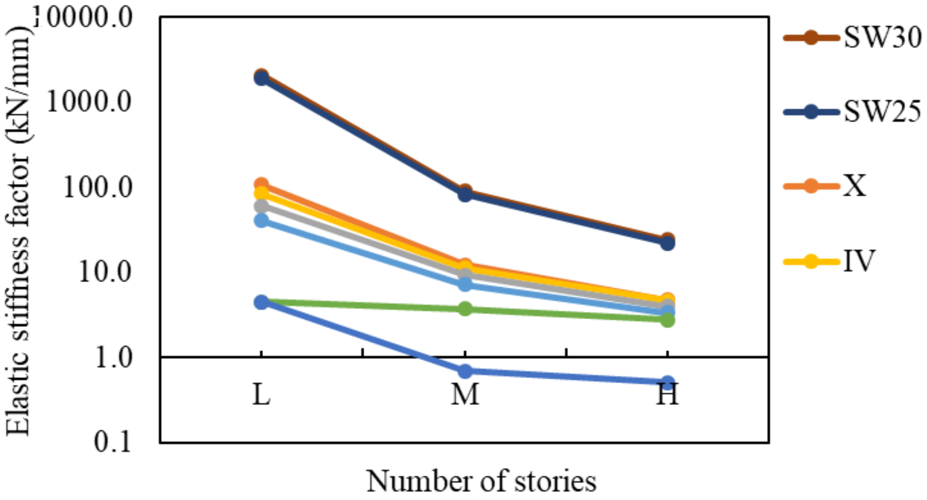


Figure 4.2: The elastic stiffness factor of the frames versus the number of stories for different types of bracings and shear walls

Table 4.2: Results of elastic stiffness factor of different forms of bracings and shear walls as number of storey changes

Number	Elastic stiffness factor (kN/m)							
of stories	OMRF	X	Z	IV	V	E	SW25	SW30
L	4.509	107.401	60.304	84.524	40.987	4.539	1892.3	2071.39
M	0.694	12.292	9.408	11.400	7.187	3.742	82.147	89.841
H	0.507	4.725	4.002	4.669	3.381	2.804	22.043	24.151

4.1.3 The effect of number of spans on the elastic stiffness factor of the steel frames for different types of bracings and shear walls

The following figure and table demonstrate the changes in the elastic stiffness factor of steel frames versus number of spans for different types of bracings and shear walls. From Figure 4.3 and A.1.23 to A.1.26 and Table 4.3 it is detected that when the building is low (one store), the elastic stiffness factor decreases for shear walls and X, IV, Z and V type bracings with increasing number of spans, but the elastic stiffness factor of OMRF and E bracing type is increased when the number of spans is changed.

Assuming fixed parameters for the figure and table S= L, L = 4.5 m and H = 3.2 m

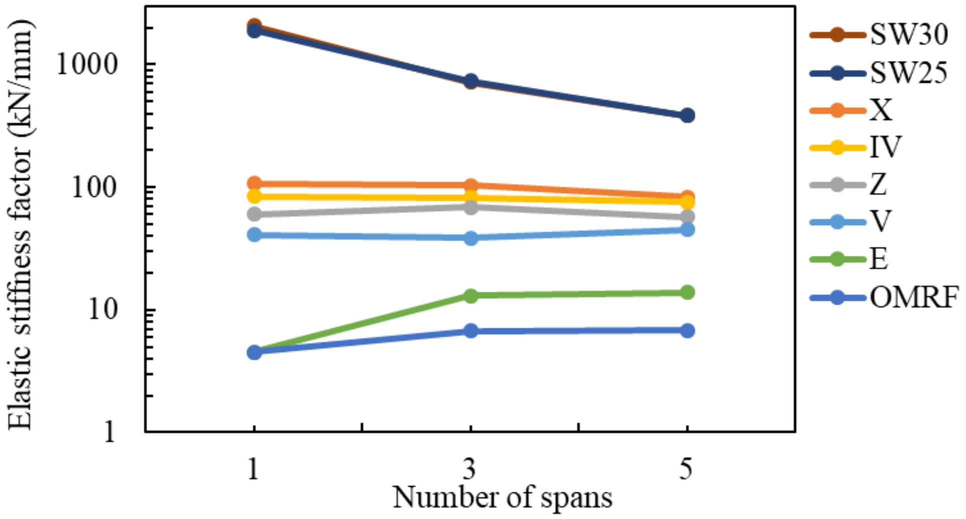


Figure 4.3: The elastic stiffness factor of the frames versus the number of spans for different types of bracings and shear walls

Table 4.3: Results of elastic stiffness factor of different forms of bracings and shear walls as number of spans changes

Number of spans	Elastic stiffness factor (kN/mm)							
	OMRF	X	Z	IV	V	E	SW25	SW30
1	4.509	107.401	60.304	84.524	40.987	4.539	1892.303	2071.39
3	6.723	103.286	68.864	81.889	38.536	13.069	732.738	714.904
5	6.798	82.823	57.344	75.680	45.068	13.867	381.629	385.236

From Figure 4.4 and A.1.27 to A.1.35 it is found that for medium and high rise 2D steel frames, the elastic stiffness factor is increased for all type of bracings and OMRF, but shear walls having 25 and 30 MPa of compressive strength of concrete are almost the same or a little decreased when the number of spans is increased as it is seen in Table 4.4. As a result, it can be said the effect of number of spans on the elastic stiffness factor depends on the height of the building.

Assuming fixed parameters for the figure and table S = M, L = 4.5 m and H = 3.2 m

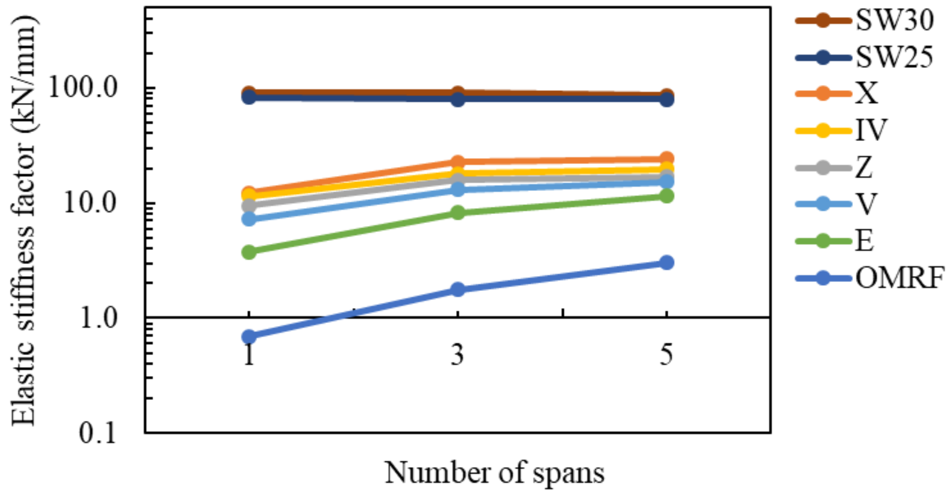


Figure 4.4: The elastic stiffness factor of the frames versus the number of spans for different types of bracings and shear walls

Table 4.4: Results of elastic stiffness factor of different forms of bracings and shear walls as number of spans changes

Number of spans	Elastic stiffness factor (kN/mm)							
	OMRF	X	Z	IV	V	E	SW25	SW30
1	0.694	12.292	9.408	11.400	7.187	3.742	82.147	89.841
3	1.776	22.704	15.706	17.875	13.003	8.258	79.323	90.589
5	3.021	23.960	16.803	19.619	15.282	11.415	78.798	85.369

4.1.4 The effect of story height change on the elastic stiffness factor of the steel frames for different types of bracings and shear walls

The following Figures 4.4, A.1.36 to A.1.43 and Table 4.5 demonstrate the changes in the elastic stiffness factor of steel frames versus story height change for different types of bracings and shear walls. From the figures and Table 4.5 it is seen that the elastic stiffness factor of steel frames for all types of bracing and shear walls is decreased with increasing the height of the stories from 3.2 m to 3.4 m. Figure 4.5, A.1.38 and A.1.41 demonstrate that when the building is low the percentage of decrease in the elastic stiffness factor of the steel frames is less compared to medium and high rise building, the reason behind this is that, the overall height of the medium and high rise buildings is much increased when the storey height is increased. As a result, the percentage of decrease in the elastic stiffness factor of high rise 2D steel frames is higher than others as it is seen from Figure A.1.41 through A.1.43 the slope is much steeper.

Assuming fixed parameters for the figure and table S= L, L = 4.5 m and N = 1

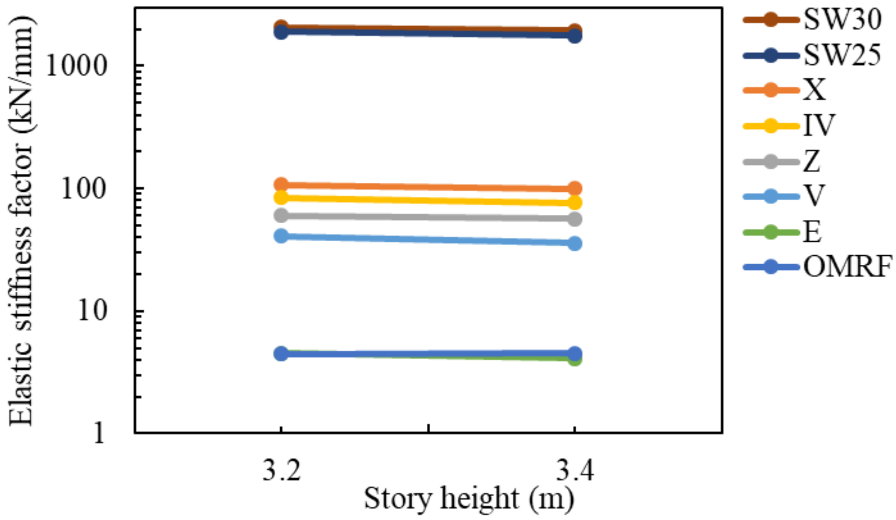


Figure 4.5 The elastic stiffness factor of the frames versus the story height change for different types of bracings and shear walls

Table 4.5: Results of elastic stiffness factor of different forms of bracings and shear walls as storey height changes

Storey	Elastic stiffness factor (kN/m)							
height (m)	OMRF	X	Z	IV	V	E	SW25	SW30
3.2	4.5	107.401	60.304	84.524	40.987	4.539	1892.3	2071.39
3.4	4.5	99.561	56.604	76.163	35.940	4.123	1770.28	1937.786

4.1.5 The effect of different lateral resisting systems on the elastic stiffness factor of the steel frames

Different lateral load resisting systems are analyzed using pushover analysis giving different initial lateral stiffness factor. The lateral load resisting systems (LLRS)used in the thesis are shown in Figure 4.6, each type of LLRS is used for 90 models and the average elastic stiffness factor of each of them are shown in Figure 4.6. As it is seen from Figure 4.6 shear wall with compressive strength of concrete 30 MPa is stiffer than other types of bracing and then SW25. Among the bracings X type concentrically bracing is stiffer than others. And OMRF has the minimum value of elastic stiffness factor. Figure 4.7 demonstrates comparison of elastic stiffness factor of different lateral resisting systems with respect to OMRF. It is found that SW30 is 156 times larger than OMRF. And SW 25 is 96 times larger than OMRF. Other results are shown in the figure as well.

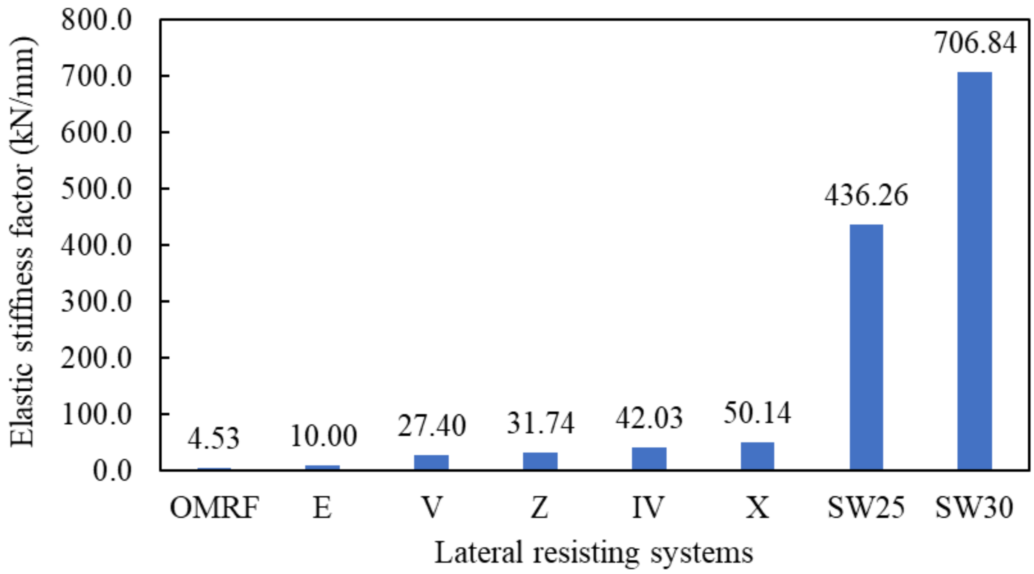


Figure 4.6: Average elastic stiffness factor of different lateral resisting systems

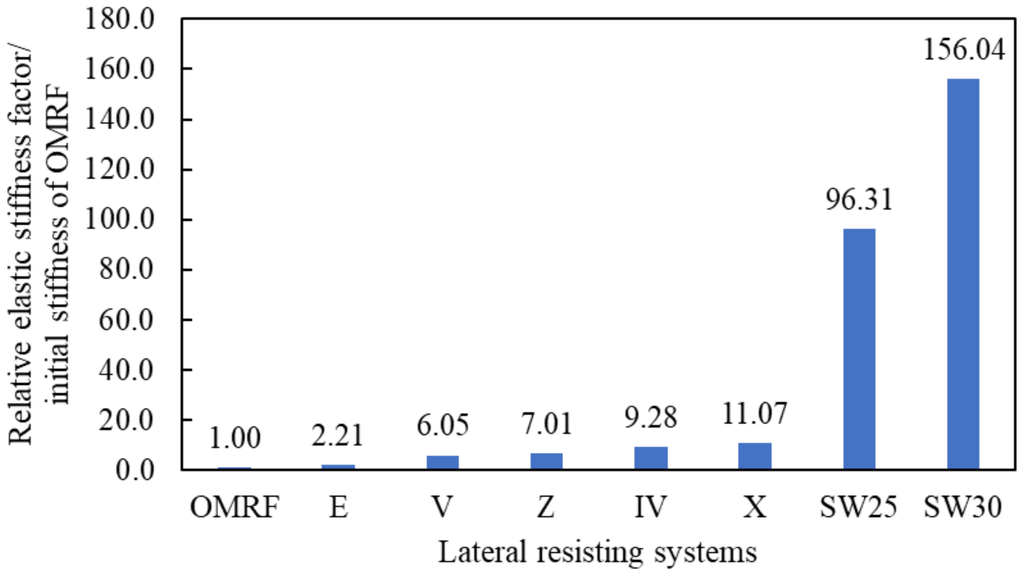


Figure 4.7: Comparison of elastic stiffness factor of different lateral resisting systems with respect to OMRF

4.2 Factors Affecting Time Period

in this section parameters affecting the natural time period of structure are showed in figures and tables, the parameters are span length, storey height, number of storeys, number of spans and different lateral load resisting systems.

4.2.1 The effect of span length on the natural time period of the steel frames for different types of bracing and shear walls

Changes in span length is an appropriate expression which effects the time period and seismic performance of the frames. Apparently, the changes in span length could have an important effect on the weight and stiffness of the frames so that any change in the span length will decrease or increase the natural time period of the steel frames, form Figure 4.8 and A.1.44 to through A.1.51 and Table 4.6 show the natural time period of the frames versus span length for different types of bracings and shear walls. From the figures and table 4.6 it is determined that after inserting lateral load resisting systems into the 2D steel frames the natural time period is decreased. SW30 has the minimum natural time period among LLRs, and among the bracings the natural time period of X type bracing is less than other type of bracings. As it is seen from Figure 4.8 and A.1.44 to through A.1.51 and table

4.6, when the span length increases the natural time period decreases for all types of bracings because the percentage increase in stiffness as a result of span length change is higher than the percentage increase in mass. But the natural time period remains the same for shear walls.

The parameters fixed for this figure and table are N = 1, S = L, H = 3.2 m

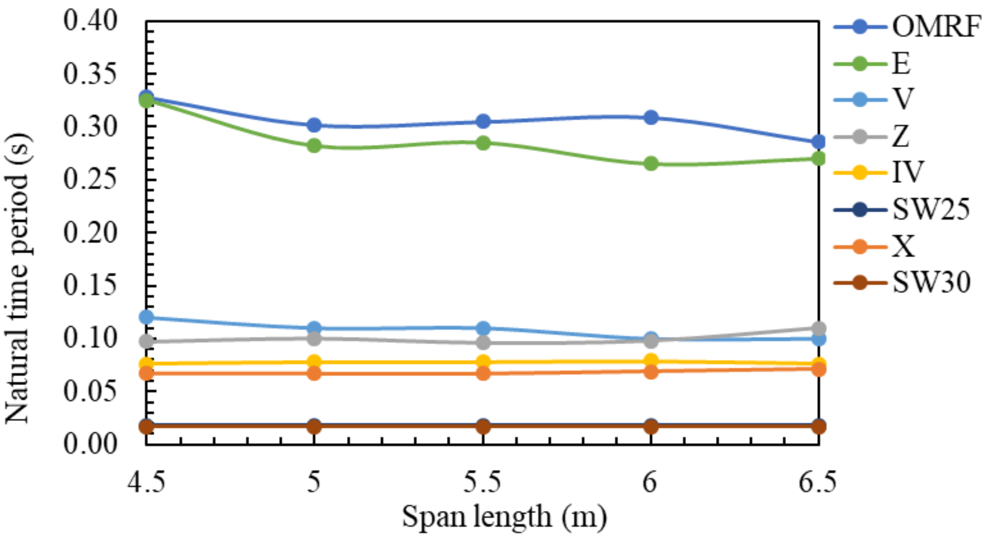


Figure 4.8: The natural time period of the frames versus span length for different types of bracings and shear walls

Table 4.6: Results of natural time period of different forms of bracings and shear walls as span length changes

Span length (m)	Natural time period (s)							
	OMRF	X	Z	IV	V	E	SW25	SW30
4.5	0.328	0.067	0.097	0.076	0.12	0.325	0.018	0.017
5	0.302	0.067	0.100	0.078	0.11	0.282	0.018	0.017
5.5	0.305	0.067	0.096	0.078	0.11	0.285	0.018	0.017
6	0.309	0.066	0.098	0.079	0.1	0.265	0.018	0.017
6.5	0.286	0.066	0.0970	0.076	0.1	0.27	0.018	0.017

4.2.2 The influence of storey number change on the natural time period of the steel frames for shear walls and bracings

Changing storey numbers is an important parameter which affects the natural time period of structures. Figure 4.9 and A.1.52 to A.1.65 and Table 4.7 demonstrate the changes in the natural time period of steel frames versus number of stories for different types of bracings and shear walls. From the figures and the table, it is observed that when the storey numbers is increases, the natural time period of the 2D steel frames increases due to increase in the mass of the frames and decrease in the overall elastic stiffness of the frames, as a result it can be said high rise buildings have larger natural time period than low rise buildings.

Assuming fixed parameters for the figure and table N = 1, L = 4.5 m and H = 3.2 m

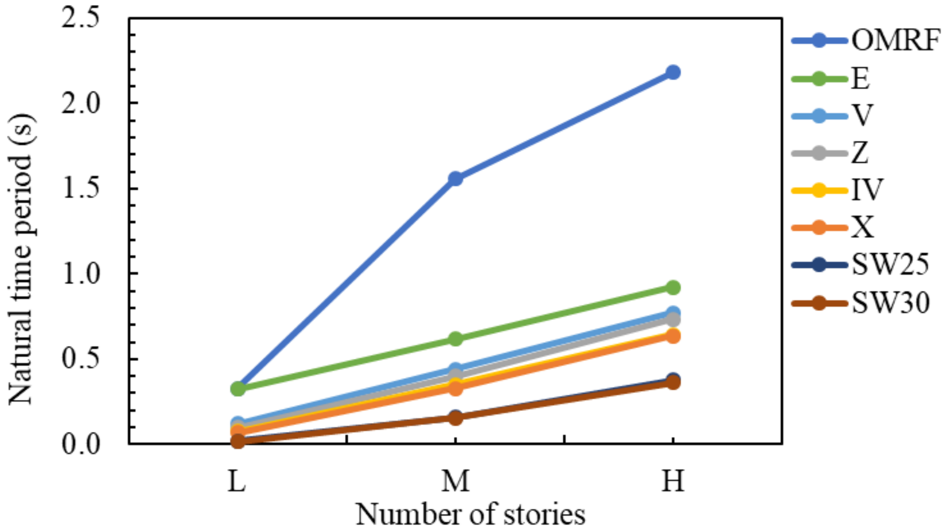


Figure 4.9: The natural time period of steel frames versus number of stories for different Types of bracings and shear walls

Table 4.7: Results of natural time period of different forms of bracings and shear walls as storey number changes

Number of stories	Natural time period (s)							
	OMRF	X	Z	IV	V	E	SW25	SW30
L	0.328	0.067	0.097	0.076	0.120	0.325	0.018	0.017
M	1.556	0.326	0.400	0.350	0.440	0.619	0.16	0.153
H	2.181	0.636	0.733	0.642	0.770	0.923	0.378	0.36

4.2.3 The effect of number of spans on the natural time period of the steel frames for different types of bracings and shear walls

Figure 4.10 and A.1.66 through A.1.79 and Table 4.8 demonstrate the changes in the natural time period of steel frames versus number of spans for different types of bracings and shear walls. From the figures and the table, it is determined that after inserting lateral load resisting systems into the 2D steel frames the natural time period is decreased. SW30 has the minimum natural time period among LLRs, and among the bracings the natural time period of X type bracing is less than other type of bracings. As it is seen from figures and the below table, when the number of spans increases from 1 to 3 and 5, the natural time period increases for all types of bracings and shear walls because the percentage increase in stiffness as a result of increasing in number of spans is lower than the percentage increase in mass, which means the natural time period is a function of mass and stiffness of the structure.

Assuming fixed parameters for the figure and table S= L, L = 4.5 m and H = 3.2 m

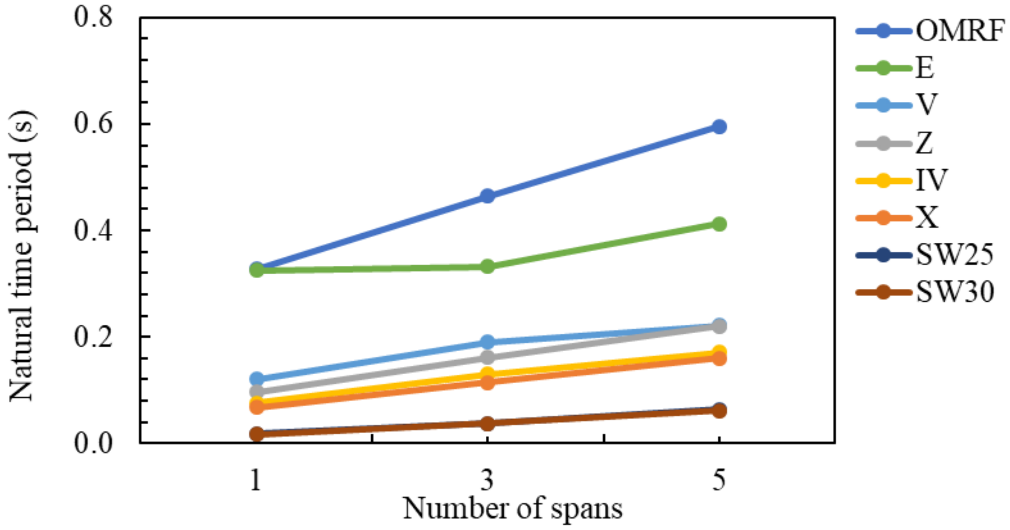


Figure 4.10: The natural time period of the frames versus the number of spans for different types of bracings and shear walls

Table 4.8: Results of natural time period of different forms of bracings and shear walls as number of spans changes

Number of spans	Natural time period (s)							
	OMRF	X	Z	IV	V	E	SW25	SW30
1	0.328	0.067	0.097	0.076	0.12	0.325	0.018	0.017
3	0.465	0.115	0.162	0.130	0.19	0.332	0.037	0.037
5	0.596	0.16	0.220	0.171	0.222	0.413	0.064	0.062

4.2.4 The effect of story height change on the natural time period of the steel frames for different types of bracings and shear walls

The following figures and table demonstrate the changes in the natural time period of steel frames versus story height change for different types of bracings and shear walls. From figure 4.11 and A.1.80 through 87 and Table 4.9, it is found that as the height of stories increases, the natural time period of the 2D steel frames is increased since the stiffness of 2D steel frames decreases and the mass is all most constant. The higher the building makes the natural time period higher.

Assuming fixed parameters for the figure and table S= L, L = 4.5 m and N = 1

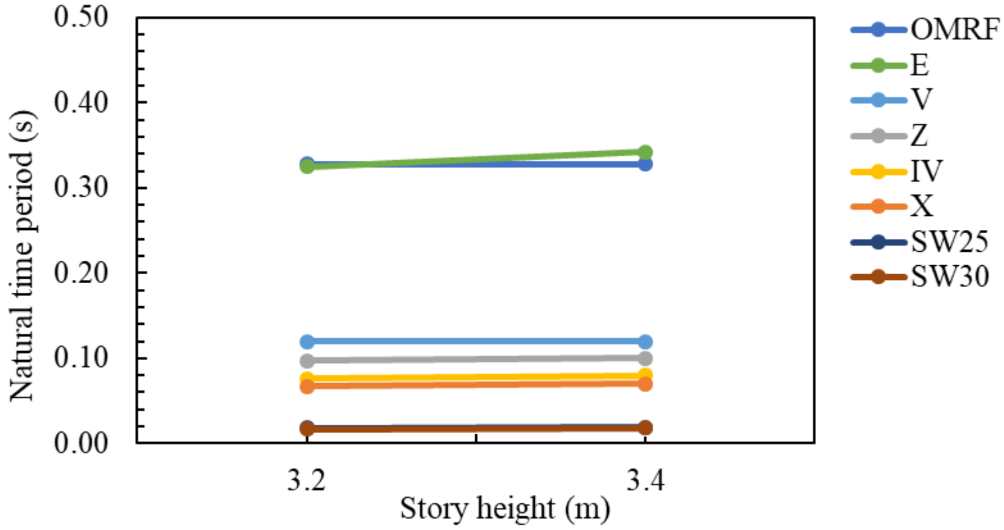


Figure 4.11: The natural time period of the frames versus the story height change for Different types of bracings and shear walls

Table 4.9: Results of natural time period of different forms of bracings and shear walls as story height changes

Story height	Natural time period (s)							
(m)	OMRF	X	Z	IV	V	E	SW25	SW30
3.2	0.328	0.067	0.097	0.076	0.12	0.325	0.018	0.017
3.4	0.33	0.07	0.100	0.080	0.12	0.342	0.019	0.018

4.2.5 The effect of different lateral resisting systems on natural time period of the steel frames

Different lateral load resisting systems are analyzed using non-linear static (pushover) analysis giving different natural time period. The lateral load resisting systems (LLRS) used in the study are shown in Figure 4.12 and 4.13, each type of LLRS is used for 90 models and the average natural time period of each of them are shown in figure 4.12 and 4.13. As it is seen from Figure 4.12 OMRF has the maximum natural time period than other types of LLRs since it is not stiff enough and displaces much more under lateral loads, shear wall with compressive strength of concrete 30 MPa is stiffer than other types of bracing so that its natural time period is minimum. Among the bracings, X type concentrically bracing has lesser natural time period than others. Figure 4.13 demonstrates comparison of natural time period of different lateral resisting systems with respect to SW30. It is found that the natural time period of OMRF is 5.7 times larger than SW30. Other results are shown in Figure 4.13 as well.

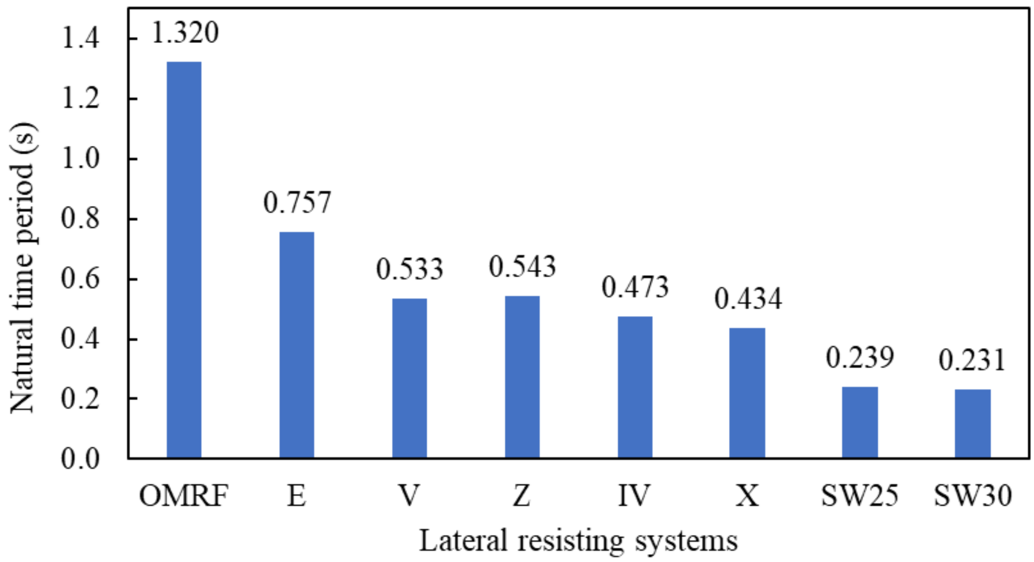


Figure 4.12: Average natural time period of different lateral resisting systems

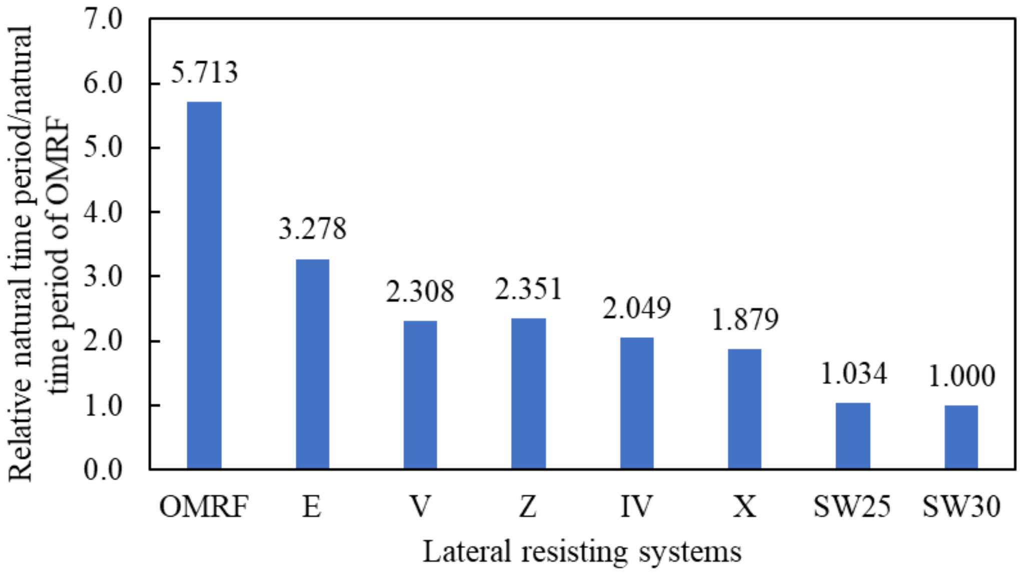


Figure 4.13: Comparison of natural time period of different lateral resisting systems with respect to SW30

4.3 Maximum Base Shear

In this section, the results of different parameters affecting the maximum base shear of 2D steel frames are shown in graphs and tables and discussed. The maximum base shear is the capacity of the frames that can withstand under lateral load it is not the designed base shear which is used to design the frames.

4.3.1 The effect of span length on the maximum base shear of the steel frames for different types of bracing and shear walls

Span length change can have an effect on the maximum base shear of steel frames. Therefor in order to evaluate that effect, the maximum base shear of the frames versus span length change for different types of bracings and shear walls are demonstrated from Figure 4.14 , A.1.88 to A.1.95 and Table 4.10, It can be seen from the graphs and the below table as the span length increases, the steel frames can withstand more base shear which means as the span length increases, the maximum base shear (maximum capacity) increases as well. From figures it is found that, SW30 showed the maximum base shear for low rise buildings, but X type bracing exhibits higher performance for high rise building as it is seen from Figure A.1.93 to A.1.95. It can be said for high rise buildings, the X type bracing is more preferable to be used to resist lateral loads (Kevadkar, M. D., Parishith J., Praveen Kumar S., Ubaid K and Shende, M. P. M).

The parameters fixed for this figure and table are N = 1, S = L, H = 3.2 m

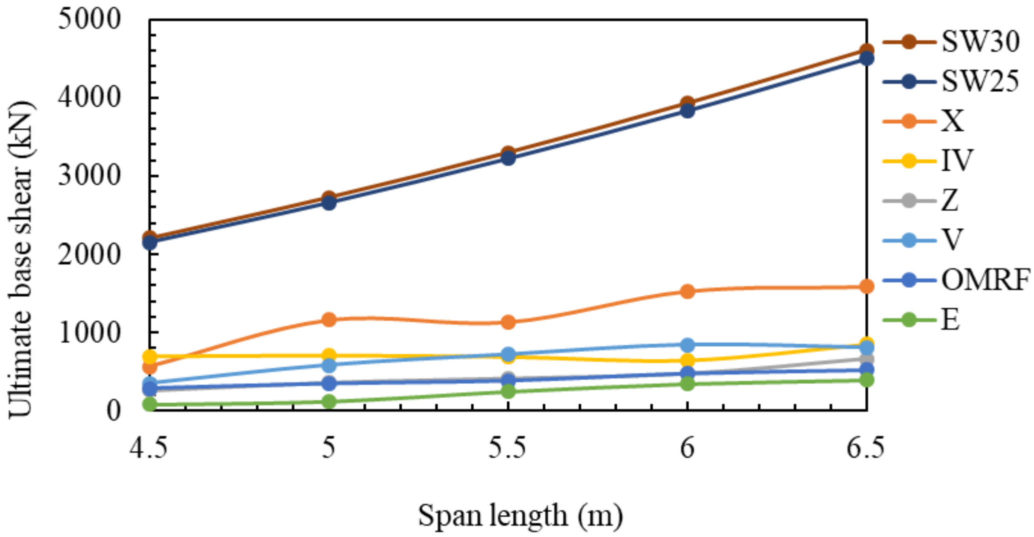


Figure 4.14: The maximum base shear strength of the frames versus span length for different types of bracings and shear walls.

Table 4.10: Results of maximum base shear of different forms of bracings and shear walls as span length changes

Span	Maximum base shear (kN)							
length (m)	OMRF	X	Z	IV	V	E	SW25	SW30
4.5	285	563	260	692	351.2	77.2	2152	2206
5	346	1158	359.2	704	582	116.6	2657	2726
5.5	383	1131	412	685.6	717	245.3	3220.9	3298
6	476	1529	472.5	637	840	345.0	3834	3926
6.5	519	1588	658	853	804.4	394.0	4501	4604

4.3.2 The effect of number of stories on the maximum base shear of the steel frames for different types of bracings and shear walls

The following figures demonstrates the changes in the maximum base shear of steel frames versus number of stories for different types of bracings and shear walls. As it is seen from Figure 4.15, A.1.96, A.1.97 and Table 4.11, the maximum base shear is decreased for SW30 and SW25 when the number of stories increases. In the other hand, the maximum base shear is increased for bracings. And it is found for high rise building the x type bracing, the capacity of the frames is higher than shear walls as it is found in some other references (Kevadkar, M. D., Parishith J., Praveen Kumar S., Ubaid K and Shende, M. P. M).

Assuming fixed parameters for the figure and table N = 1, L = 4.5 m and H = 3.2 m

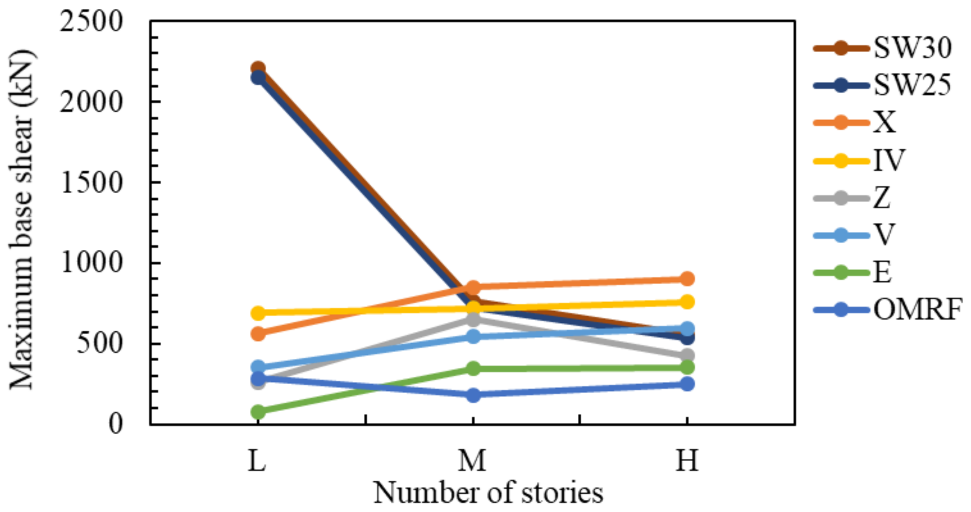


Figure 4.15: The maximum base shear of steel frames versus number of stories for different types of bracings and shear walls

Table 4.11: Results of maximum base shear of different forms of bracings and shear walls as number of storeys changes

Number of stories	Maximum base shear (kN)							
	OMRF	X	Z	IV	V	E	SW25	SW30
L	285.0	563	260	692	351.2	77.15	2152	2206
M	183.0	849.46	652.2	717.2	543.4	346	723	764
H	249.0	902	424	757	596.4	351.46	537	556

4.3.3 The effect of number of spans on the maximum base shear of the steel frames for different types of bracings and shear walls

The following figures and table demonstrate the changes in the maximum base shear of steel frames versus number of spans for different types of bracings and shear walls, as it is seen from Figure 4.16, A.1.98, A.1.99 and Table 4.12 the maximum base shear increases when the number of spans is increased. The percentage of increase in high rise buildings is higher compared to low rise buildings as it is observed in Figure A.1.98 and A.1.99.

Assuming fixed parameters for the figure and table S= L, L = 4.5 m and H = 3.2 m

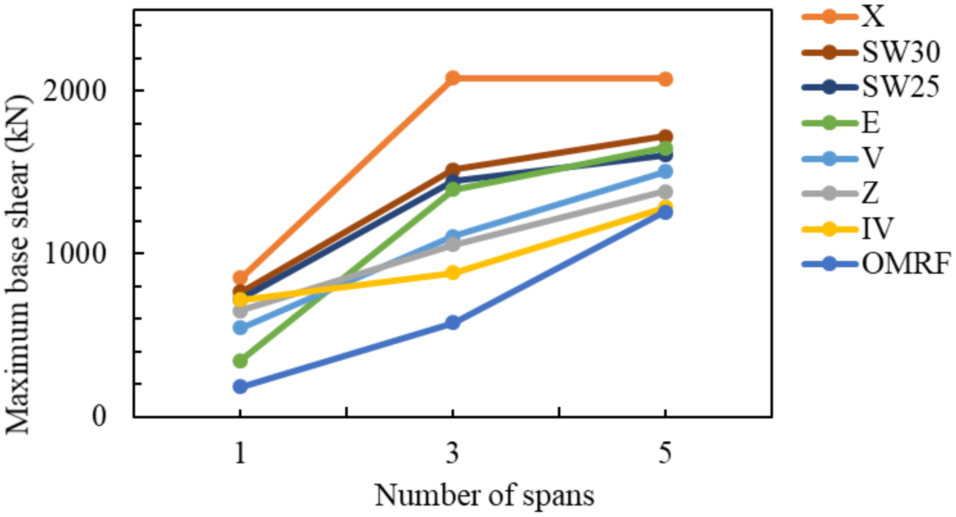


Figure 4.16: The maximum base shear of steel frames versus number of spans for different types of bracings and shear walls

Table 4.12: Results of maximum base shear of different forms of bracings and shear walls as number of spans changes

Number of spans	Maximum base shear (kN)							
	OMRF	X	Z	IV	V	E	SW25	SW30
1	183	849.46	652.2	717.2	543.4	346	723	764
3	576	2078	1055.2	884	1107	1393.5	1446	1517
5	1256	2076	1381	1284	1502	1649	1605	1721

4.3.4 The effect of story height change on the maximum base shear of the steel frames for different types of bracings and shear walls

The following figures and table demonstrate the changes in the maximum base shear steel frames versus story height change for different types of bracings and shear walls. As it is seen from Figure 4.17, A.1.100, A.1.101 and Table 4.13, as the storey height increases, the maximum base shear is reduced almost for all types of lateral load resisting systems. And as it is seen from the Figure A.1.100 and A.1.101as the building changes from low rise to high rise, X type bracing has the maximum base shear compared to other lateral load resisting systems.

Assuming fixed parameters for the figure and table S= H, L = 4.5 m and N = 1

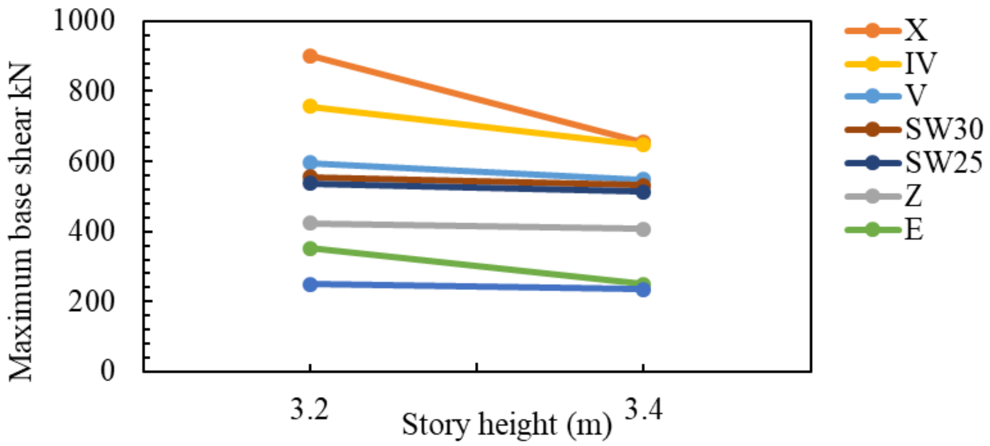


Figure 4.17: The maximum base shear of steel frames versus story height change for different types of bracings and shear walls

Table 4.13: Results of maximum base shear of different forms of bracings and shear walls as storey height changes

Story height	Maximum base shear (kN)							
(m)	OMRF	X	Z	IV	V	E	SW25	SW30
3.2	249	902	424	757	596.4	351.46	537	556
3.4	234.26	655	408	647.9	548.6	249.6	514	531

4.3.5 The effect of different lateral resisting systems on maximum base shear of the steel frames

Different lateral load resisting systems are analyzed using pushover analysis giving different maximum base shear. The lateral load resisting systems (LLRS)used in the thesis are shown in Figure 4.18, each type of LLRS is used for 90 models and the average maximum base shear of each of them showed in figure 4.18. As it is seen from the figure, shear wall with compressive strength of concrete 30 MPa has maximum base shear than other types of lateral load resisting systems and then SW25. Among the bracings X type concentrically bracing is stronger than others in terms of maximum base shear. And OMRF has the minimum value of maximum base shear. Figure 4.19 demonstrates comparison of maximum base shear of different lateral resisting systems with respect to OMRF. It is found that the maximum base shear of SW30 is 2.889 times larger than OMRF. And SW25 is 2.754 times larger than OMRF. Other results are shown in the figure as well.

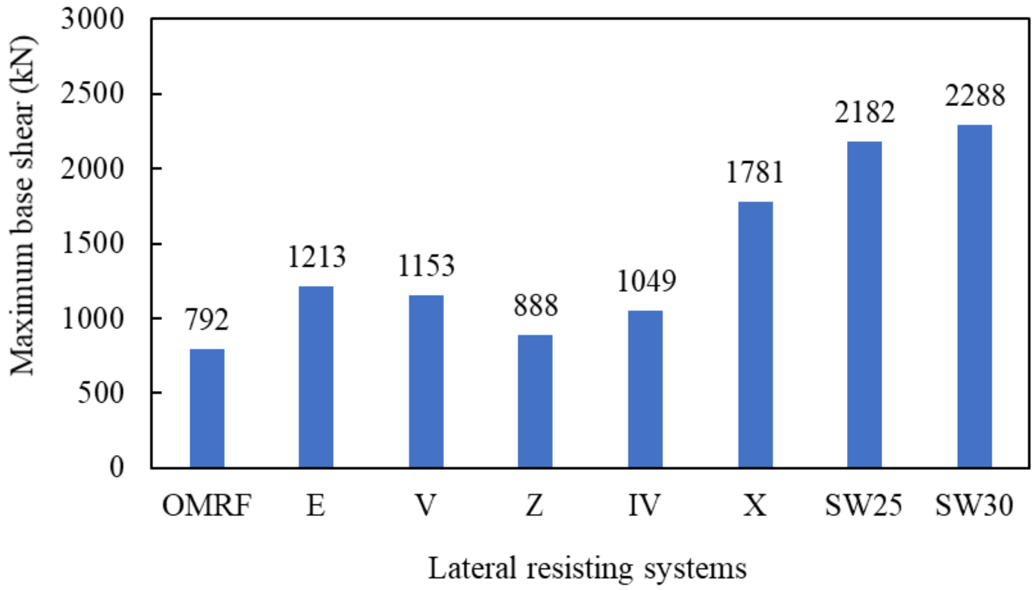


Figure 4.18: Average ultimate base shear of lateral load resisting systems

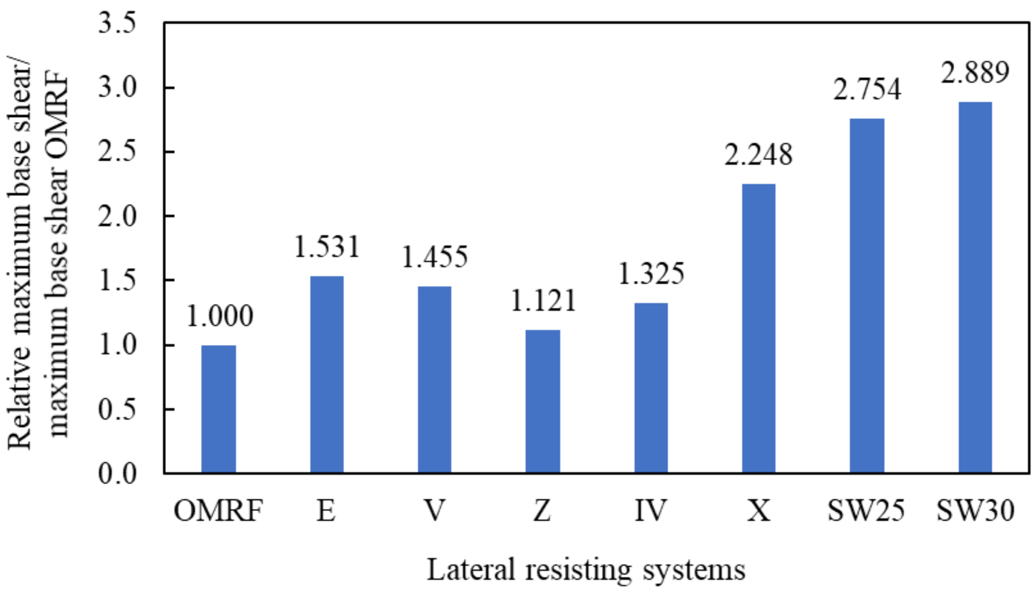


Figure 4.19: Comparison of ultimate base shear of lateral load resisting systems with respect to OMRF

4.4 Factors Affecting Pushover Curves

The effect of different parameters on the pushover curve of steel frames are studied in this section.

4.4.1 The effect of different types of bracings and shear walls on the pushover curve of steel frames

Pushover curves are drawn in Figure 4.20 through 4.22 for different lateral load resisting systems with some fixed parameters. The elastic stiffness (the initial slope of the pushover curves) of the steel frames are different, after inserting lateral load resisting systems into the steel frames the initial slope becomes more steeper, as it is seen in the figure the elastic stiffness of SW30 is higher than other LLRS, and OMRF has the minimum capacity compared to other LLRS. But as it is observed from the pushover curves, E type bracing (eccentrically bracing) provides better ductility than others. In some pushover curves of bracings, a sudden drop down has occurred due to failure of bracings in buckling.

Assuming fixed parameters S = H, N = 1, L = 6 m and H = 3.2 m

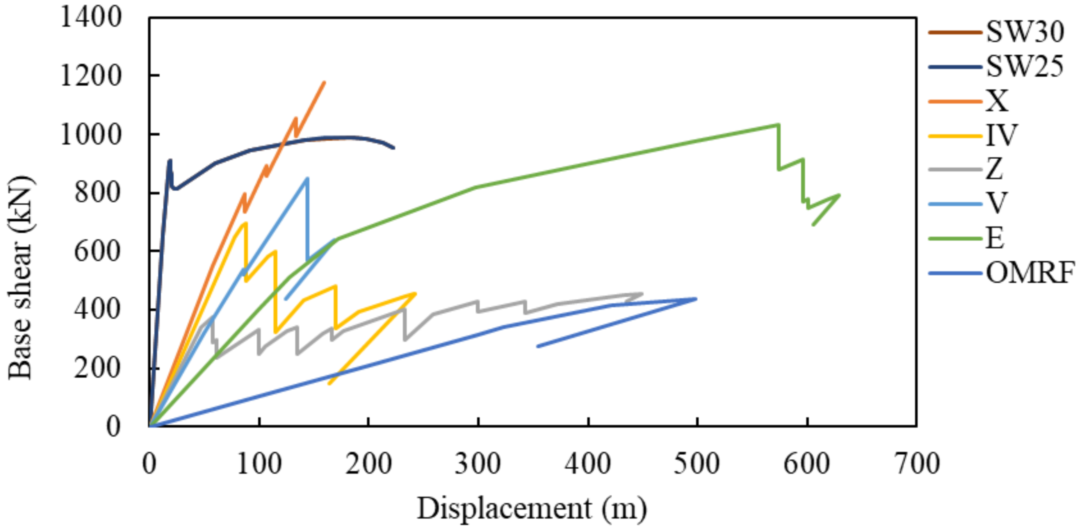


Figure 4.20: Pushover curve for different types of bracings and shear walls

Assuming fixed parameters S = H, N = 3, L = 6 m and H = 3.2 m

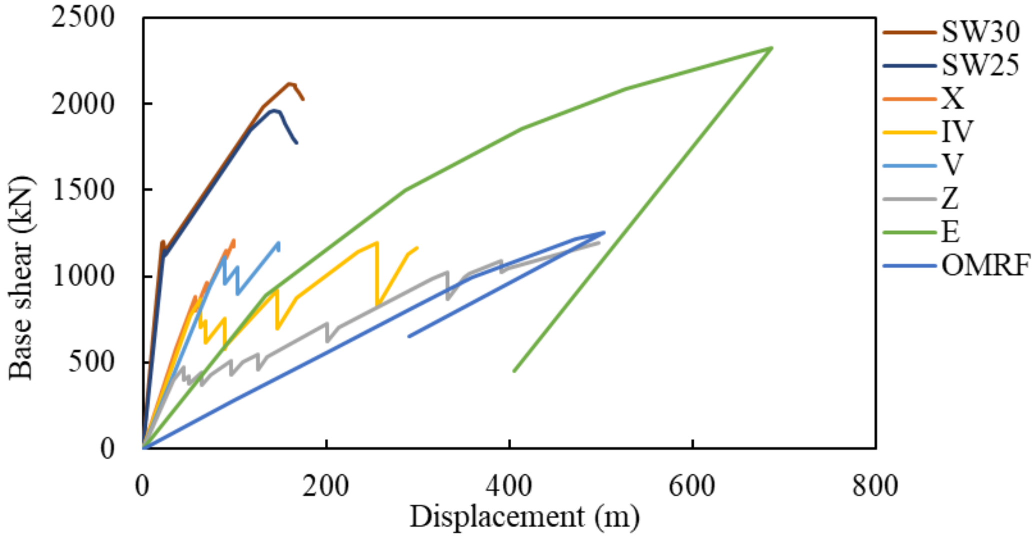


Figure 4.21: Pushover curve for different types of bracings and shear walls

Assuming fixed parameters S = H, N = 5, L = 6 m and H = 3.2 m

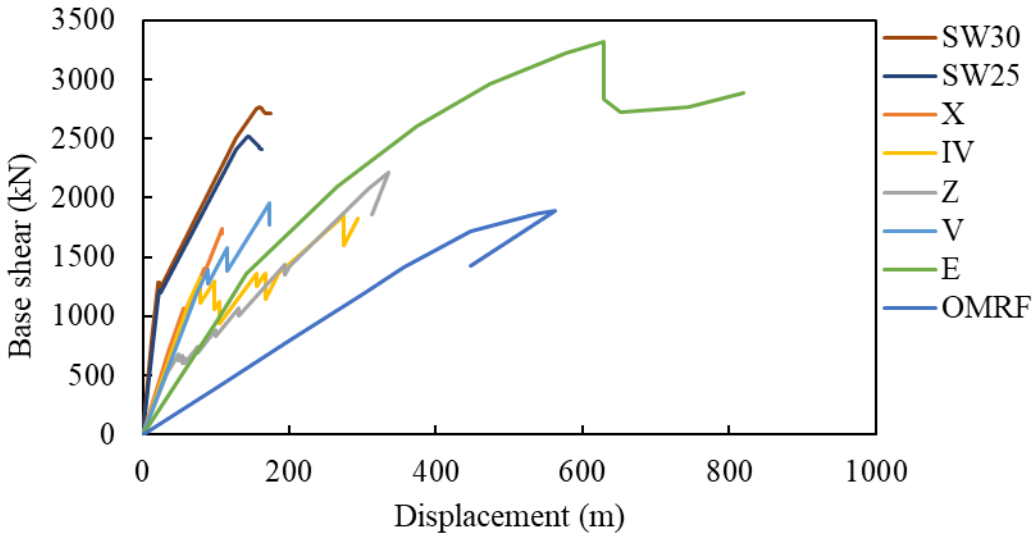


Figure 4.22: Pushover curve for different types of bracings and shear walls

4.4.2 The effect of number of spans on the pushover curve of steel frames for the assumed lateral load resisting systems

Figure 4.23 shows the effect of number of spans on the pushover curve of steel frames when other parameters are fixed. It is found from the figure as the number of spans increases the capacity of the steel frames becomes higher and increases, first OMRF having 1-span has the minimum capacity but when the number of spans is increased for 5, its capacity increases and the initial slope of the curve becomes higher. As it is observed from the figure, after inserting SW 30 and X bracings to the systems, the capacity of the frames has increased significantly. Simultaneously with increasing the number of spans for SW30 and X- type bracing, the capacity of the steel frames has increased.

The fixed parameters are H= 3.2 m, S= H, L= 6 m and LLRS= OMRF, X- bracing and SW30

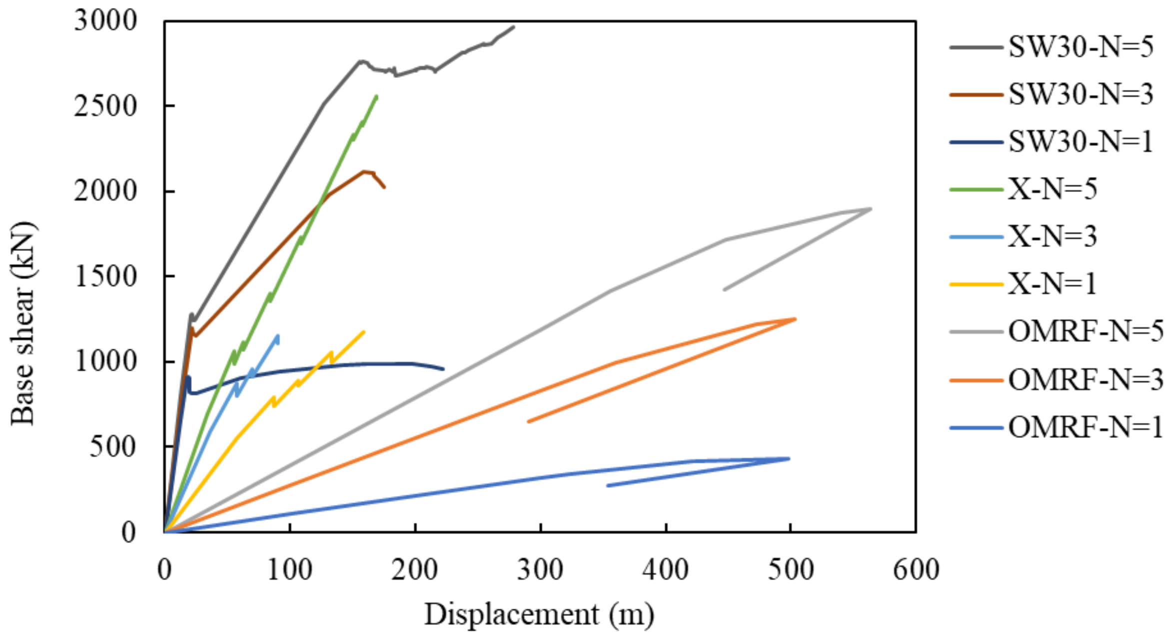


Figure 4.23: Effect of number of spans on pushover curve of the selected LLRS

4.4.3 The effect of span length changes on the pushover curve of steel frames for the assumed lateral load resisting systems

Figure 4.24 demonstrates the effect of span length changes on the pushover curve of the selected LLRS, as it is seen from the figure as the span length increased from 5 m to 6 m, the capacity and elastic stiffness of the steel frames has increased as well. when the span length increases the cross section of the steel frames increases which results in the higher capacity of pushover curve for all LLRS.

The fixed parameters are H= 3.2 m, S= H, N= 5 and LLRS= OMRF, X- bracing and SW30

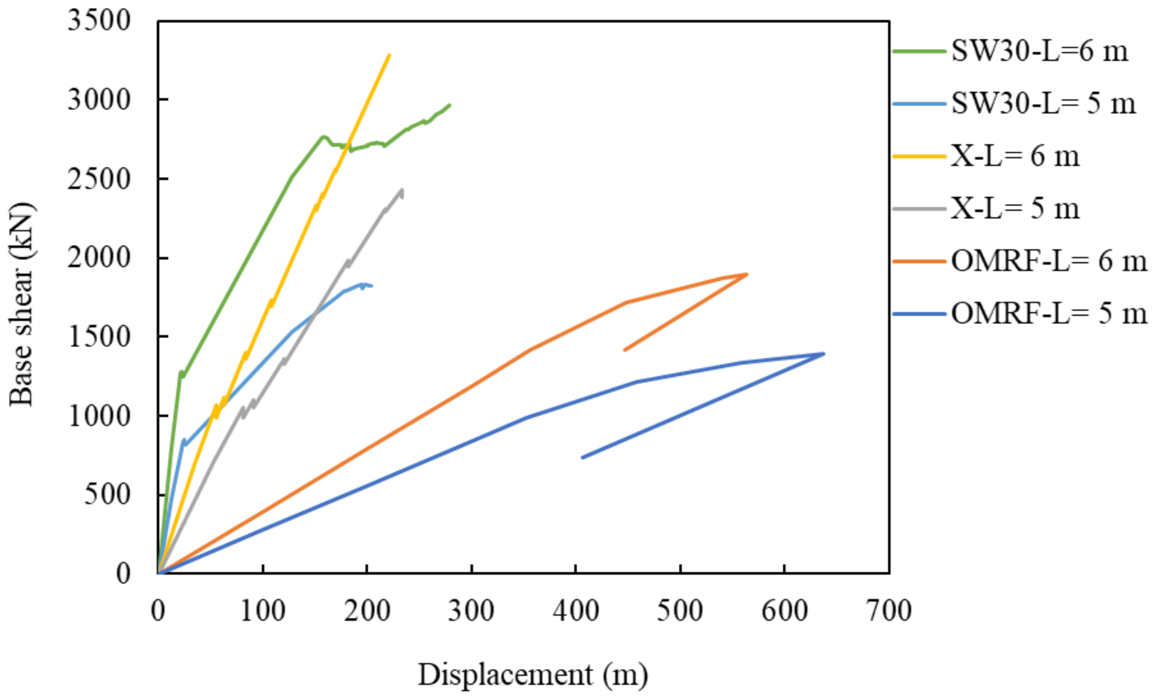


Figure 4.24: Effect of span length on the pushover curves of the selected LLRS

4.4.4 The effect of number of storey changes on the pushover curve of steel frames for the assumed lateral load resisting systems

Figure 4.25 shows the effect of number of stories on the pushover curve of steel frames of selected LLRS. It is determined that as the number of stories changes from low to high the capacity of the steel frames decreases. But buildings with larger number of stories exhibited better ductility than low rise buildings. OMRF with larger number (S= H) of stories has the minimum capacity compared to others but SW30 with S= L has the maximum initial slope and a sudden rupture occurs since its stiffness is too high and fails due to shear.

The fixed parameters are H= 3.2 m, N= 5, L= 6 m and LLRS= OMRF, X- bracing and SW30

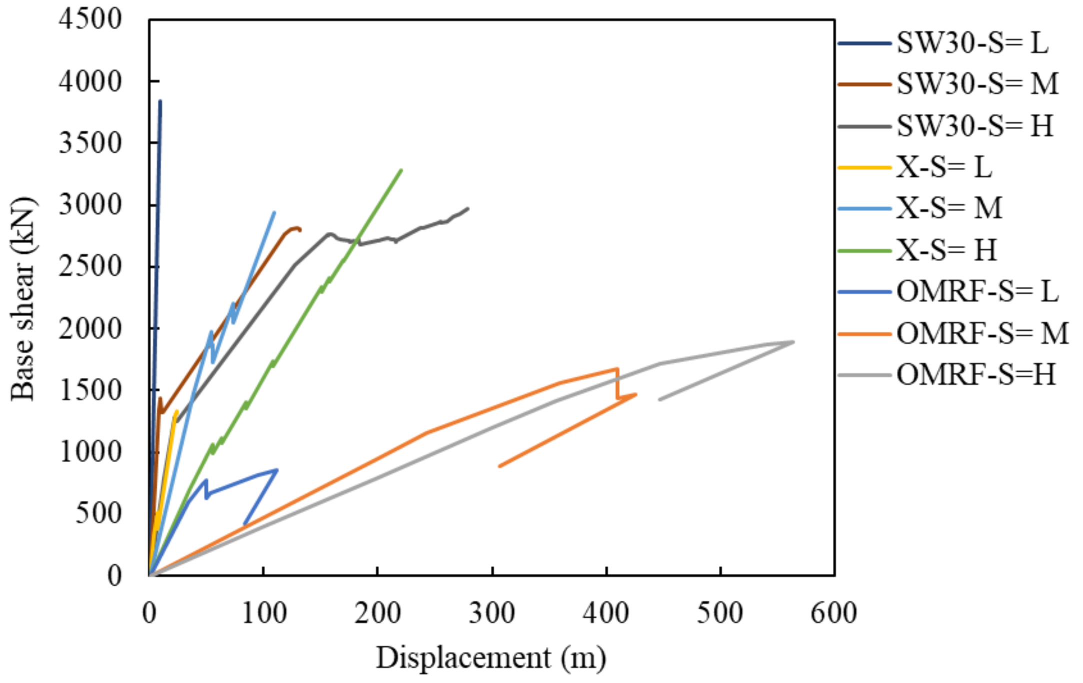


Figure 4.25: Effect of storey number changes on the pushover curves of the selected LLRS

4.4.5 The effect of storey height changes on the pushover curve of steel frames for the assumed lateral load resisting systems

Changing height of stories in a building results a change in the overall height of the building, in Figure 4.26 the story height of the steel frames is changed from 3.2 m to 3.4 m, and it is found that when the height of the stories increases the capacity of the steel frames for all LLRS is decreased.

The fixed parameters are S= H, N= 5, L= 6 m and LLRS= OMRF, X- bracing and SW30

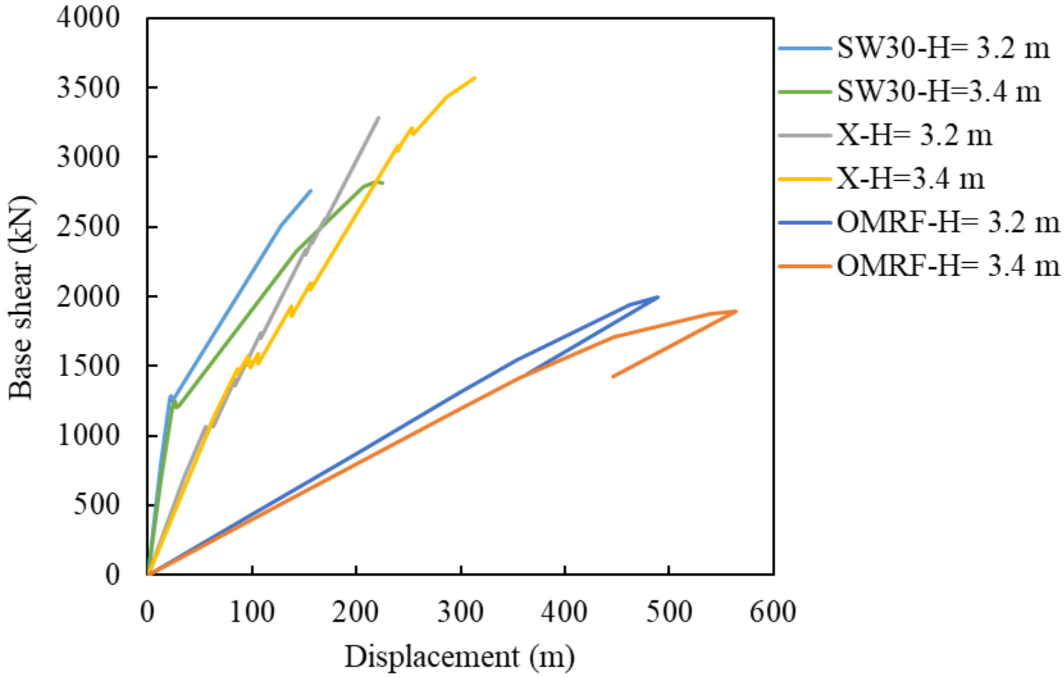


Figure 4.26: Effect of story height changes on the pushover curves of the selected LLRS

CHAPTER 5
CONCLUSIONS AND RECOMMENDATION

5.1 Conclusions

In this research, pushover analysis is used to evaluate the elastic stiffness factor, natural time period, maximum base shear and pushover curves of 2D steel frames for different lateral load resisting systems. First, 720 2D steel models have been analyzed and designed using equivalent lateral force procedure, after that by using pushover analysis method, the results of all models have been analyzed, compared and evaluated, then the effect of number of parameters on the elastic stiffness factor, time period, maximum base shear and pushover curves are considered including, different lateral load resisting systems, span length, number of stories, number of spans and story height. Based on the pushover analysis method in this study, by applying the effect of parameters considered in this study, the elastic stiffness factor, time period, maximum base shear pushover curves of the structure with an acceptable result can be evaluated. The summarized conclusion of this study are as follows:

The summarized conclusions on the elastic stiffness factor are drawn as follows

1- Provision of lateral load resisting systems into structures results in increasing the elastic stiffness factor of 2D steel frames, shear walls having compressive strength of concrete 30 and 25 MPa are stiffer than concentrically and eccentrically bracings respectively. And among the bracings, X type bracing has the maximum elastic stiffness factor. OMRF provides the minimum elastic stiffness among all lateral load resisting systems.
2- By increasing the span length from 4.5m to 6.5 with 0.5m intervals, the elastic stiffness of the steel frames has increased for all types of lateral load resisting systems.
3- Changing the number of stories from 1 to 5 and then to 8 causes a decrease in the elastic stiffness factor of the 2D steel frames for all types of LLRS.
4- The elastic stiffness of the steel frames for bracings and OMRF increases when the number of spans is increased, but the elastic stiffness factor of SW30 and SW25 reduces

for low rise buildings and increases for high rise building when the number of spans increases from 1 to 3 and then to 5.

5- As the story height increases, the elastic stiffness factor is reduced. And the reduction in the elastic stiffness factor is much more in the case of buildings with a large number of stories.

The summarized conclusions on natural time period are drawn as follows

1- Provision of lateral load resisting systems into structures results in decreasing the natural time period of 2D steel frames, shear walls having compressive strength of concrete 30 and 25 MPa have the minimum value of natural time period because they are much stiffer than other types of lateral load resisting systems. And among the bracings, X type bracing has the minimum natural time period. OMRF does not provide enough stiffness in the structure compared to other LLRS so the natural time period of OMRF are higher among all types of LLRS.
2- By increasing the length of the span, the natural time period decreases for all types of bracings and shear walls because the percentage increase in mass as a result of span length change is smaller than the percentage increase in stiffness.
3- Increasing the number of stories from 1 to 5 and then to 8 causes an increase in the natural time period of the 2D steel frames for all types of LLRS due to increase in the mass and decrease in the overall elastic stiffness of the frames, as a result it can be said high rise buildings have larger natural time period than low rise buildings.
4- When the number of spans increases from 1 to 3 and 5, the natural time period increases for all types of bracings and shear walls because the percentage increase in mass as a result of increasing in number of spans is greater than the percentage increase in stiffness.
5- As the story height increases, the natural time period is increased. And the increasing in the natural time period is much more in the case of buildings with a large number of stories.

The summarized conclusions on maximum base shear are drawn as follows

1- Provision of lateral load resisting systems into structures results in increasing the maximum base shear of 2D steel frames, shear walls having compressive strength of concrete 30 and 25 MPa are stiffer than concentrically and eccentrically bracings when the number of stories is low. but X type bracing has the maximum base shear (maximum capacity) in the case of 2D steel frames with large number of stories. OMRF provides the minimum base shear among all lateral load resisting systems.
2- As the span length increases, the steel frames can withstand more base shear which means as the span length increases, the maximum base shear (maximum capacity) increases as well for all types of LLRS
3- Changing the number of stories from 1 to 5 and then to 8 causes an increase in the maximum base shear of steel frames of the 2D steel frames for SW30 and SW25, but causes an increase in the maximum base shear for bracings
4- The maximum base shear increases when the number of spans is increased. The percentage of increase in high rise buildings is higher compared to low rise buildings
5- As the story height increases, the maximum base shear is reduced almost for all types of lateral load resisting systems.

The summarized conclusions on the pushover curve of steel frames are drawn as follows

1- Provision of LLRS into structures results in increasing the capacity of 2D steel frames, shear walls having compressive strength of concrete 30 has higher performance than concentrically and eccentrically bracings. And among the bracings, X type bracing has the maximum capacity, but OMRF provides the minimum capacity among all lateral load resisting systems. Among all LLRS, K- shaped eccentrically bracing showed better ductility and displaced more before rupture takes place.
2- By increasing the span length of the steel frames, the capacity of the steel frames has increased for all types of lateral load resisting systems.
3- Changing the number of stories from 1 to 5 and then to 8 causes a decrease in the performance and capacity of the 2D steel frames for all types of LLRS. In the other

hand when the number of stories increases, the ductility of the steel frames has increased.

4- The capacity of the steel frames for bracings and OMRF against lateral loading increases when the number of spans is increased, but the capacity of SW30 and SW25 reduces for low rise buildings and increases for high rise building when the number of spans increases from 1 to 3 and then to 5.
5- As the story height increases, the capacity of the steel is reduced. And the reduction in the capacity factor is much more in the case of buildings with a large number of stories.

5.2 Recommendation

In this study, only 2D steel frames are considered which means the lateral load are applied only in one direction so the further analysis of 3D steel frames is necessary to account different directions of earthquake at the same time. Additionally, some structures may not be regular in plan which causes the occurrence of torsion since the center of rigidity and center mass are located in different places, in this study the effect of torsion is not considered because all the sample are 2D steel frames. The torsional effects could change the overall design of the frames so the effect of torsion should be considered in the future works.

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APPENDICES

APPENDIX 1

A.1.1 Factors Affecting Elastic Stiffness Factor

A.1.1.1 The effect of span length on the elastic stiffness of the 2D steel frames for varied types of concentrically and eccentrically bracing and shear walls

The parameters fixed for this figure are N = 3, S = L, H = 3.2 m

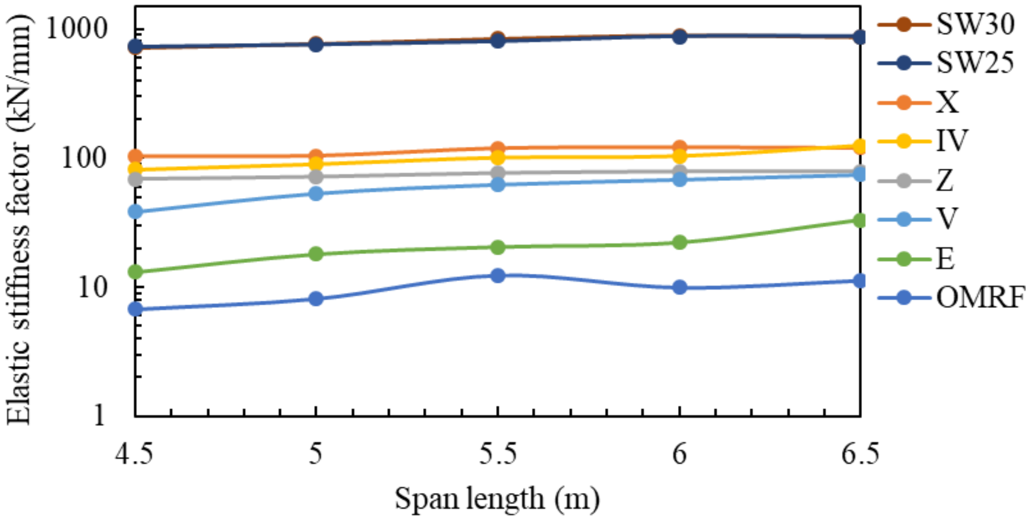

Figure A.1.1: The elastic stiffness factor of the frames versus span length for different Types of bracings and shear walls

The parameters fixed for this figure are N = 5, S = L, H = 3.2 m

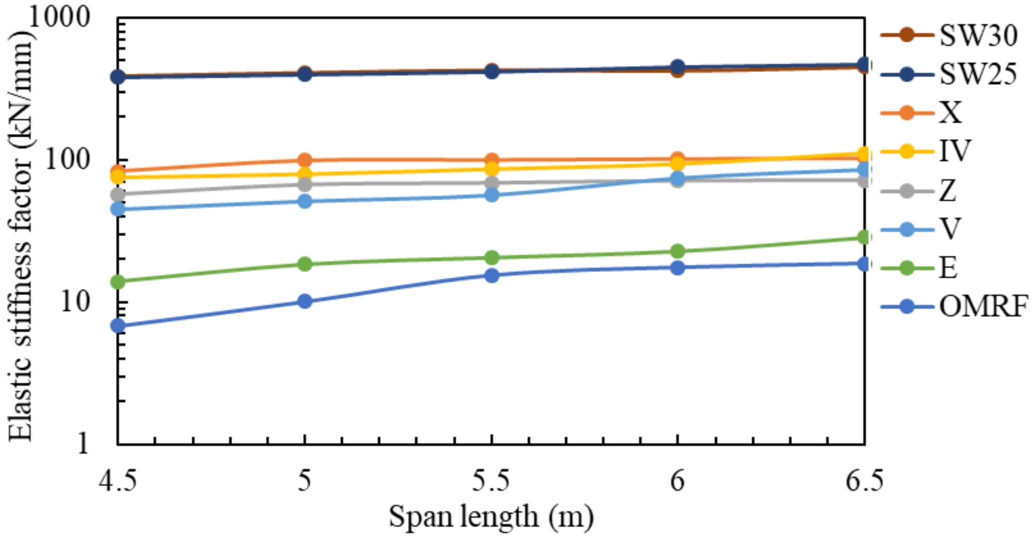

Figure A.1.2: The elastic stiffness factor of the frames versus span length for different types of bracings and shear walls

The parameters fixed for this figure are N = 1, S = M, H = 3.2 m

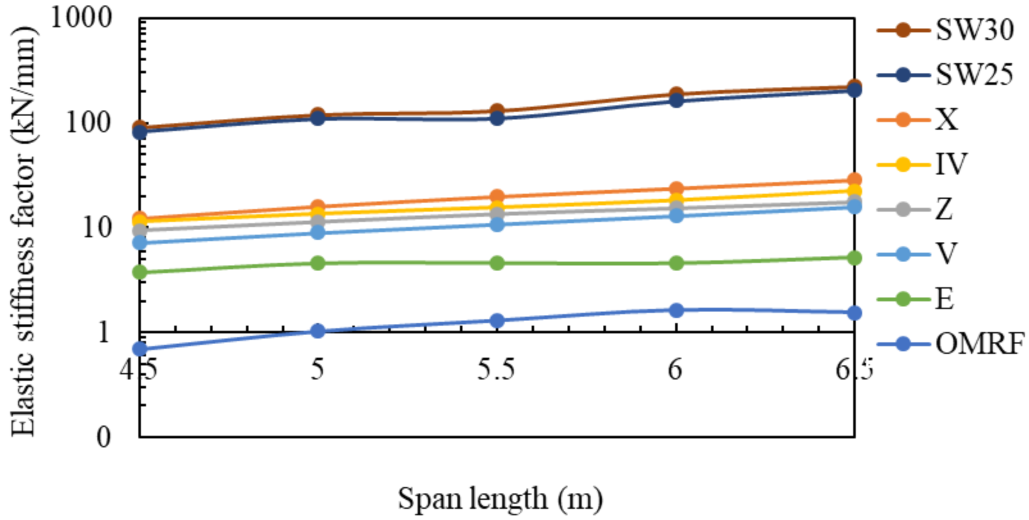

Figure A.1.3: The elastic stiffness factor of the frames versus span length for different types of bracings and shear walls

The parameters fixed for this figure are N = 3, S = M, H = 3.2 m

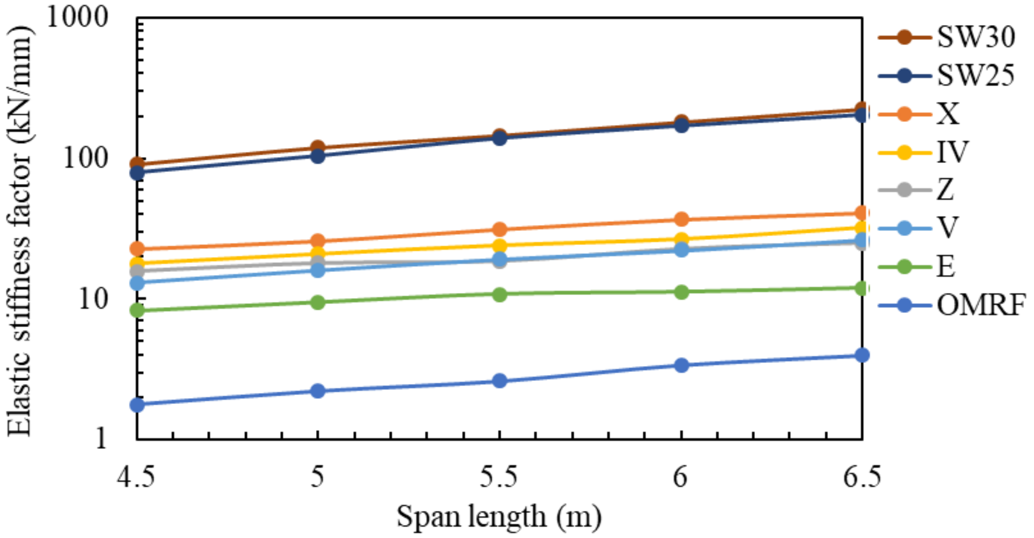

Figure A.1.4: The elastic stiffness factor of the frames versus span length for different types of bracings and shear walls

The parameters fixed for this figure are N = 5, S = M, H = 3.2 m

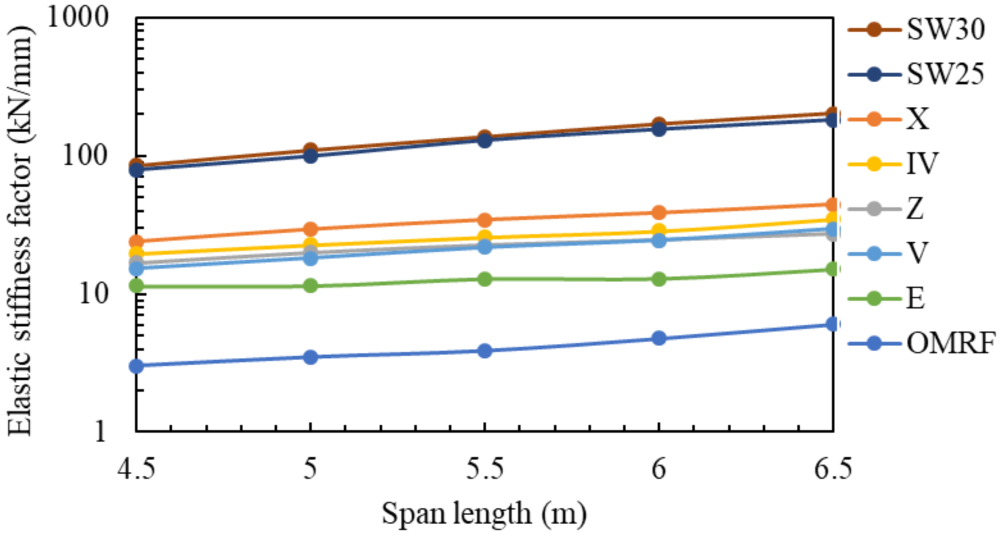

Figure A.1.5: The elastic stiffness factor of the frames versus span length for different types of bracings and shear walls

The parameters fixed for this figure are N = 1, S = H, H = 3.2 m

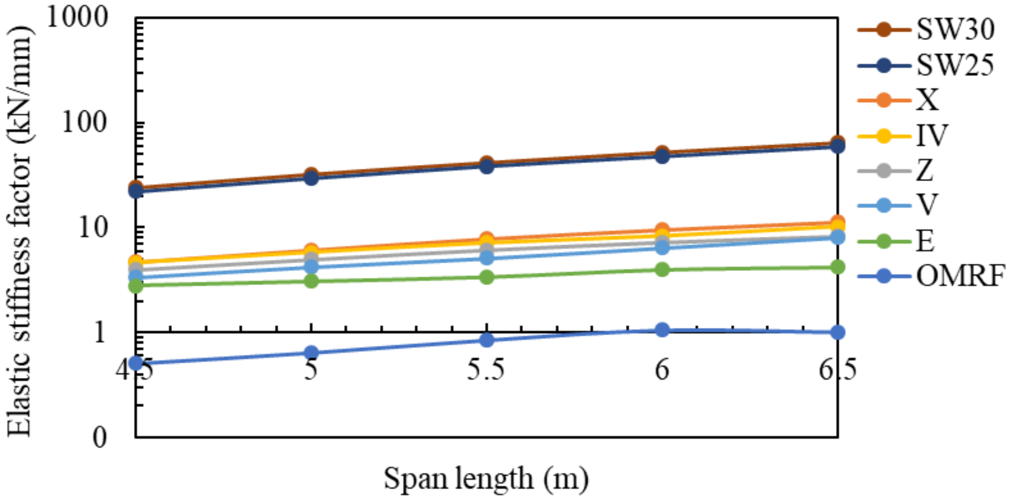

Figure A.1.6: The elastic stiffness factor of the frames versus span length for different types of bracings and shear walls

The parameters fixed for this figure are N = 3, S = H, H = 3.2 m

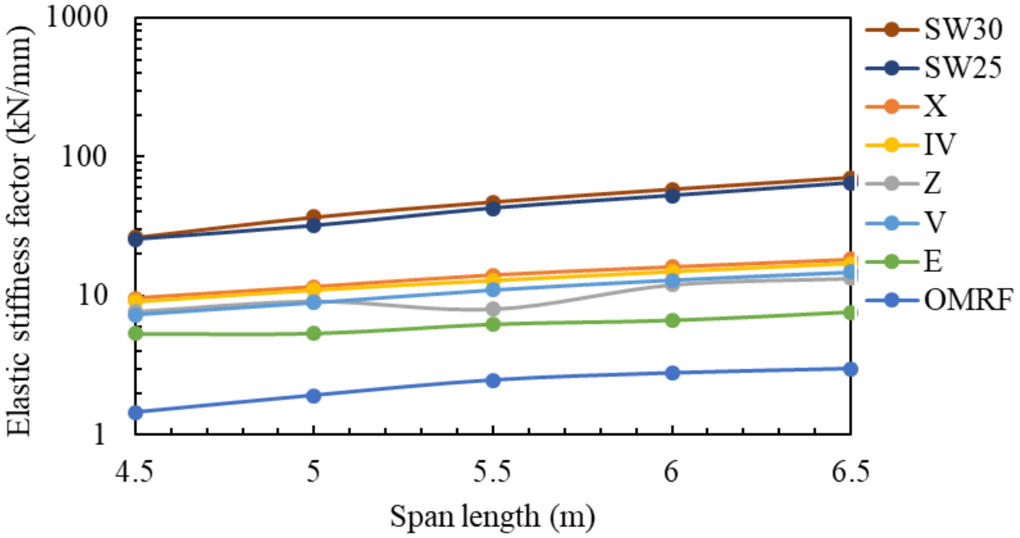

Figure A.1.7: The elastic stiffness factor of the frames versus span length for different types of bracings and shear walls

The parameters fixed for this figure are N = 5, S = H, H = 3.2 m

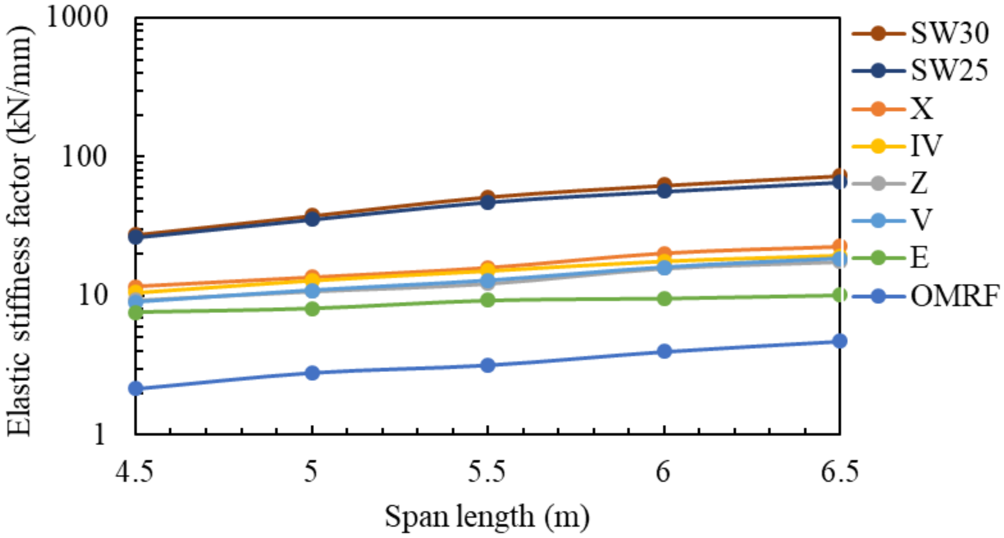

Figure A.1.8: The elastic stiffness factor of the frames versus span length for different types of bracings and shear walls

A.1.1.2 The effect of number of stories on the elastic stiffness factor of the steel frames for different types of bracings and shear walls

Assuming fixed parameters N = 1, L = 5 m and H = 3.2 m

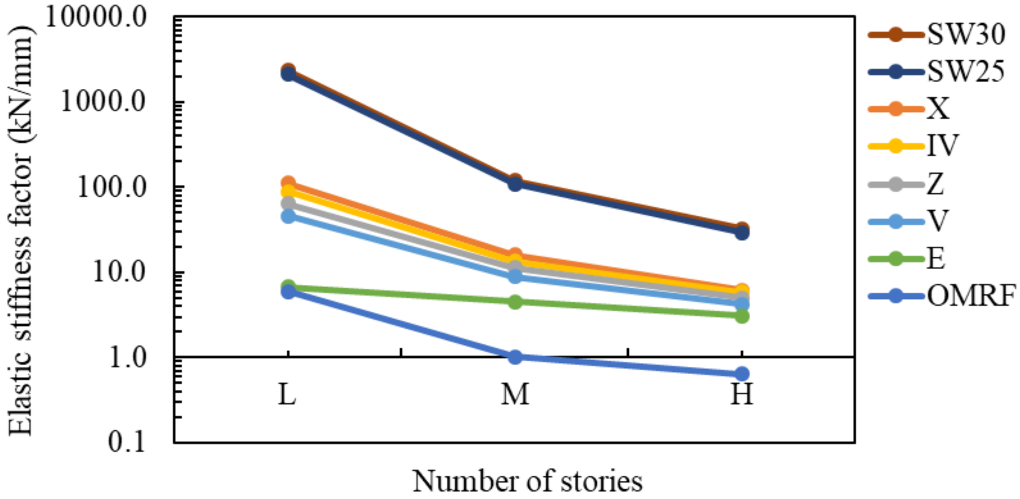

Figure A.1.9: The elastic stiffness factor of the frames versus the number of stories for different types of bracings and shear walls

Assuming fixed parameters N = 1, L = 5.5 m and H = 3.2 m

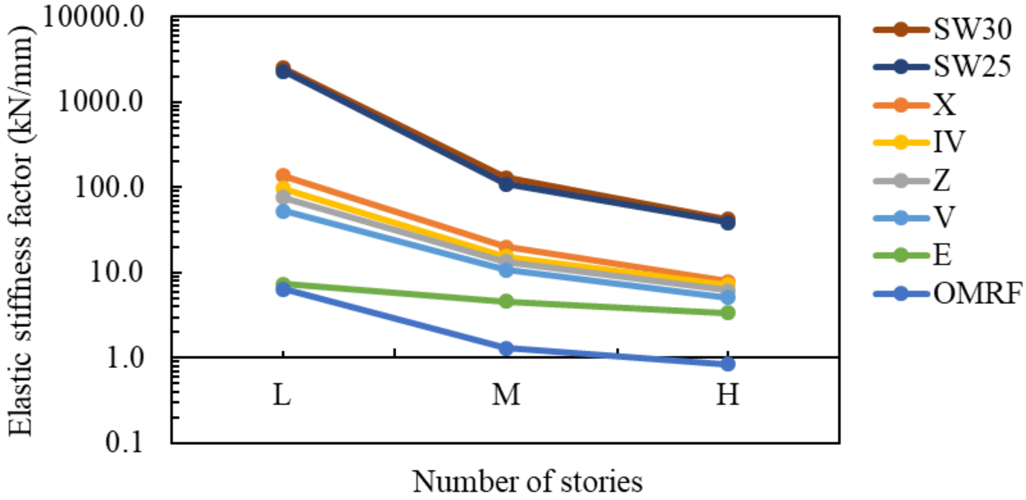

Figure A.1.10: The elastic stiffness factor of the frames versus the number of stories for different types of bracings and shear walls

Assuming fixed parameters N = 1, L = 6 m and H = 3.2 m

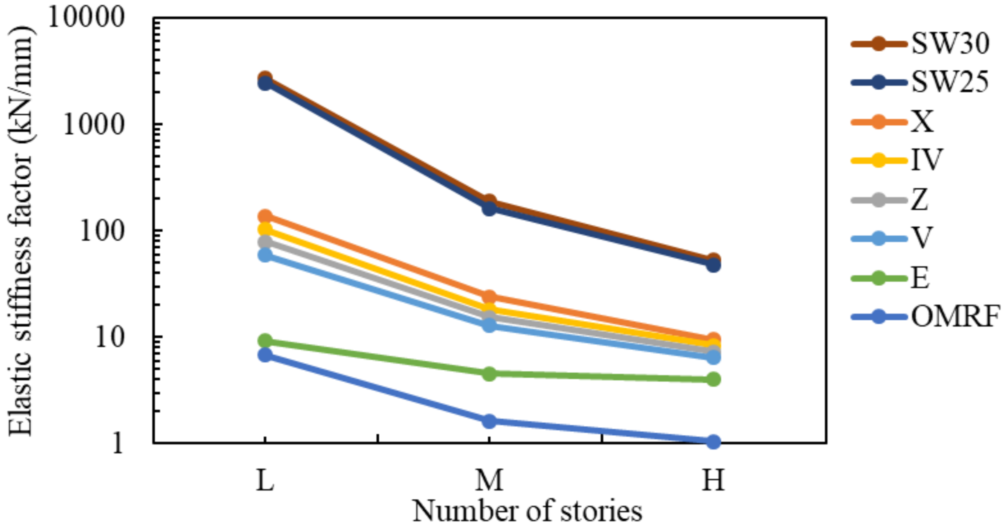

Figure A.1.11: The elastic stiffness factor of the frames versus the number of stories for different types of bracings and shear walls

Assuming fixed parameters N = 1, L = 6.5 m and H = 3.2 m

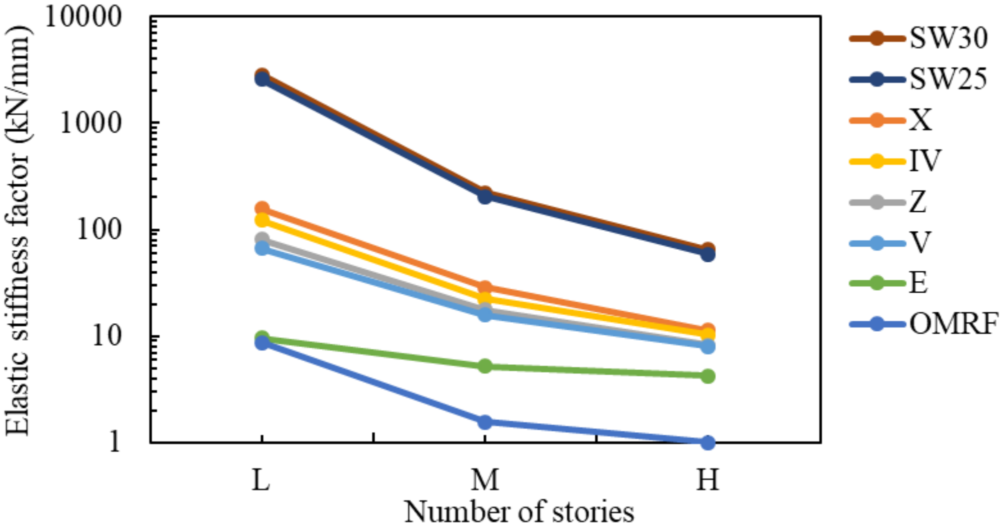

Figure A.1.12: The elastic stiffness factor of the frames versus the number of stories for different types of bracings and shear walls

Assuming fixed parameters N = 3, L = 4.5 m and H = 3.2 m

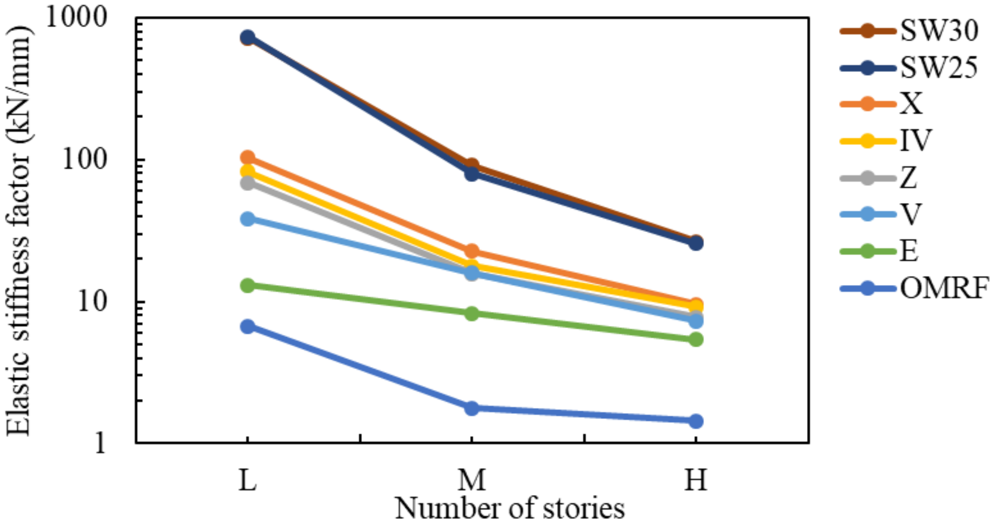

Figure A.1.13: The elastic stiffness factor of the frames versus the number of stories for different types of bracings and shear walls

Assuming fixed parameters N = 3, L = 5 m and H = 3.2 m

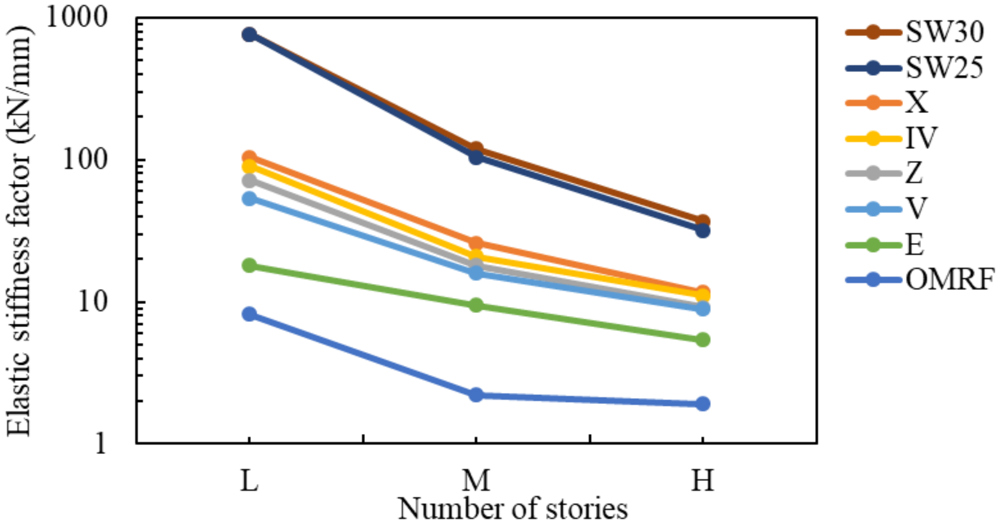

Figure A.1.14: The elastic stiffness factor of the frames versus the number of stories for different types of bracings and shear walls

Assuming fixed parameters N = 3, L = 5.5 m and H = 3.2 m

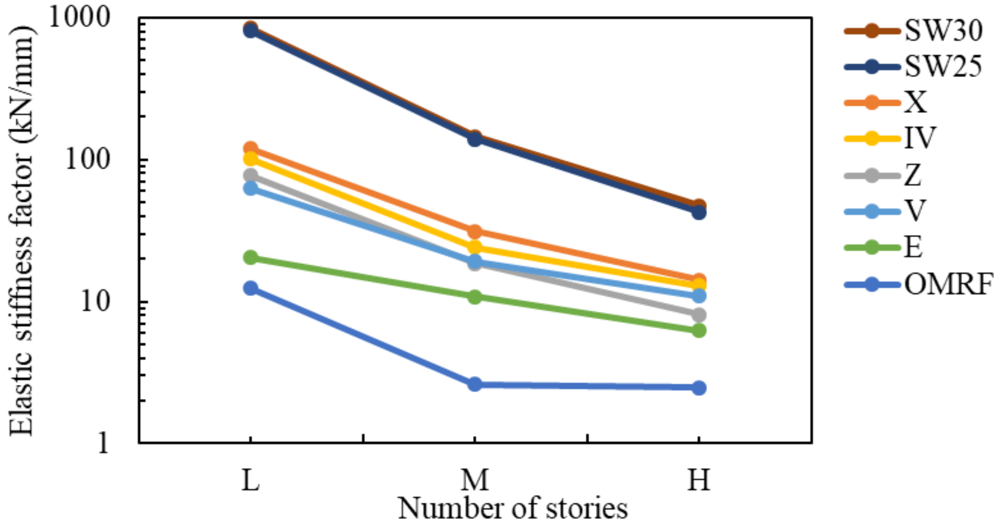

Figure A.1.15: The elastic stiffness factor of the frames versus the number of stories for different types of bracings and shear walls

Assuming fixed parameters N = 3, L = 6 m and H = 3.2 m

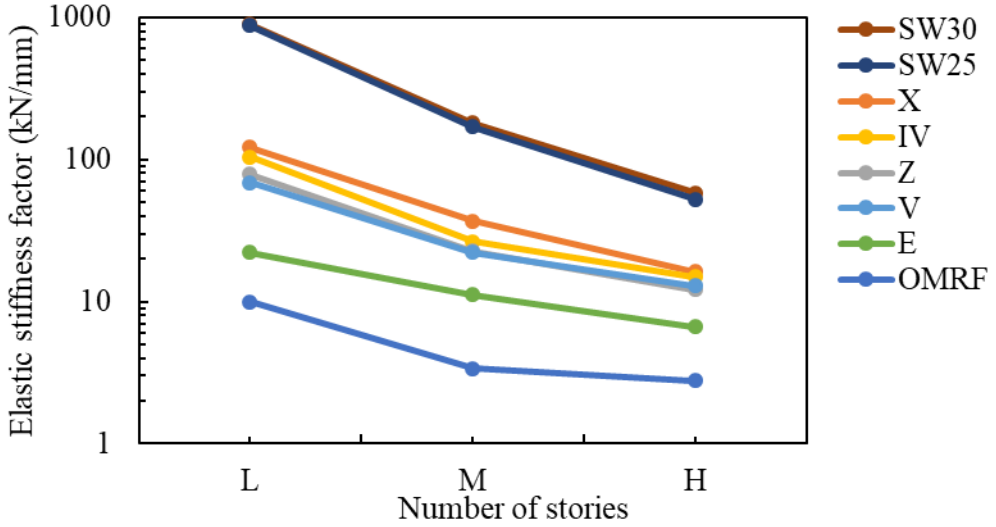

Figure A.1.16: The elastic stiffness factor of the frames versus the number of stories for different types of bracings and shear walls

Assuming fixed parameters N = 3, L = 6.5 m and H = 3.2 m

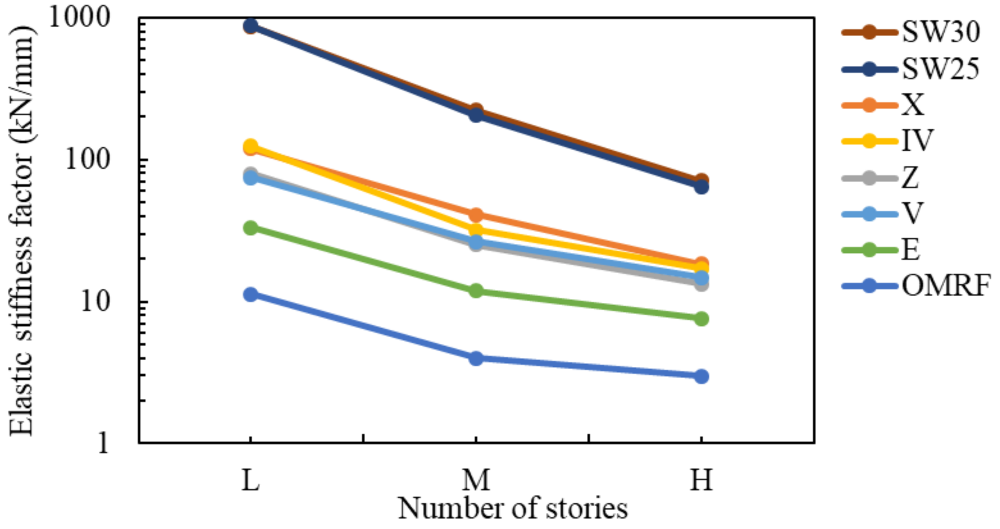

Figure A.1.17: The elastic stiffness factor of the frames versus the number of stories for different types of bracings and shear walls

Assuming fixed parameters N = 5, L = 4.5 m and H = 3.2 m

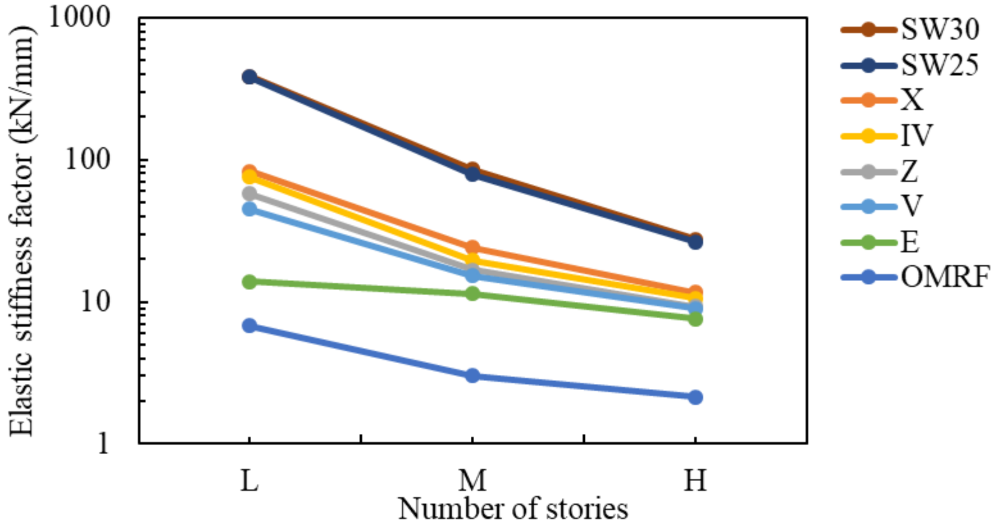

Figure A.1.18: The elastic stiffness factor of the frames versus the number of stories for different types of bracings and shear walls

Assuming fixed parameters N = 5, L = 5 m and H = 3.2 m

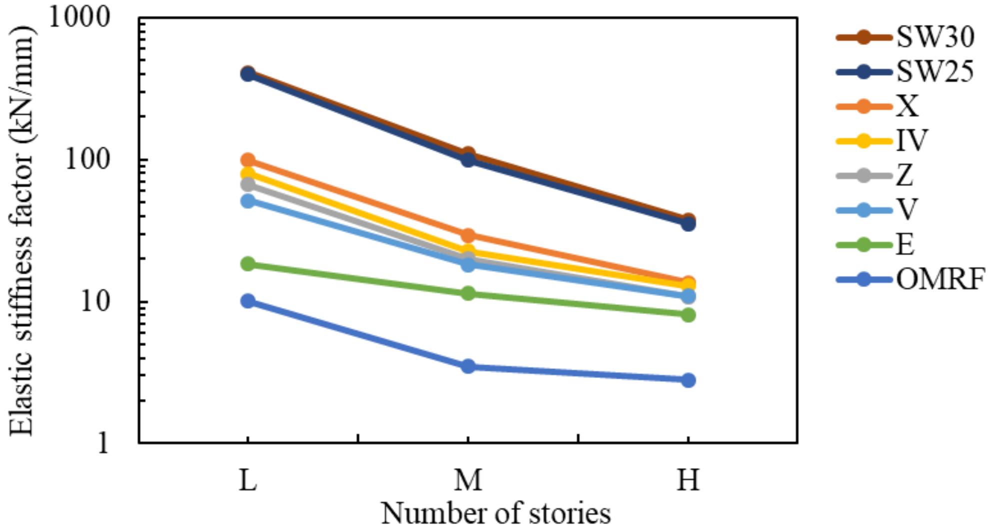

Figure A.1.19: The elastic stiffness factor of the frames versus the number of stories for different types of bracings and shear walls

Assuming fixed parameters N = 5, L = 5.5 m and H = 3.2 m

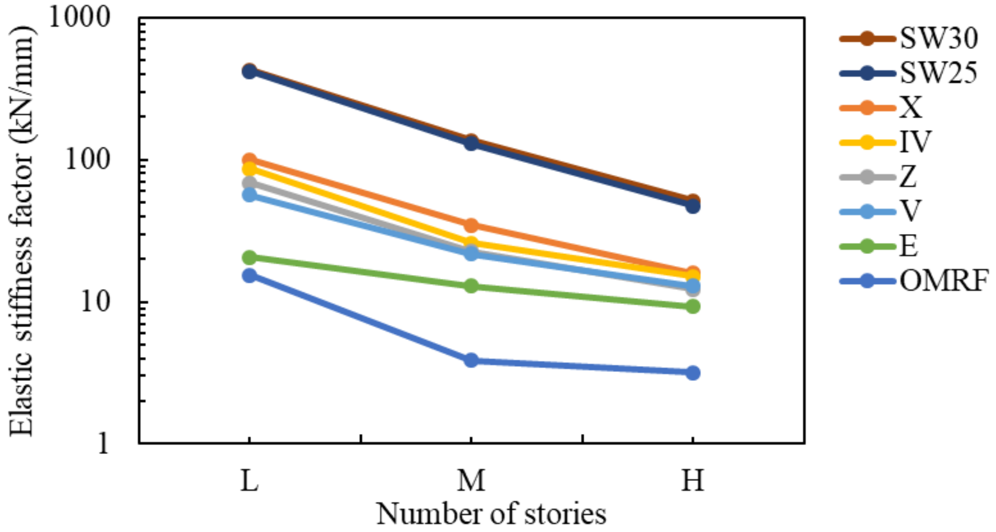

Figure A.1.20: The elastic stiffness factor of the frames versus the number of stories for different types of bracings and shear walls

Assuming fixed parameters N = 5, L = 6 m and H = 3.2 m

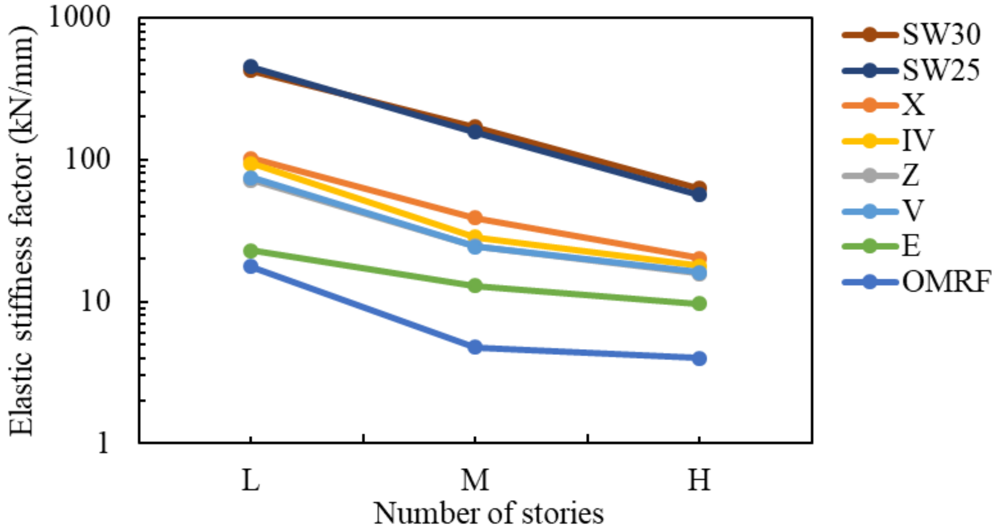

Figure A.1.21: The elastic stiffness factor of the frames versus the number of stories for different types of bracings and shear walls

Assuming fixed parameters N = 5, L = 6.5 m and H = 3.2 m

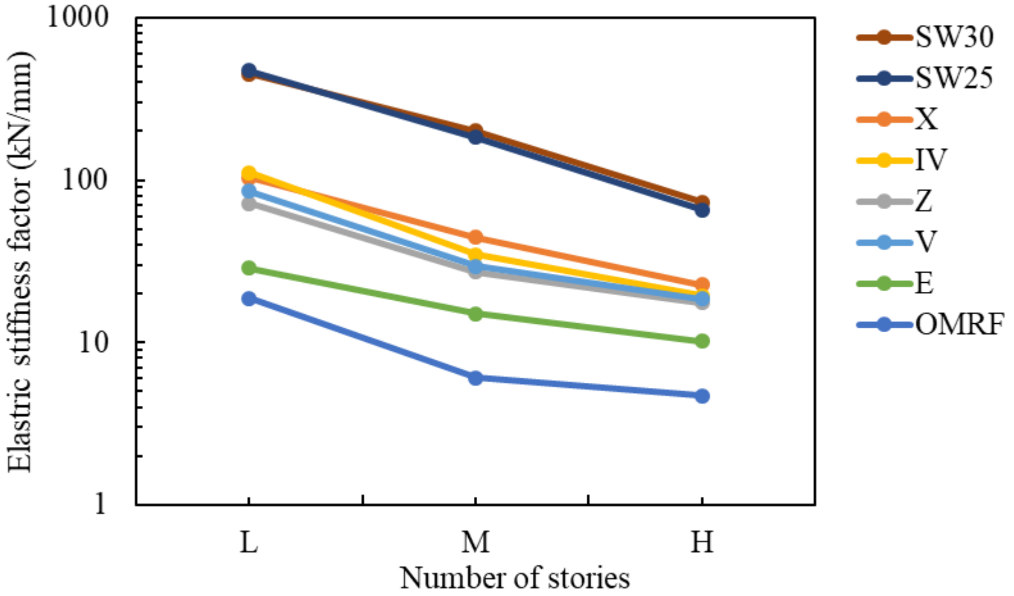

Figure A.1.22: The elastic stiffness factor of the frames versus the number of stories for different types of bracings and shear walls

A.1.1.3 The effect of number of spans on the elastic stiffness factor of the steel frames for different types of bracings and shear walls

Assuming fixed parameters S= L, L = 5 m and H = 3.2 m

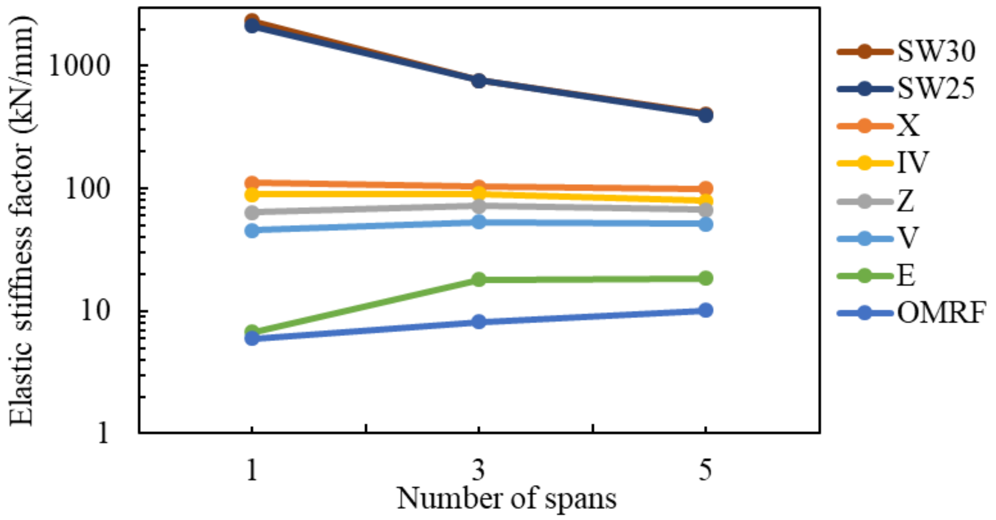

Figure A.1.23: The elastic stiffness factor of the frames versus the number of spans for different types of bracings and shear walls

Assuming fixed parameters S = L, L = 5.5 m and H = 3.2 m

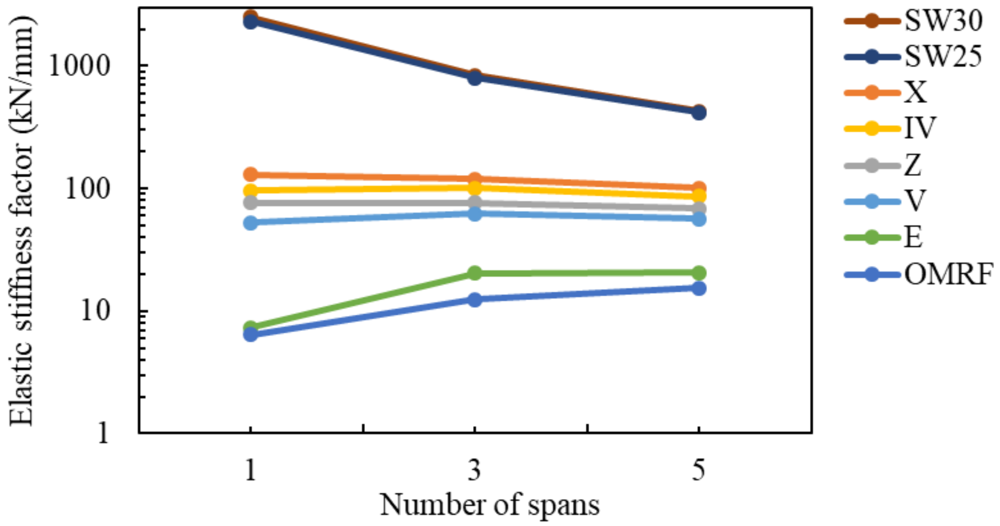

Figure A.1.24: The elastic stiffness factor of the frames versus the number of spans for different types of bracings and shear walls

Assuming fixed parameters S = L, L = 6 m and H = 3.2 m

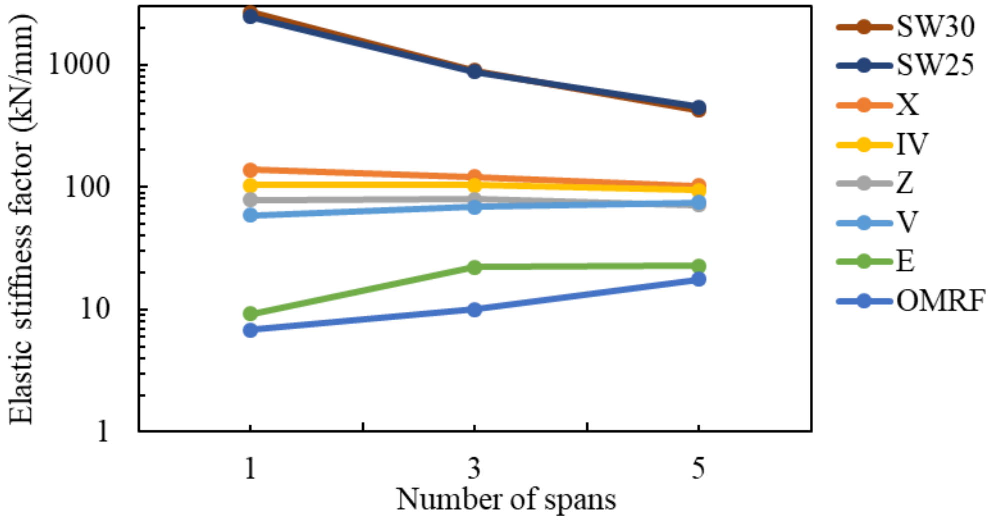

Figure A.1.25: The elastic stiffness factor of the frames versus the number of spans for different types of bracings and shear walls

Assuming fixed parameters S = L, L = 6.5 m and H = 3.2 m

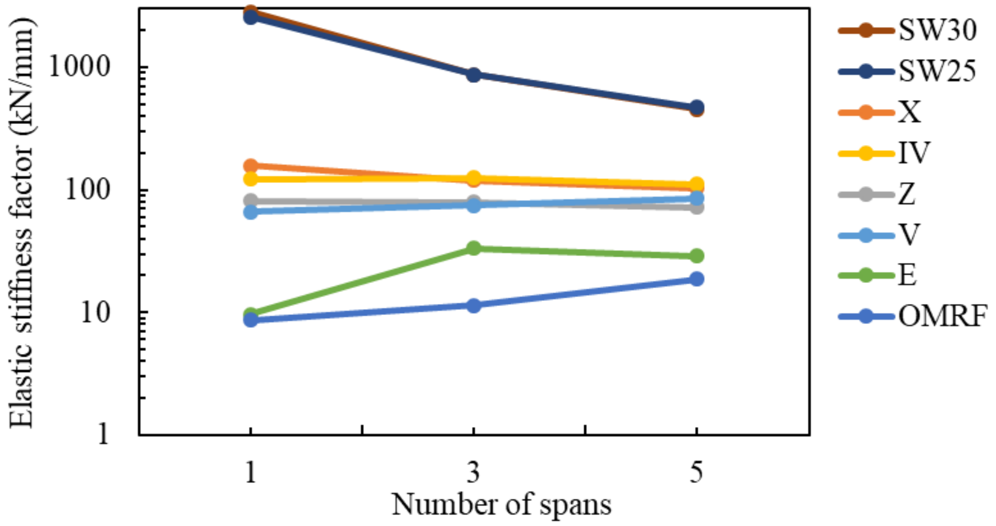

Figure A.1.26: The elastic stiffness factor of the frames versus the number of spans for different types of bracings and shear walls

Assuming fixed parameters S = M, L = 5 m and H = 3.2 m

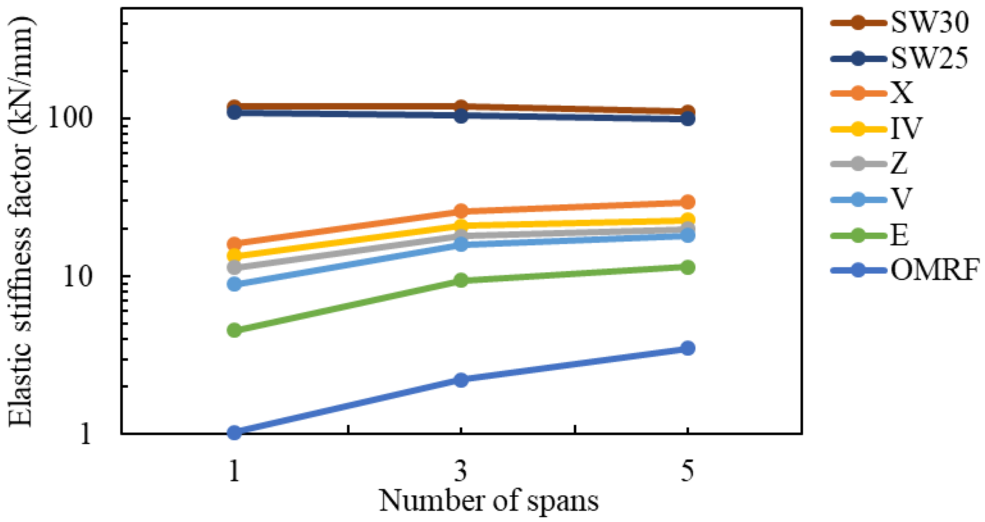

Figure A.1.27: The elastic stiffness factor of the frames versus the number of spans for different types of bracings and shear walls

Assuming fixed parameters S = M, L = 5.5 m and H = 3.2 m

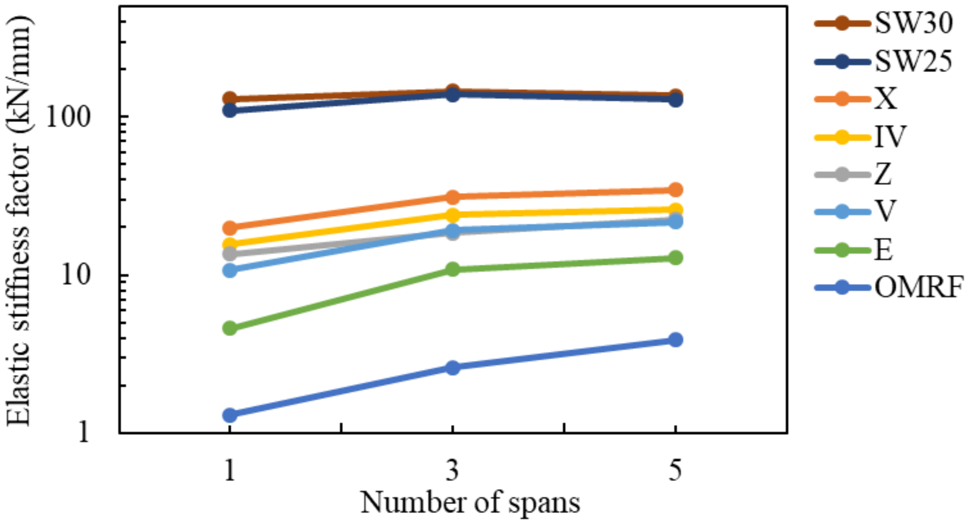

Figure A.1.28: The elastic stiffness factor of the frames versus the number of spans for different types of bracings and shear walls

Assuming fixed parameters S = M, L = 6 m and H = 3.2 m

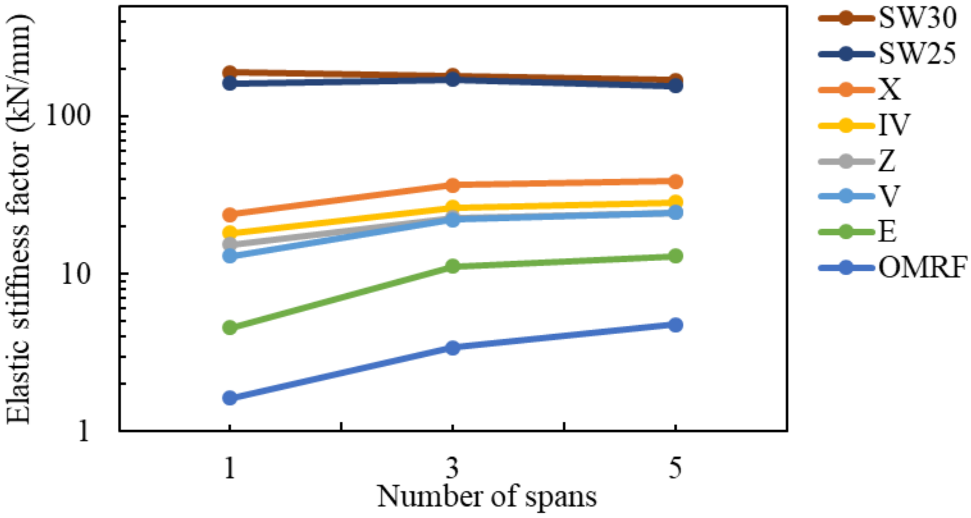

Figure A.1.29: The elastic stiffness factor of the frames versus the number of spans for different types of bracings and shear walls

Assuming fixed parameters S = M, L = 6.5 m and H = 3.2 m

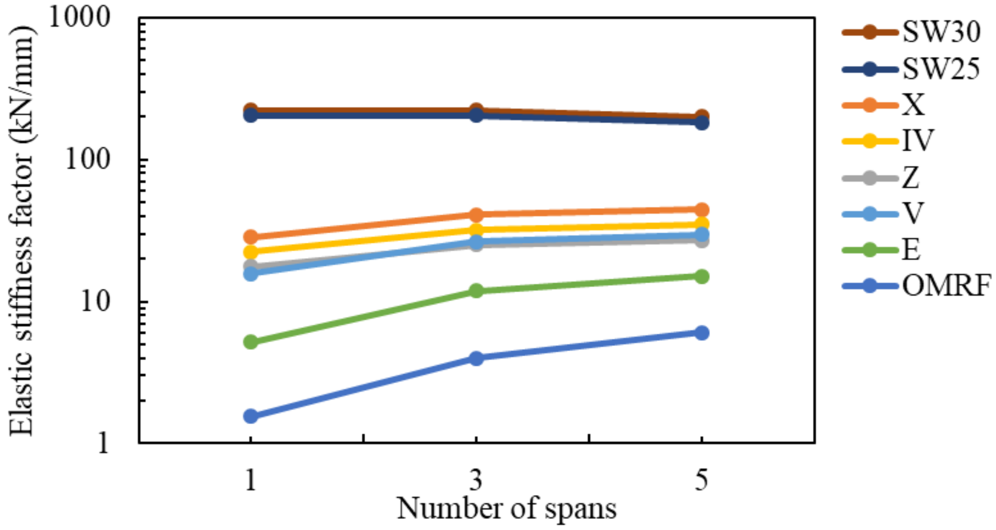

Figure A.1.30: The elastic stiffness factor of the frames versus the number of spans for different types of bracings and shear walls

Assuming fixed parameters S = H, L = 4.5 m and H = 3.2 m

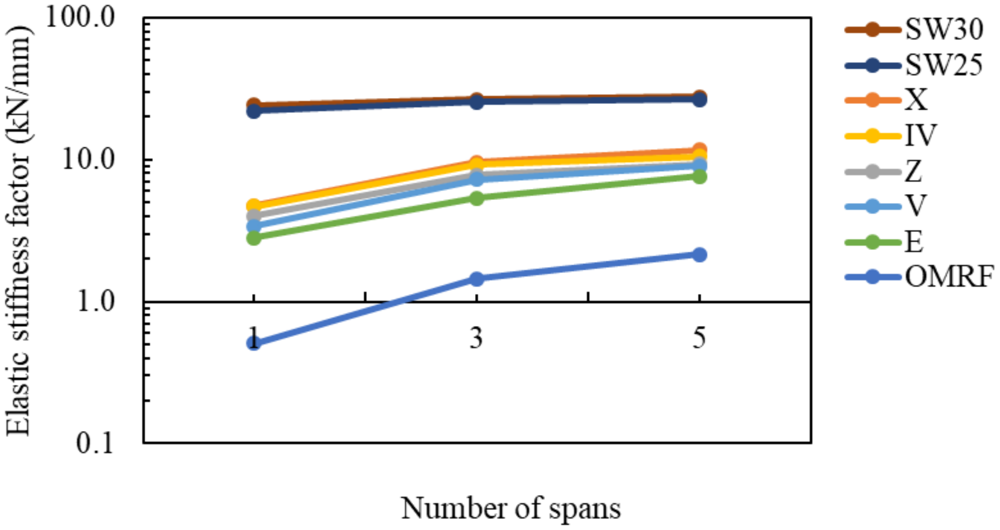

Figure A.1.31: The elastic stiffness factor of the frames versus the number of spans for different types of bracings and shear walls

Assuming fixed parameters S = H, L = 5 m and H = 3.2 m

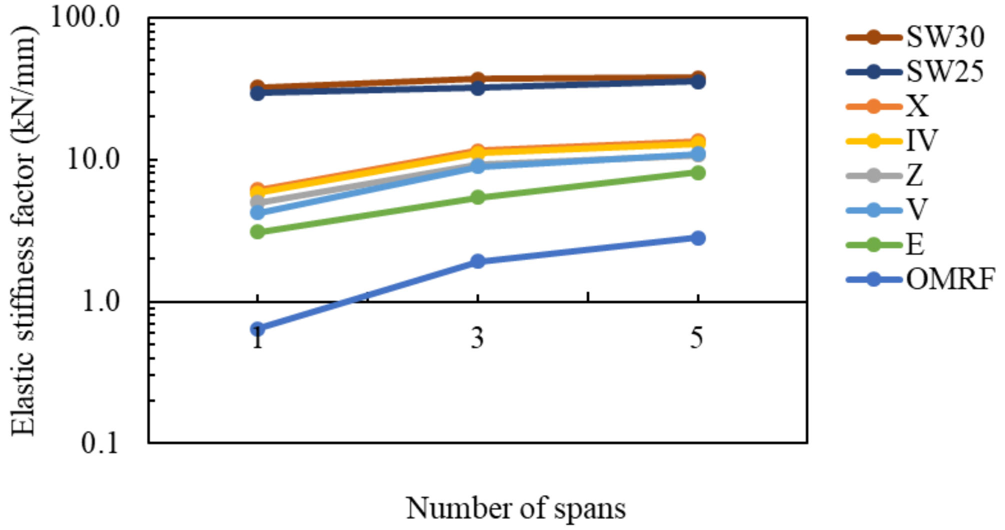

Figure A.1.32: The elastic stiffness factor of the frames versus the number of spans for different types of bracings and shear walls

Assuming fixed parameters S = H, L = 5.5 m and H = 3.2 m

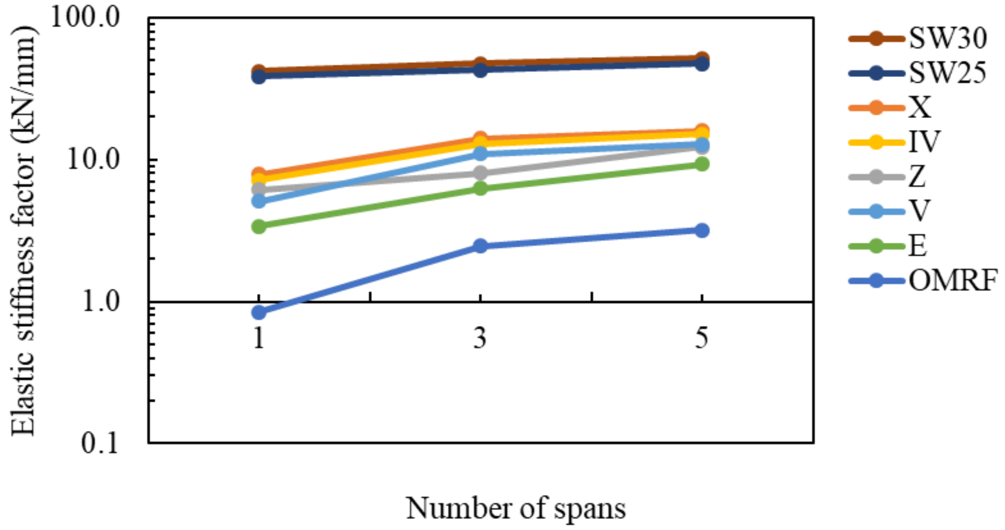

Figure A.1.33: The elastic stiffness factor of the frames versus the number of spans for different types of bracings and shear walls

Assuming fixed parameters S = H, L = 6 m and H = 3.2 m

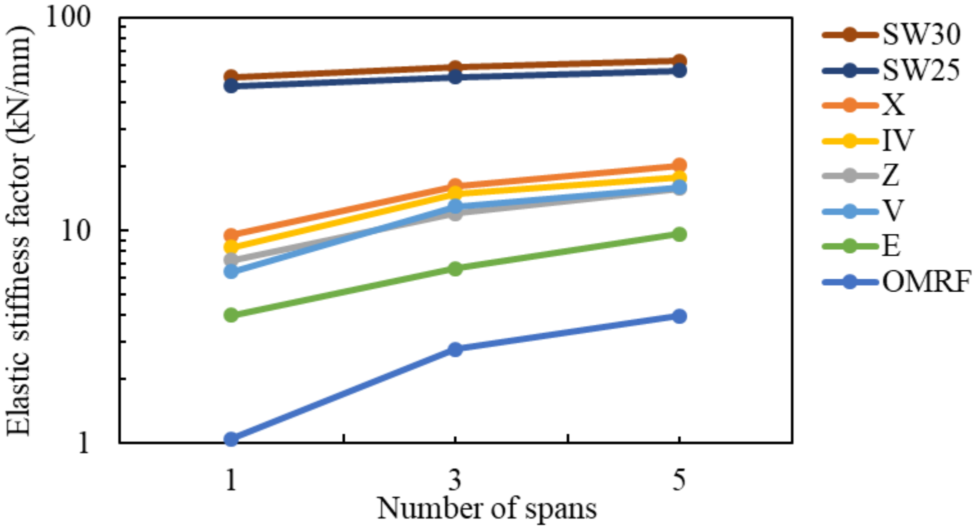

Figure A.1.34: The elastic stiffness factor of the frames versus the number of spans for different types of bracings and shear walls

Assuming fixed parameters S = H, L = 6.5 m and H = 3.2 m

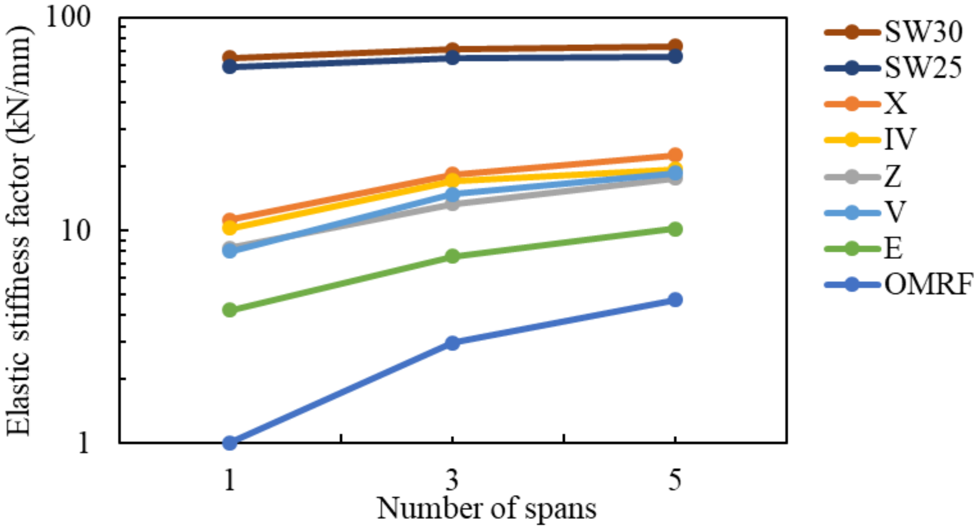

Figure A.1.35: The elastic stiffness factor of the frames versus the number of spans for different types of bracings and shear walls

A.1.1.4 The effect of story height change on the elastic stiffness factor of the steel frames for different types of bracings and shear walls

Assuming fixed parameters S= L, L = 5.5 m and N = 3

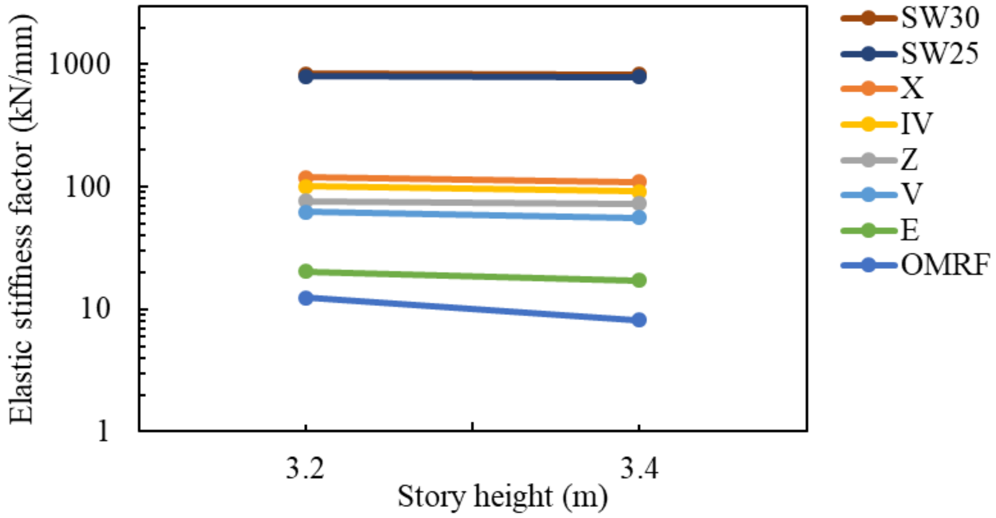

Figure A.1.36: The elastic stiffness factor of the frames versus the story height change for different types of bracings and shear walls

Assuming fixed parameters S= L, L = 6.5 m and N = 5

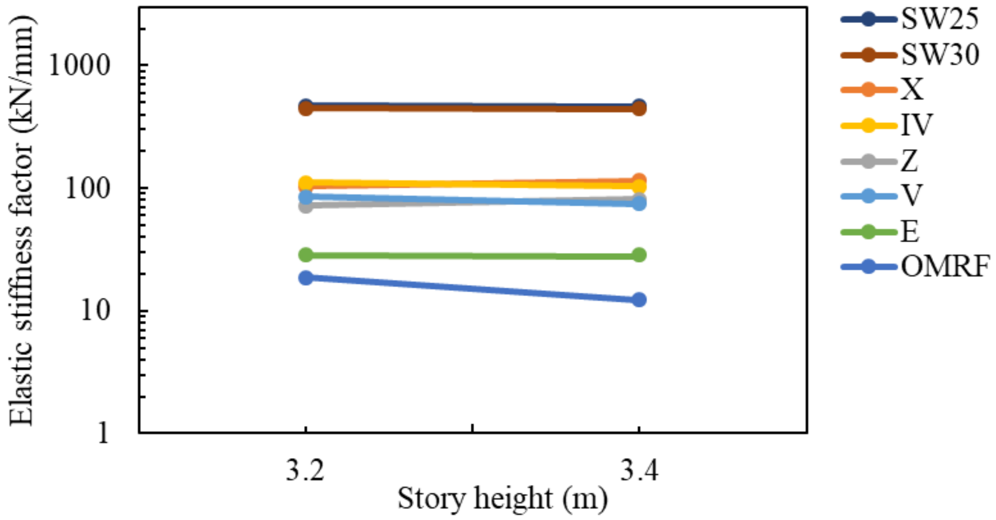

Figure A.1.37: The elastic stiffness factor of the frames versus the story height change for different types of bracings and shear walls

Assuming fixed parameters S= M, L = 4.5 m and N = 1

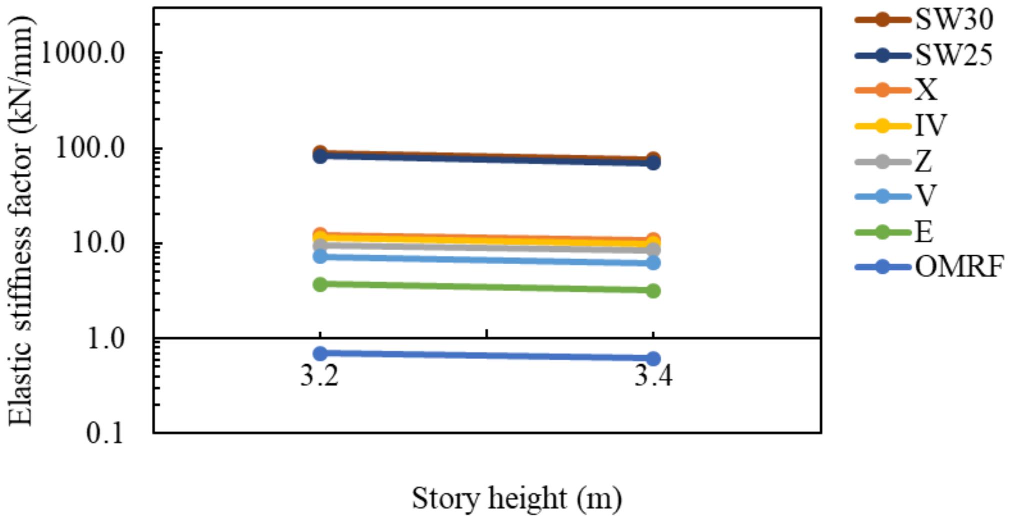

Figure A.1.38: The elastic stiffness factor of the frames versus the story height change for different types of bracings and shear walls

Assuming fixed parameters S= M, L = 5.5 m and N = 3

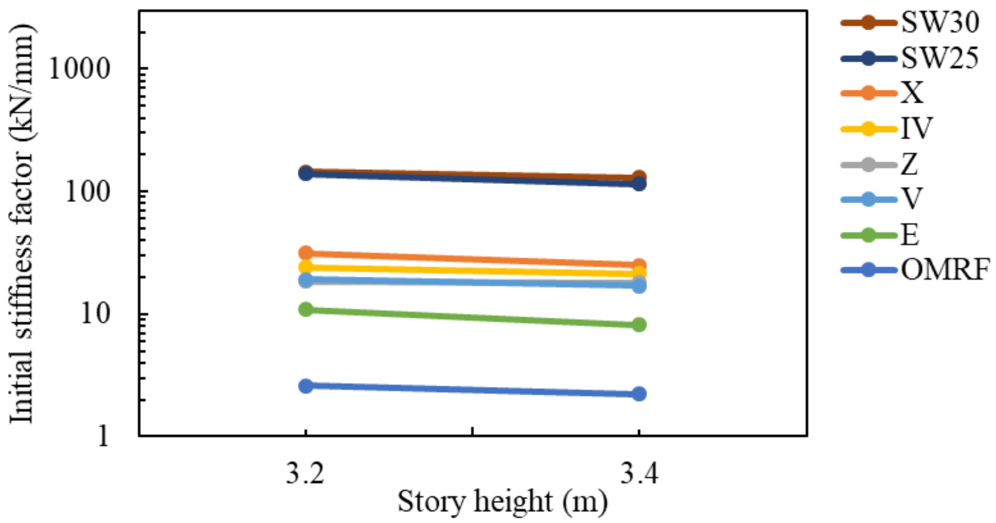

Figure A.1.39: The elastic stiffness factor of the frames versus the story height change for different types of bracings and shear walls

Assuming fixed parameters S= M, L = 6.5 m and N = 5

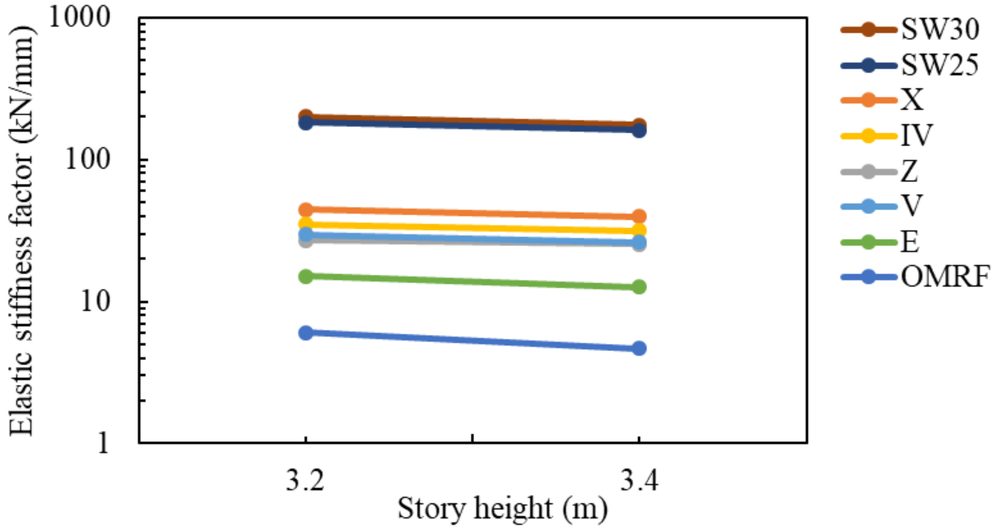

Figure A.1.40: The elastic stiffness factor of the frames versus the story height change for different types of bracings and shear walls

Assuming fixed parameters S= H, L = 4.5 m and N = 1

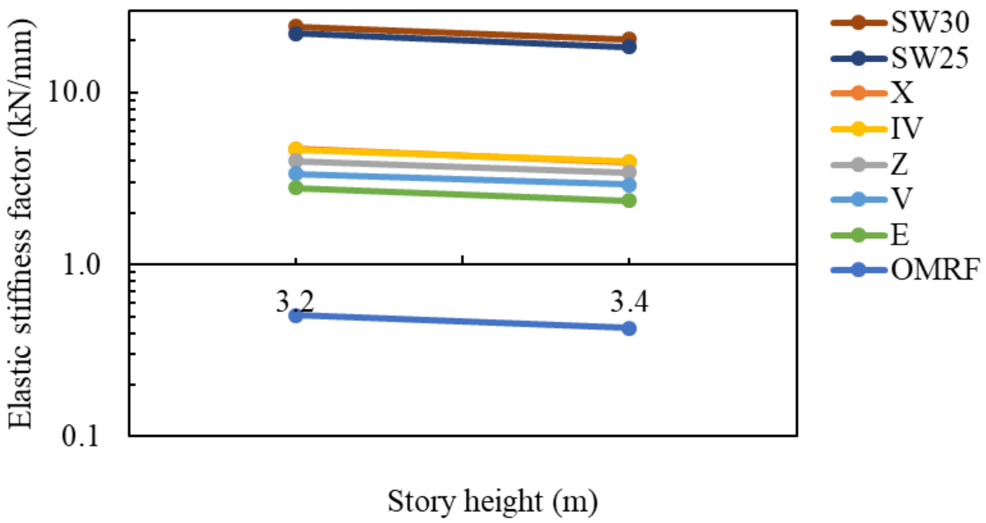

Figure A.1.41: The elastic stiffness factor of the frames versus the story height change for different types of bracings and shear walls

Assuming fixed parameters S= H, L = 5.5 m and N = 3

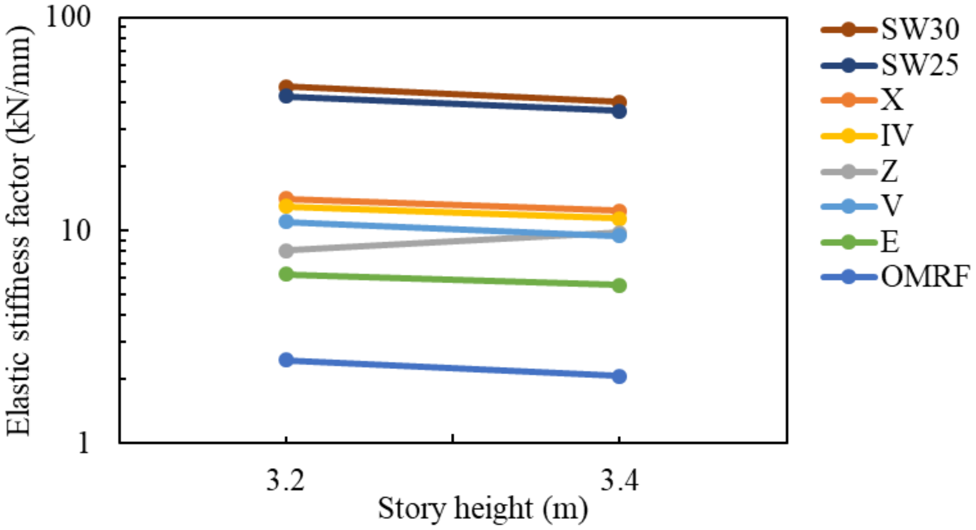

Figure A.1.42: The elastic stiffness factor of the frames versus the story height change for different types of bracings and shear walls

Assuming fixed parameters S= H, L = 6.5 m and N = 5

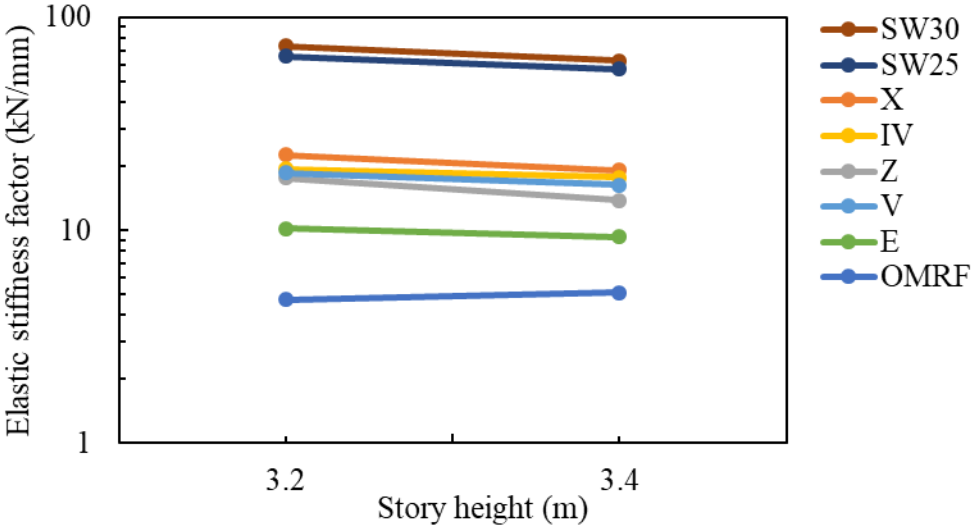

Figure A.1.43: The elastic stiffness factor of the frames versus the story height change for different types of bracings and shear walls

A.1.2 Factors Affecting Natural Time Period

A.1.2.1 The effect of span length on the natural time period of the steel frames for different types of bracing and shear walls

The parameters fixed for this figure are N = 3, S = L, H = 3.2 m

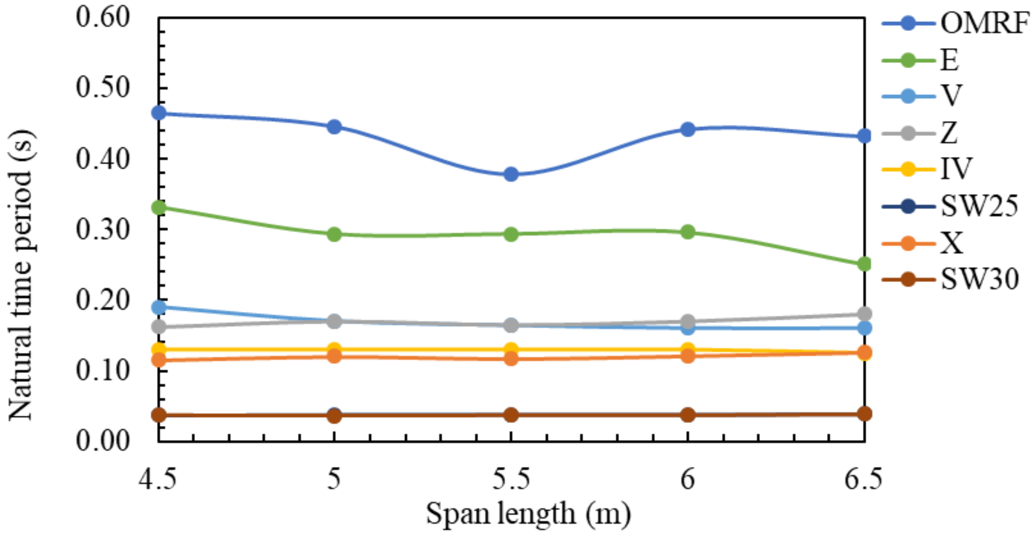

Figure A.1.44: The natural time period of the frames versus span length for different types of bracings and shear walls

The parameters fixed for this figure are N = 5, S = L, H = 3.2 m

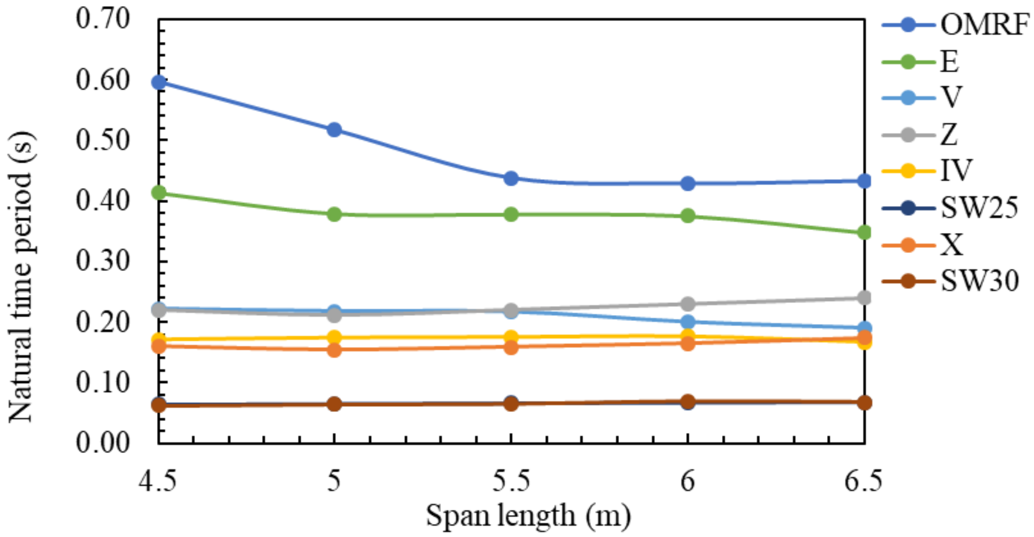

Figure A.1.45: The natural time period of the frames versus span length for different types of bracings and shear walls

The parameters fixed for this figure are N = 1, S = M, H = 3.2 m

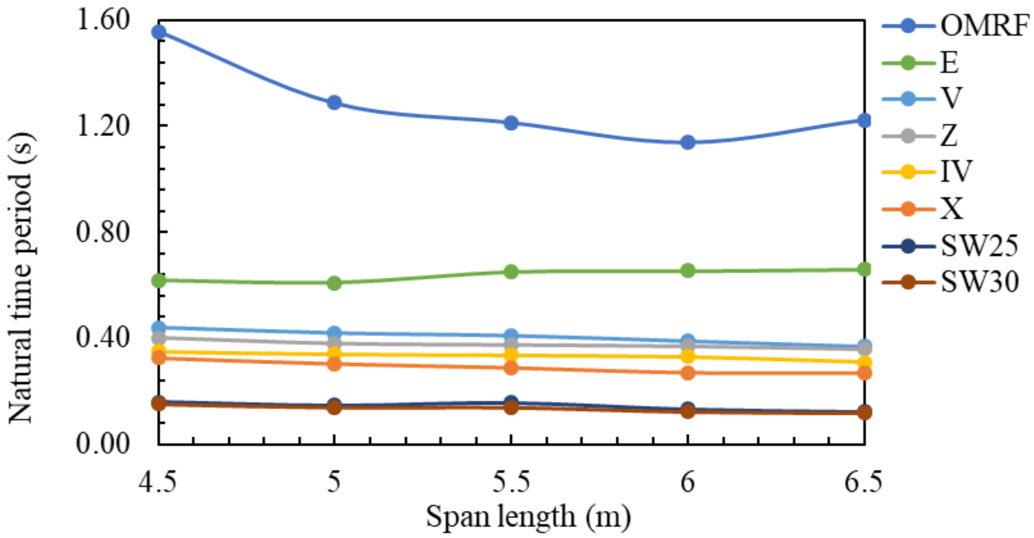

Figure A.1.46: The natural time period of the frames versus span length for different types of bracings and shear walls

The parameters fixed for this figure are N = 3, S = M, H = 3.2 m

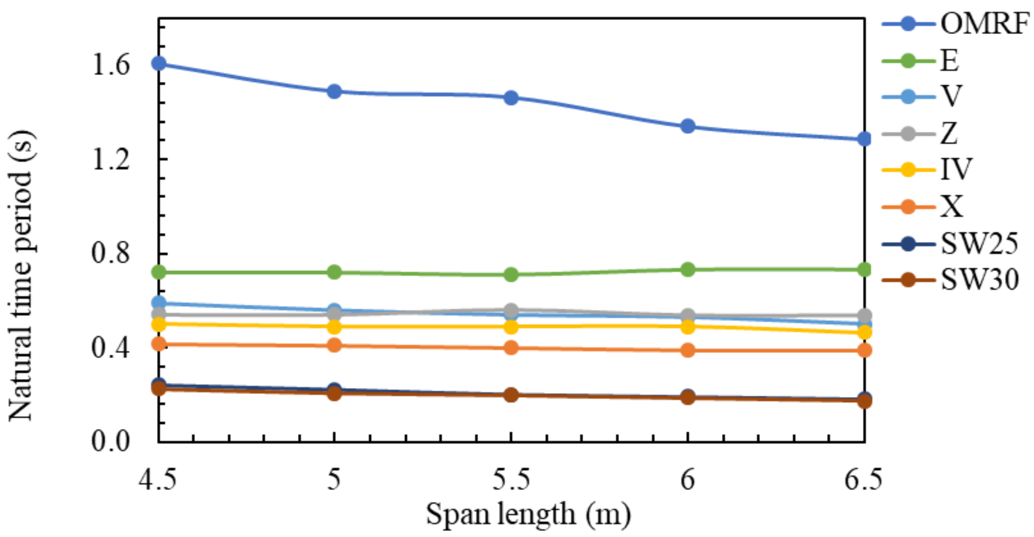

Figure A.1.47: The natural time period of the frames versus span length for different types of bracings and shear walls

The parameters fixed for this figure are N = 5, S = M, H = 3.2 m

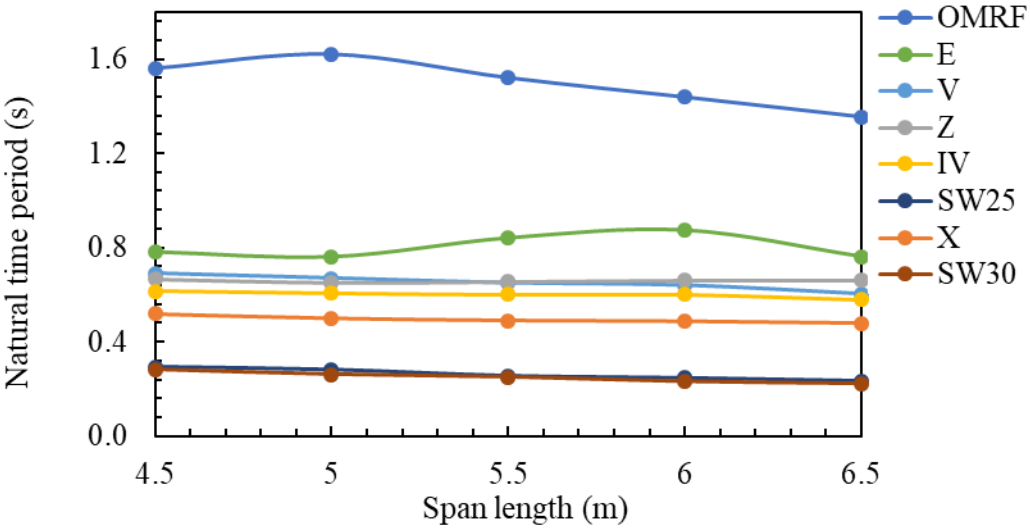

Figure A.1.48: The natural time period of the frames versus span length for different types of bracings and shear walls

The parameters fixed for this figure are N = 1, S = H, H = 3.2 m

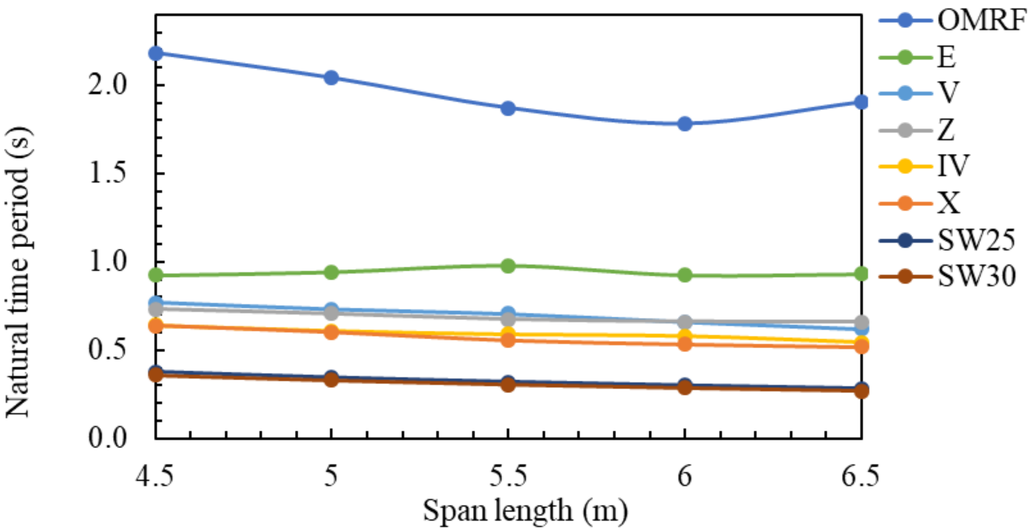

Figure A.1.49: The natural time period of the frames versus span length for different types of bracings and shear walls

The parameters fixed for this figure are N = 3, S = H, H = 3.2 m

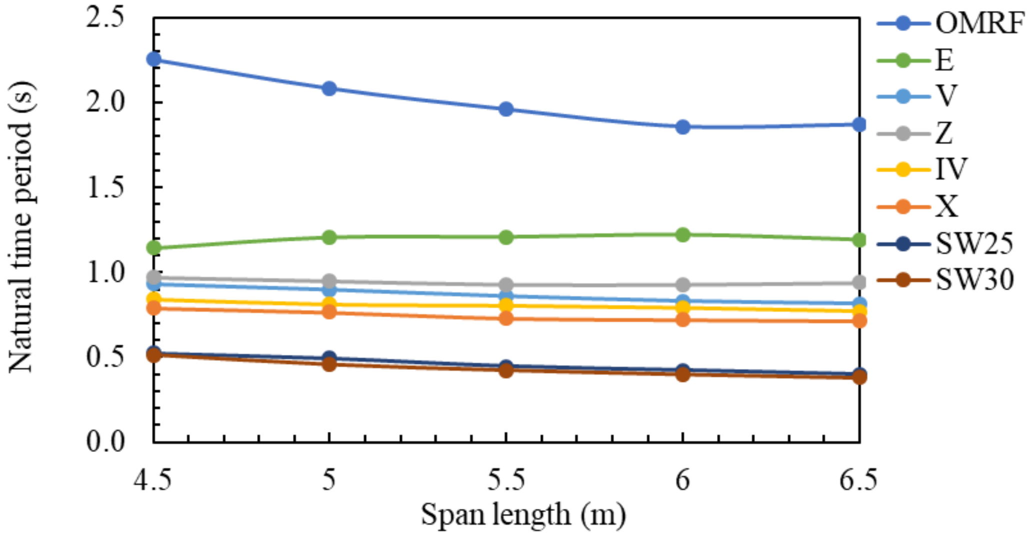

Figure A.1.50: The natural time period of the frames versus span length for different types of bracings and shear walls

The parameters fixed for this figure are N = 5, S = H, H = 3.2 m

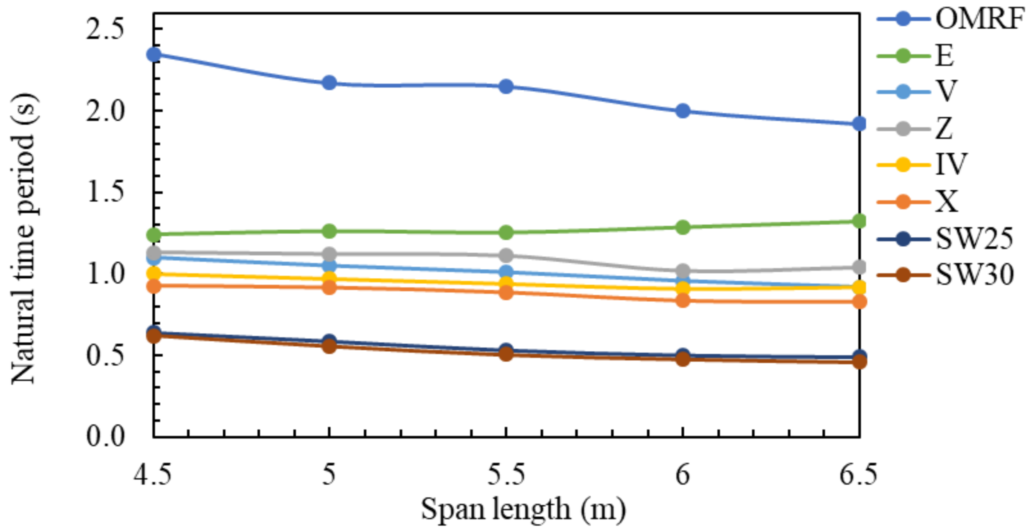

Figure A.1.51: The natural time period of the frames versus span length for different types of bracings and shear walls

A.1.2.2 The influence of storey number change on the natural time period of the steel frames for shear walls and bracings

Assuming fixed parameters N = 1, L = 5 m and H = 3.2 m

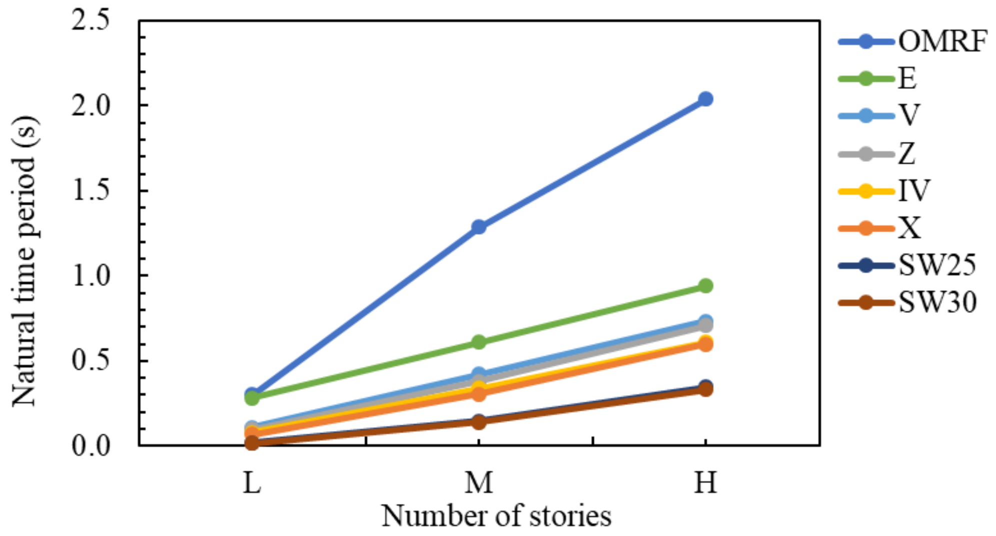

Figure A.1.52: The natural time period of steel frames versus number of stories for different types of bracings and shear walls

Assuming fixed parameters N = 1, L = 5.5 m and H = 3.2 m

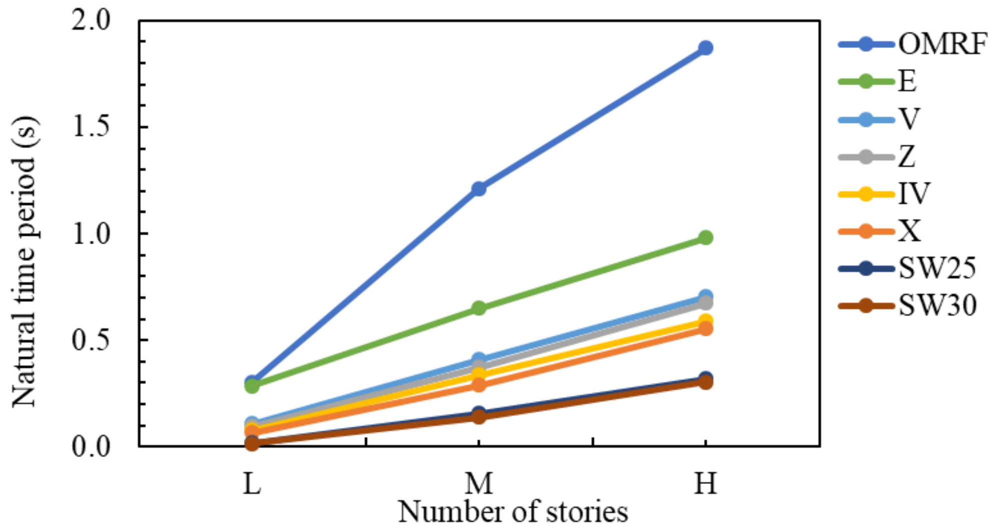

Figure A.1.53: The natural time period of steel frames versus number of stories for different types of bracings and shear walls

Assuming fixed parameters N = 1, L = 6 m and H = 3.2 m

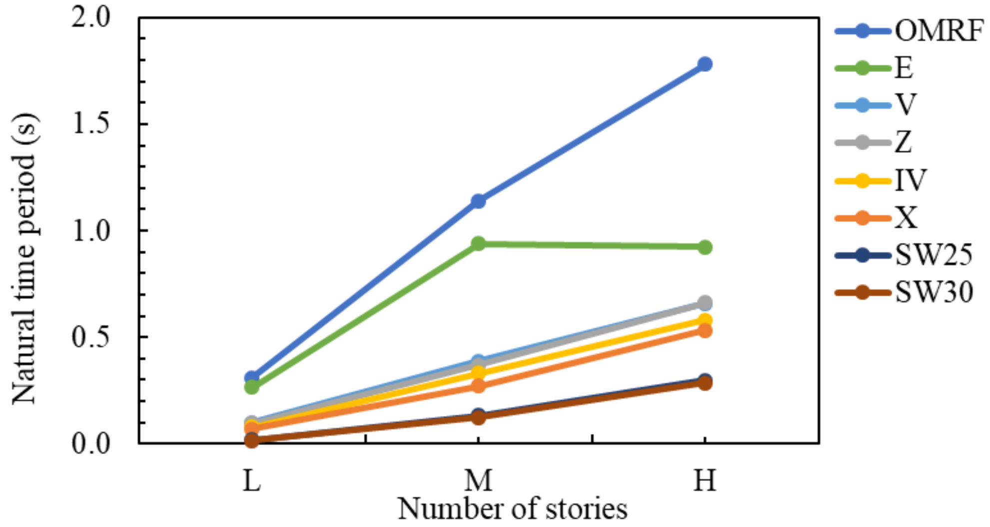

Figure A.1.54: The natural time period of steel frames versus number of stories for different types of bracings and shear walls

Assuming fixed parameters N = 1, L = 6.5 m and H = 3.2 m

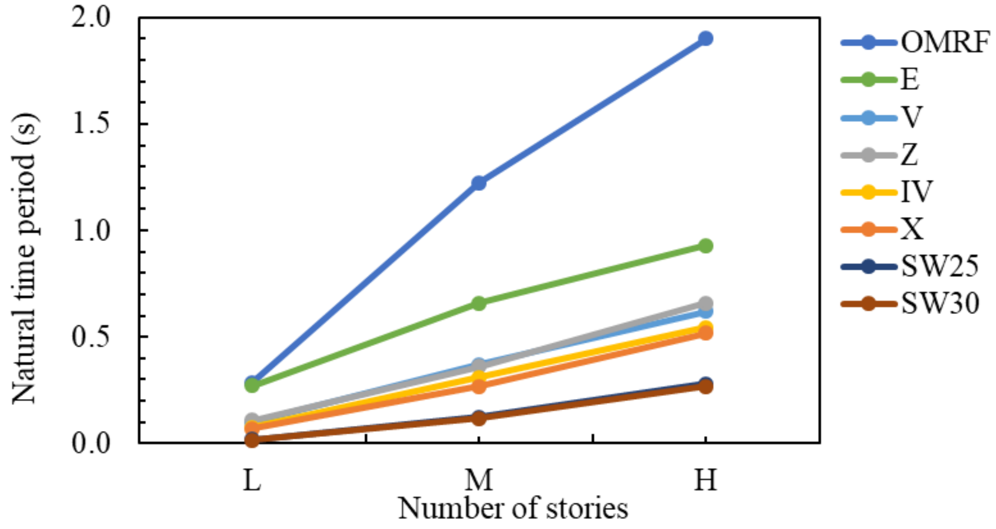

Figure A.1.55: The natural time period of steel frames versus number of stories for different types of bracings and shear walls

Assuming fixed parameters N = 3, L = 4.5 m and H =3.2 m

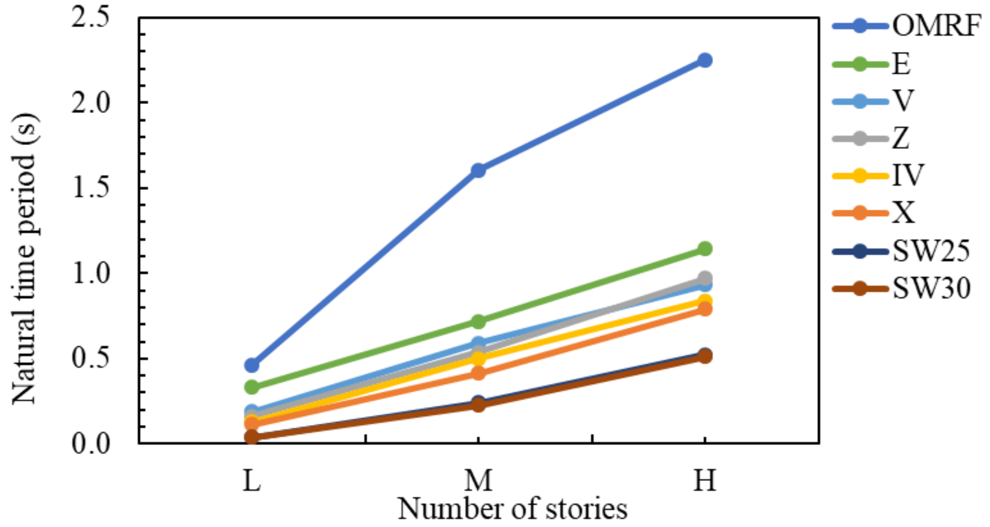

Figure A.1.56: The natural time period of steel frames versus number of stories for different types of bracings and shear walls

Assuming fixed parameters N = 3, L = 5 m and H = 3.2

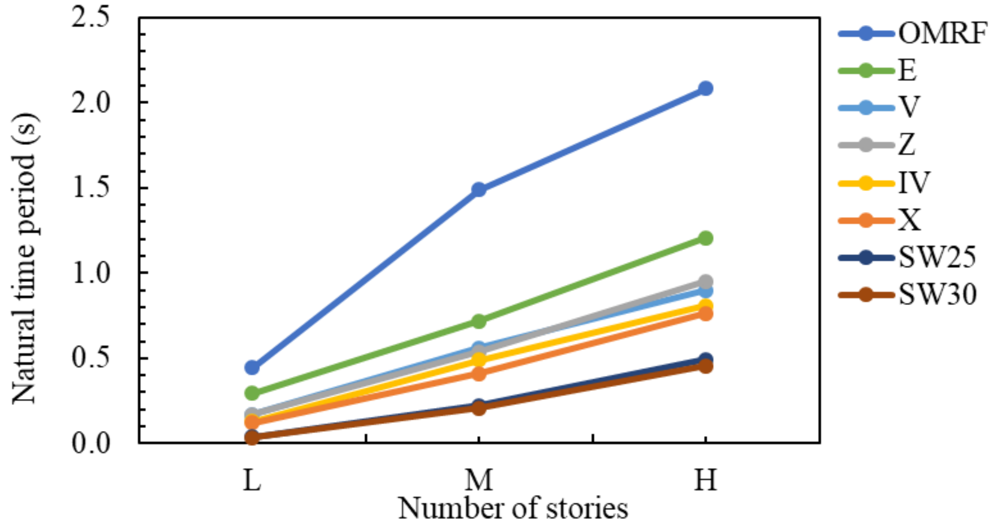

Figure A.1.57: The natural time period of steel frames versus number of stories for different types of bracings and shear walls

Assuming fixed parameters N = 3, L = 5.5 m and H = 3.2 m

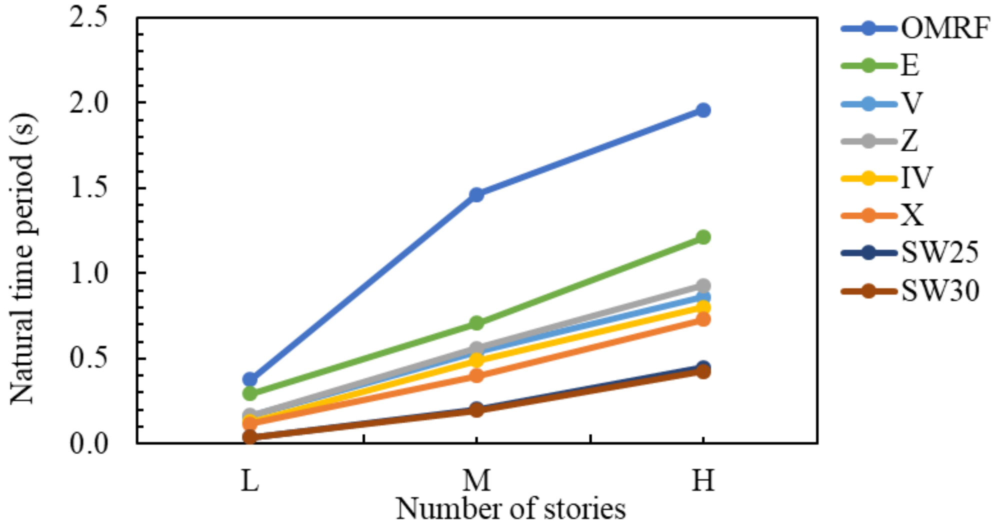

Figure A.1.58: The natural time period of steel frames versus number of stories for different types of bracings and shear walls

Assuming fixed parameters N = 3, L = 6 m and H = 3.2 m

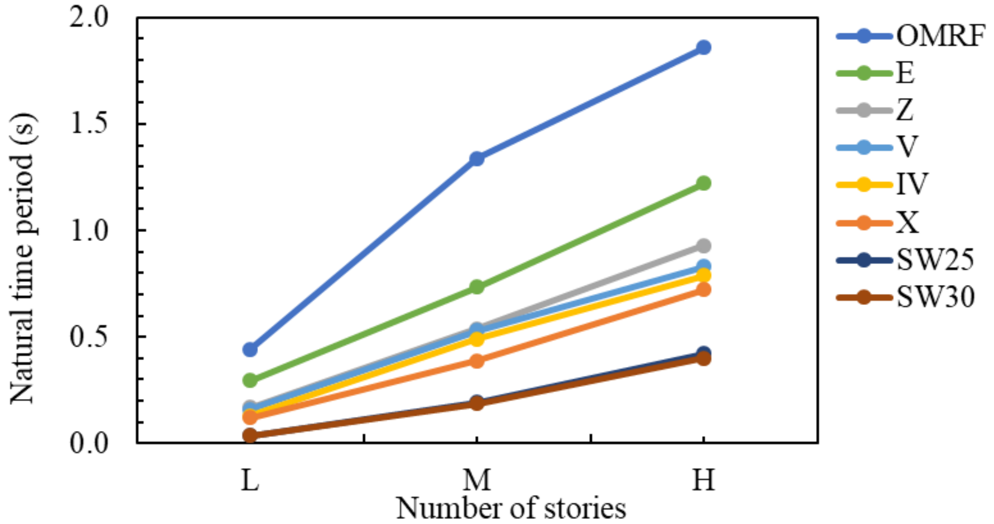

Figure A.1.59: The natural time period of steel frames versus number of stories for different types of bracings and shear walls

Assuming fixed parameters N = 3, L = 6.5 m and H = 3.2 m

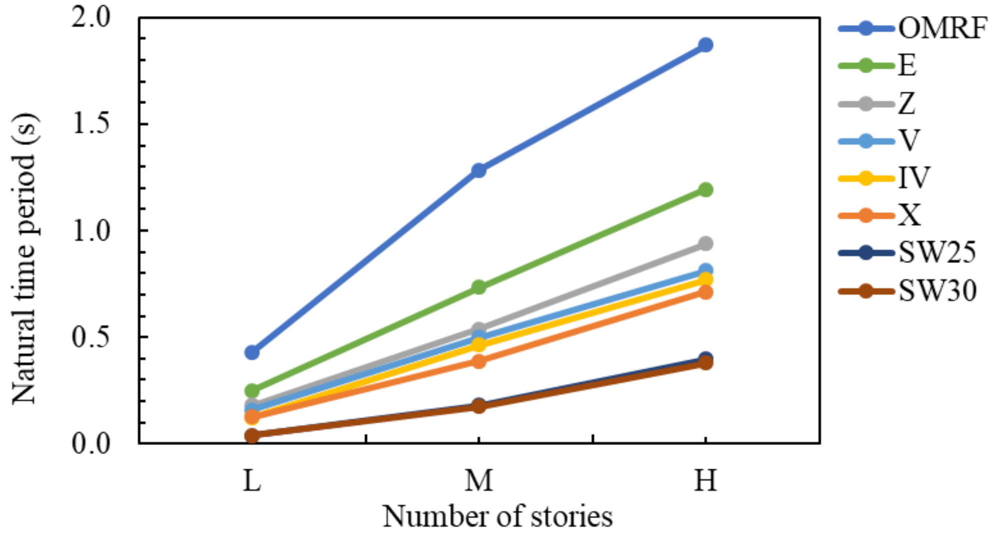

Figure A.1.60: The natural time period of steel frames versus number of stories for different types of bracings and shear walls

Assuming fixed parameters N = 5, L = 4.5 m and H = 3.2 m

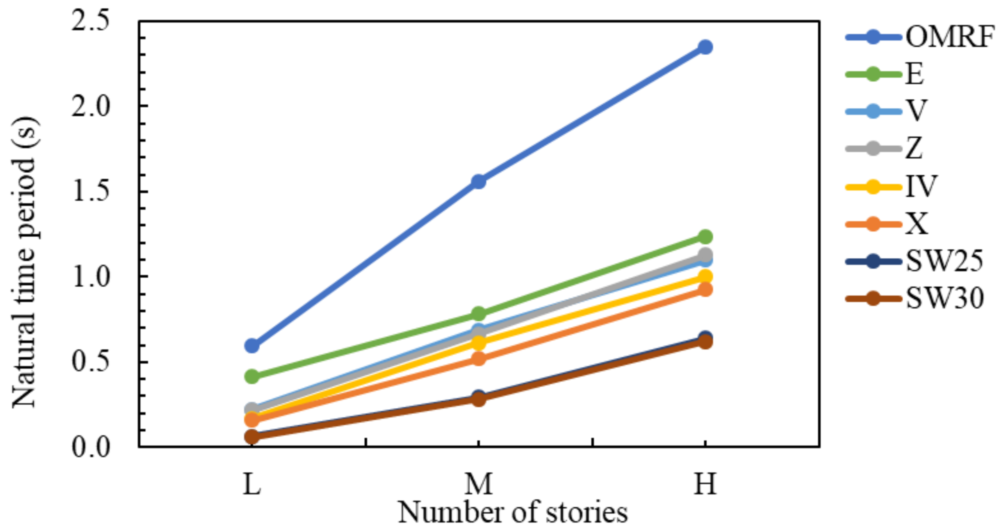

Figure A.1.61: The natural time period of steel frames versus number of stories for different types of bracings and shear walls

Assuming fixed parameters N = 5, L = 5 m and H = 3.2 m

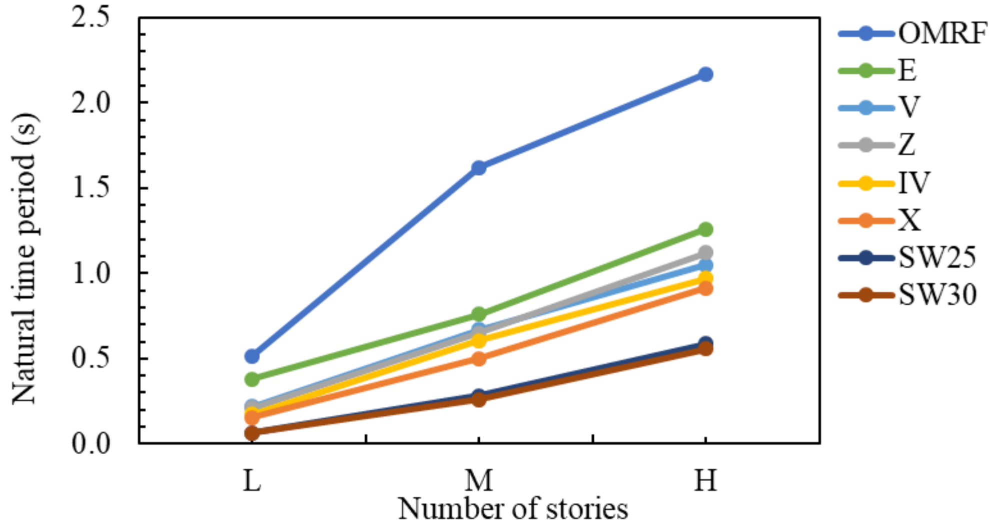

Figure A.1.62: The natural time period of steel frames versus number of stories for different types of bracings and shear walls

Assuming fixed parameters N = 5, L = 5.5 m and H = 3.2 m

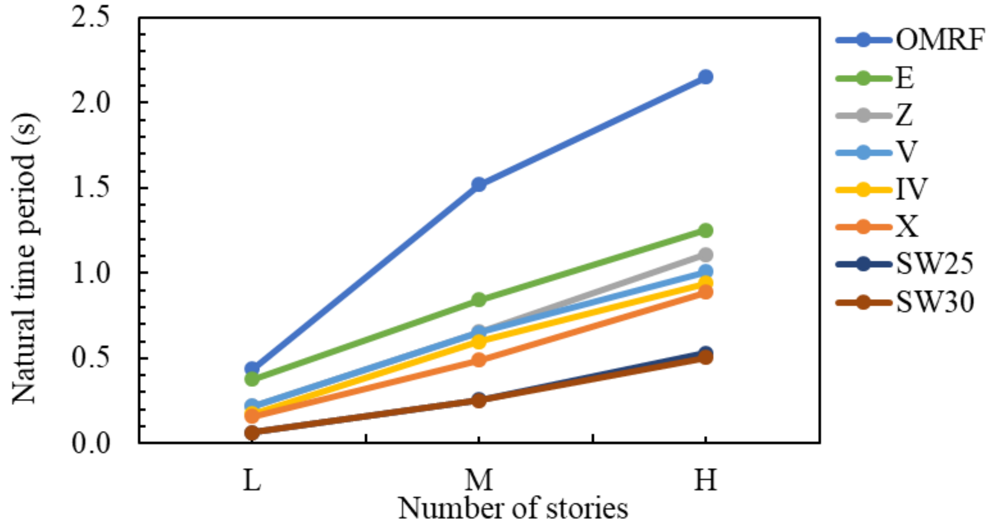

Figure A.1.63: The natural time period of steel frames versus number of stories for different types of bracings and shear walls

Assuming fixed parameters N = 5, L = 6 m and H = 3.2 m

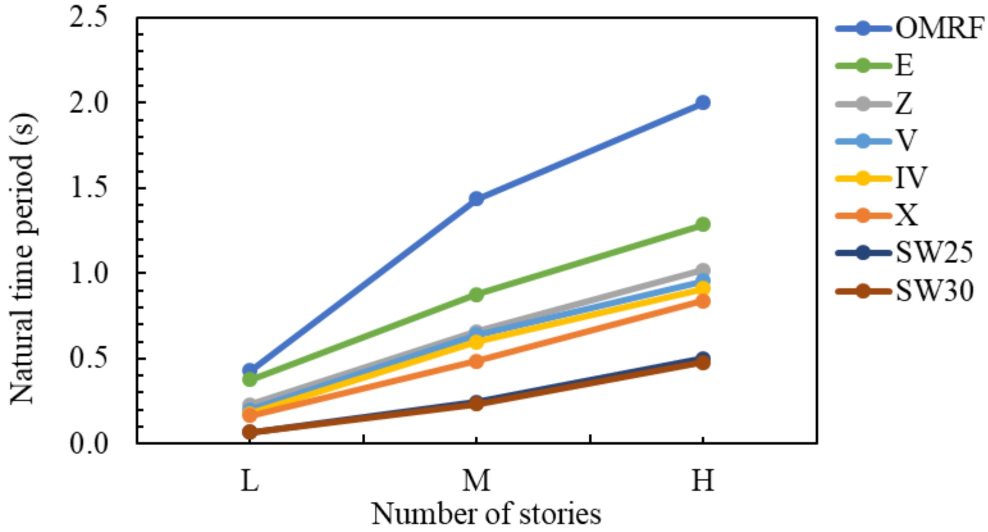

Figure A.1.64: The natural time period of steel frames versus number of stories for different types of bracings and shear walls

Assuming fixed parameters N = 5, L = 6.5 m and H = 3.2 m

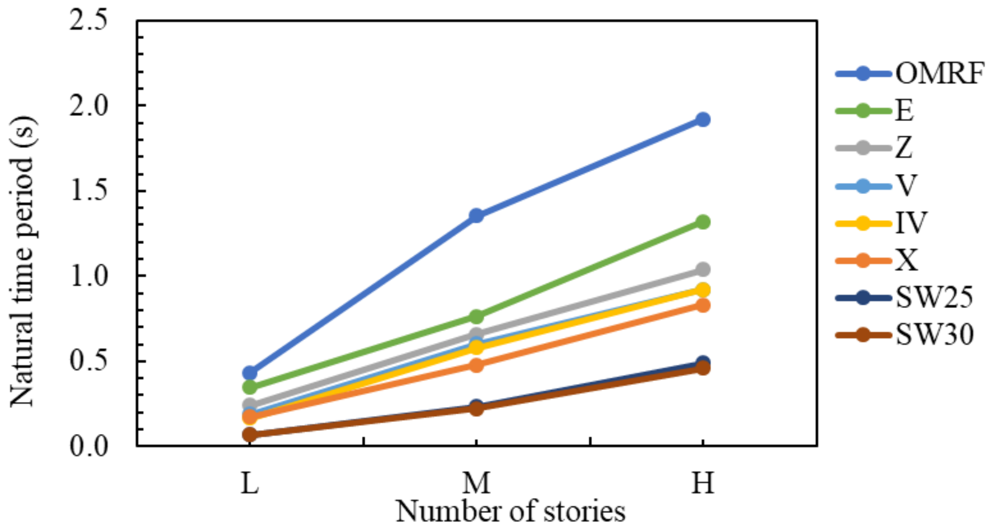

Figure A.1.65: The natural time period of steel frames versus number of stories for different types of bracings and shear walls

A.1.2.3 The effect of number of spans on the natural time period of the steel frames for different types of bracings and shear walls

Assuming fixed parameters S= L, L = 5 m and H = 3.2 m

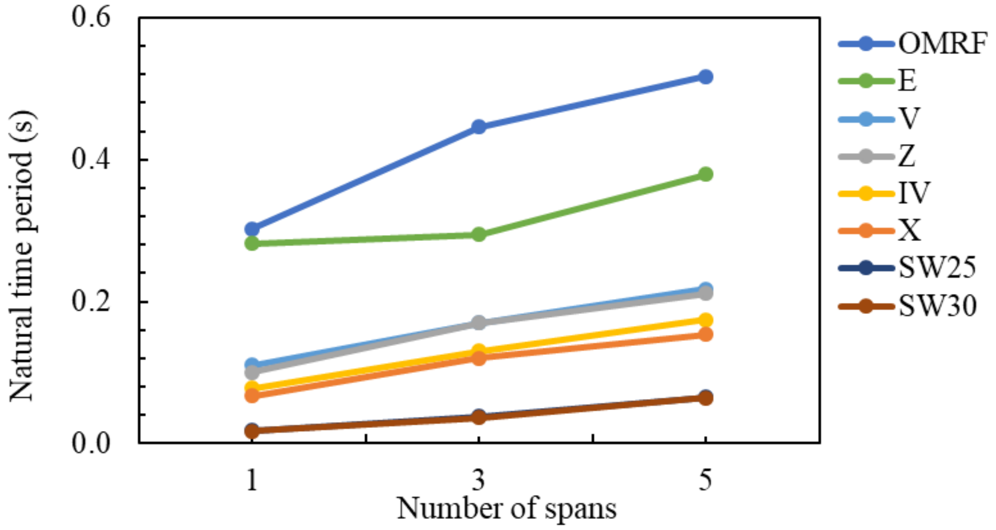

Figure A.1.66: The natural time period of the frames versus the number of spans for different types of bracings and shear walls

Assuming fixed parameters S = L, L = 5.5 m and H = 3.2 m

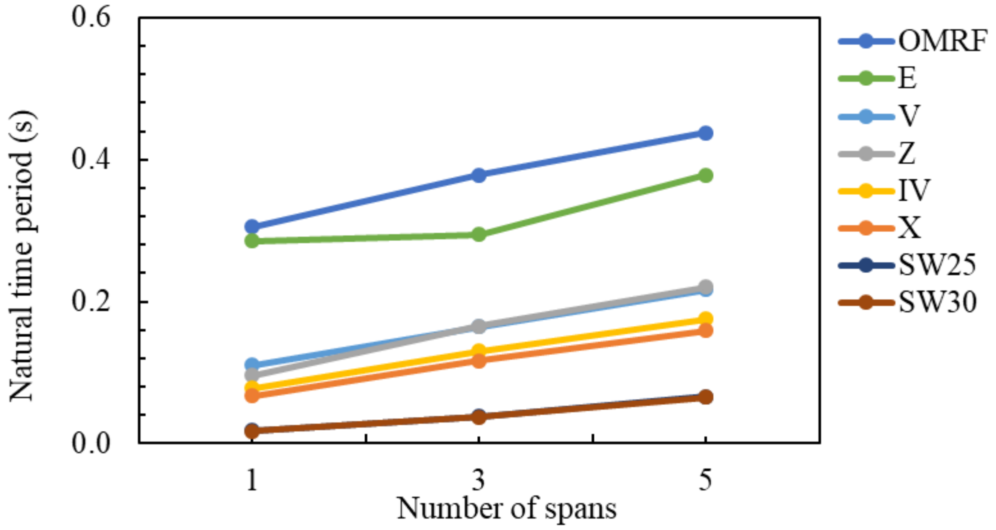

Figure A.1.67: The natural time period of the frames versus the number of spans for different types of bracings and shear walls

Assuming fixed parameters S = L, L = 6 m and H = 3.2 m

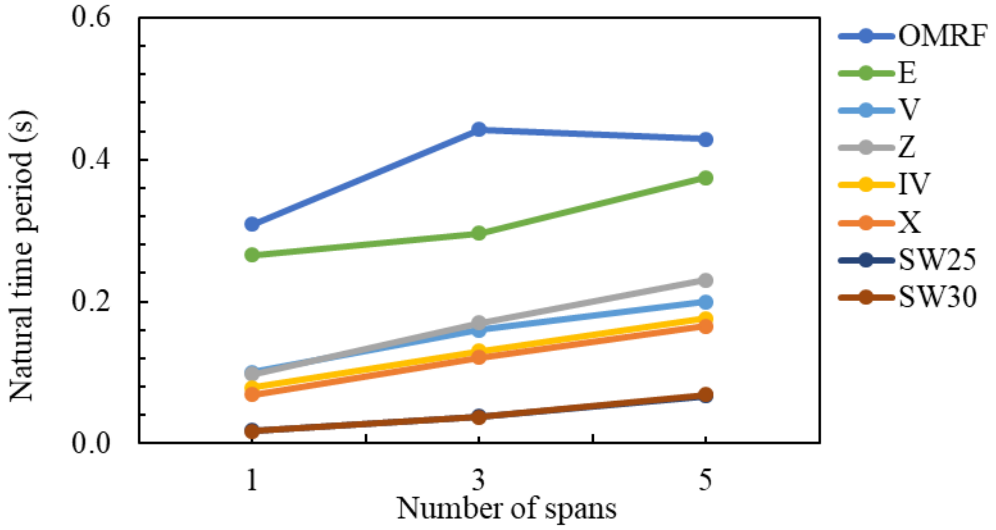

Figure A.1.68: The natural time period of the frames versus the number of spans for different types of bracings and shear walls

Assuming fixed parameters S = L, L = 6.5 m and H = 3.2 m

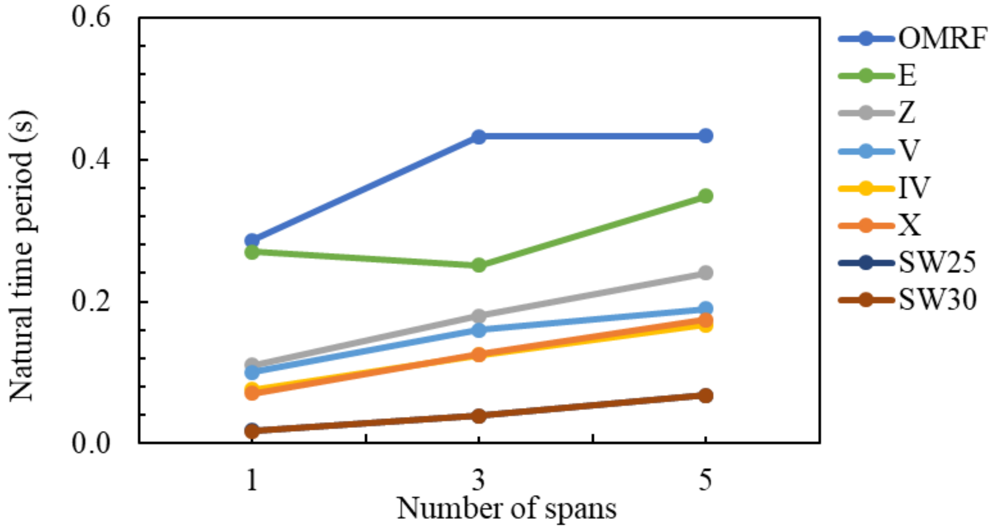

Figure A.1.69: The natural time period of the frames versus the number of spans for different types of bracings and shear walls

Assuming fixed parameters S = M, L = 4.5 m and H = 3.2 m

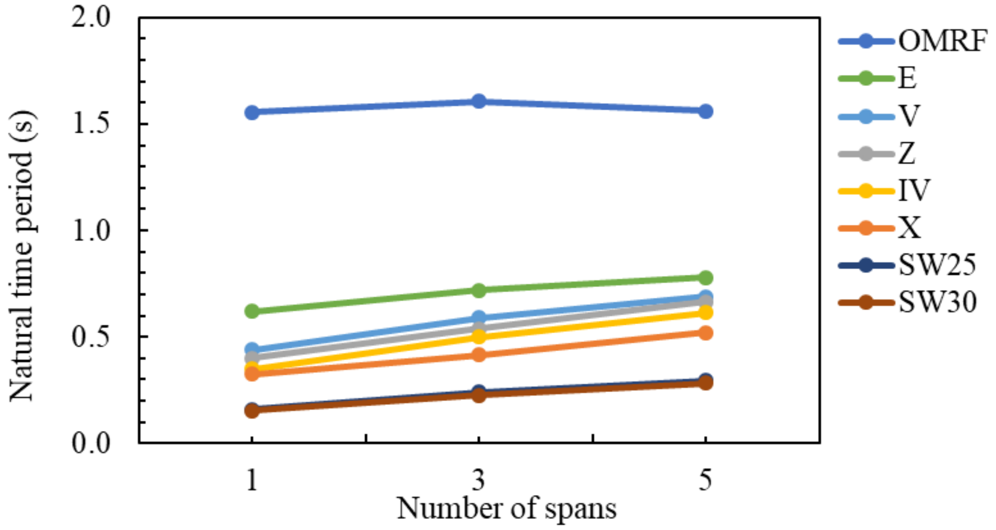

Figure A.1.70: The natural time period of the frames versus the number of spans for different types of bracings and shear walls

Assuming fixed parameters S = M, L = 5 m and H = 3.2 m

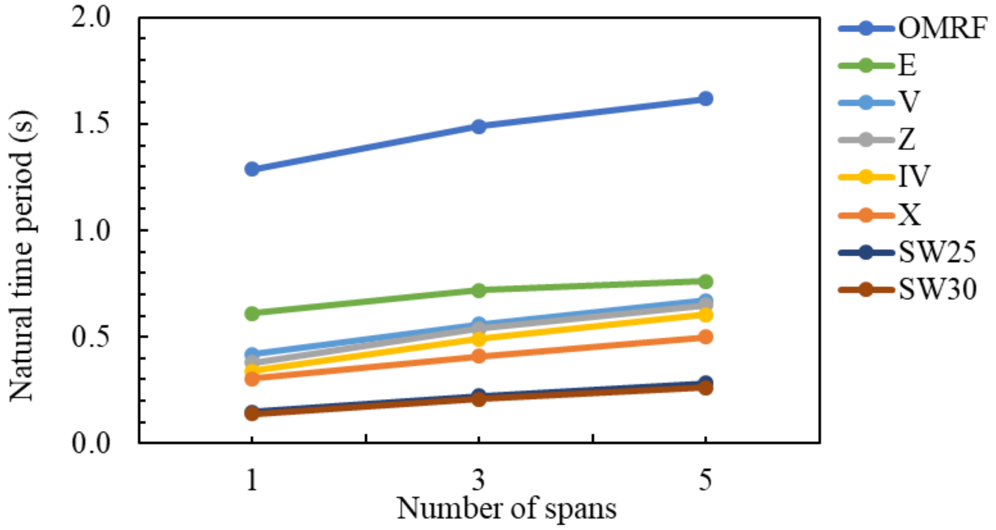

Figure A.1.71: The natural time period of the frames versus the number of spans for different types of bracings and shear walls

Assuming fixed parameters S = M, L = 5.5 m and H = 3.2 m

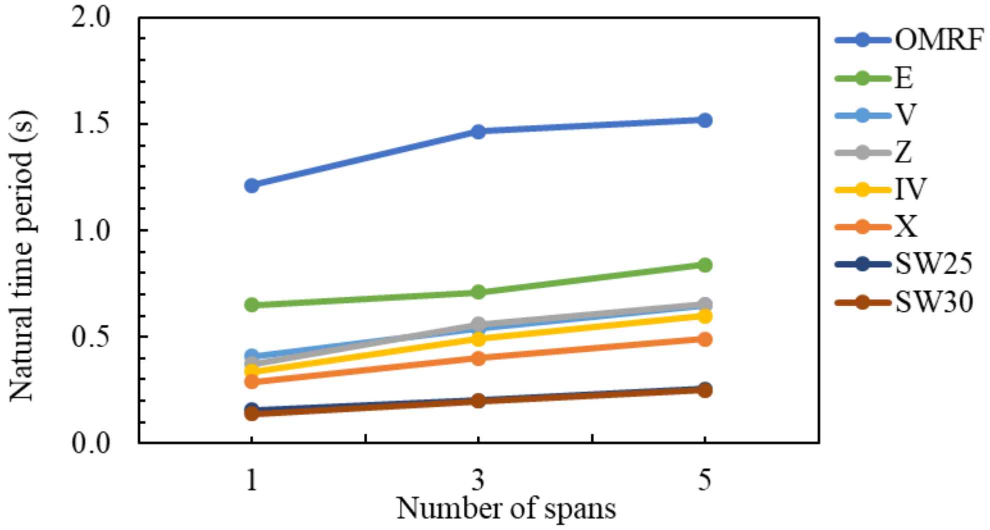

Figure A.1.72: The natural time period of the frames versus the number of spans for different types of bracings and shear walls

Assuming fixed parameters S = M, L = 6 m and H = 3.2 m

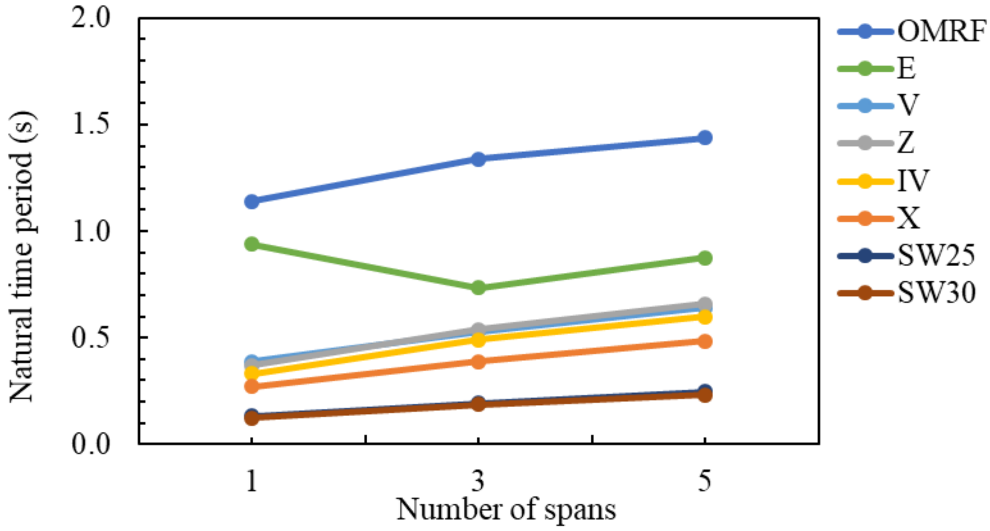

Figure A.1.73: The natural time period of the frames versus the number of spans for different types of bracings and shear walls

Assuming fixed parameters S = M, L = 6.5 m and H = 3.2 m

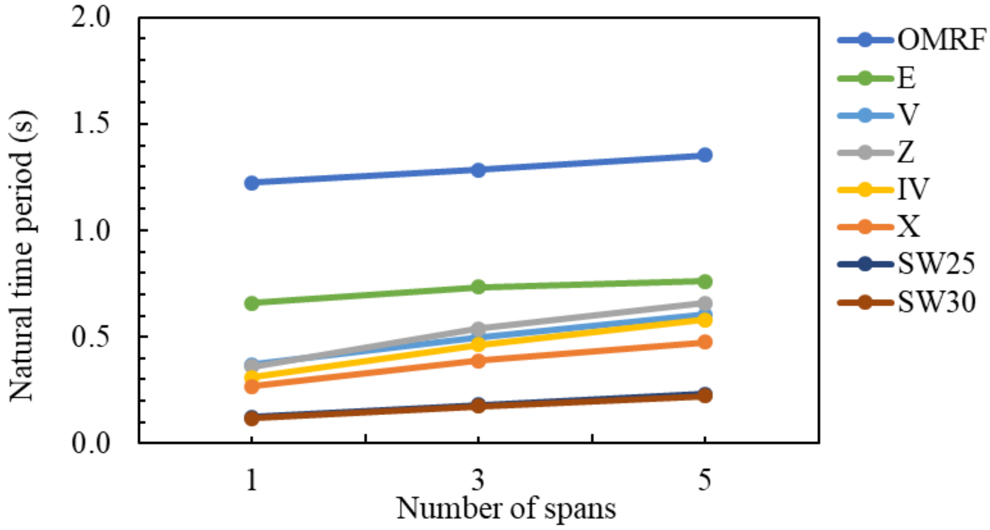

Figure A.1.74: The natural time period of the frames versus the number of spans for different types of bracings and shear walls

Assuming fixed parameters S = H, L = 4.5 m and H = 3.2 m

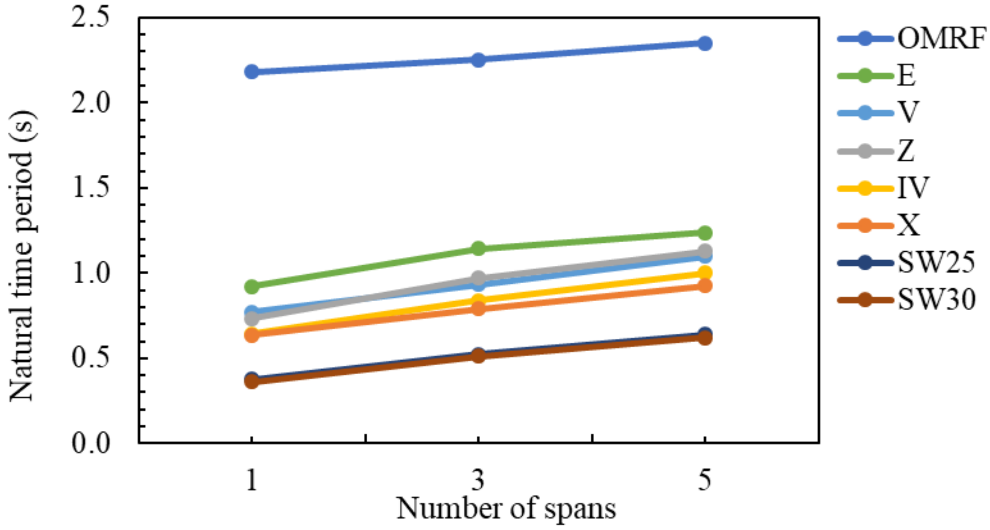

Figure A.1.75: The natural time period of the frames versus the number of spans for different types of bracings and shear walls

Assuming fixed parameters S = H, L = 5 m and H = 3.2 m

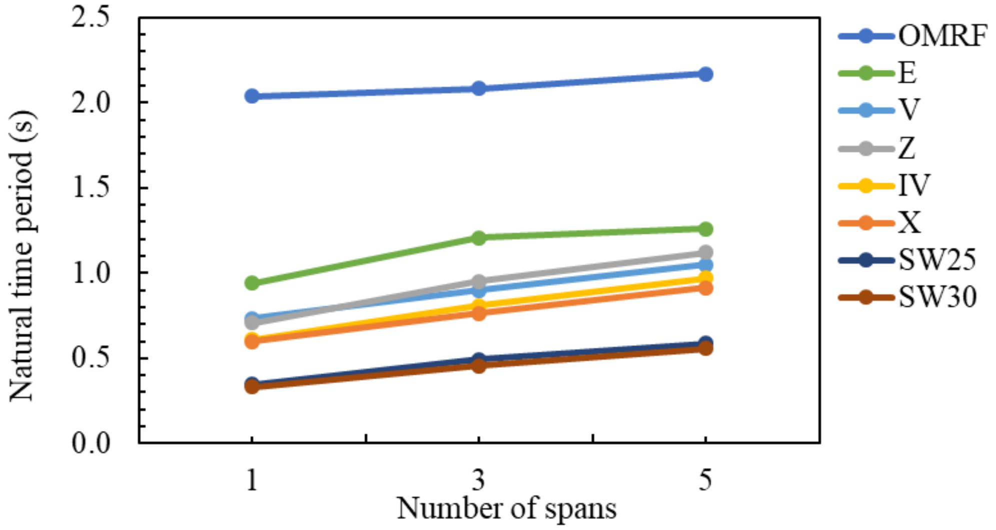

Figure A.1.76: The natural time period of the frames versus the number of spans for different types of bracings and shear walls

Assuming fixed parameters S = H, L = 5.5 m and H = 3.2 m

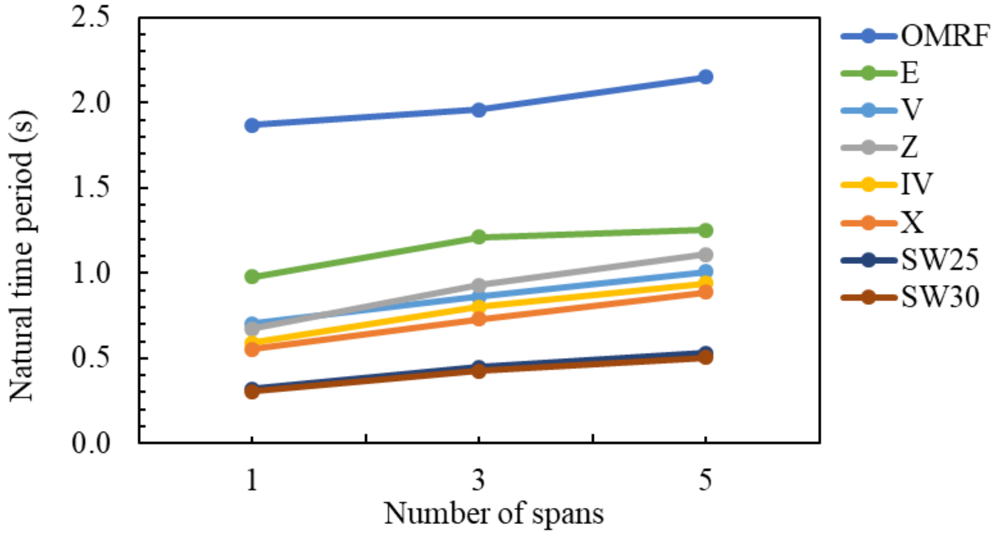

Figure A.1.77: The natural time period of the frames versus the number of spans for different types of bracings and shear walls

Assuming fixed parameters S = H, L = 6 m and H = 3.2 m

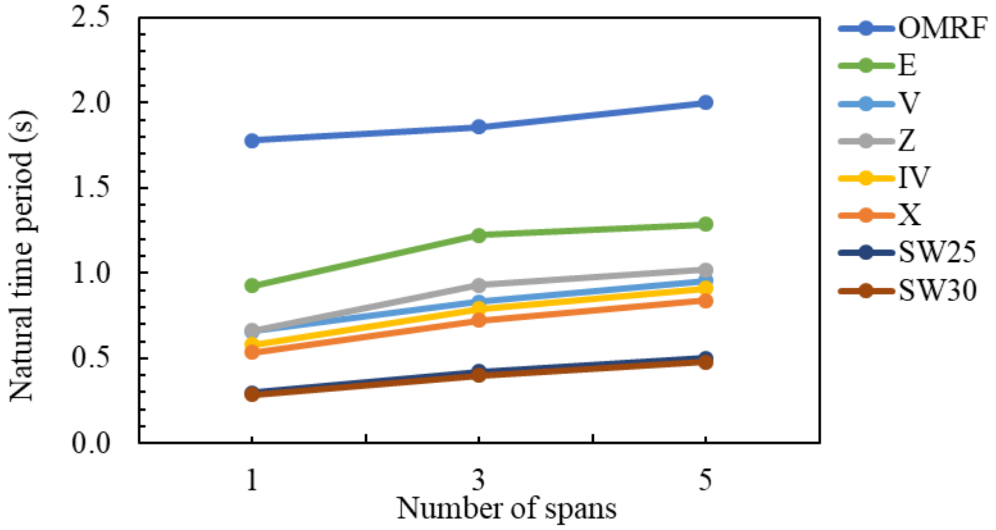

Figure A.1.78: The natural time period of the frames versus the number of spans for different types of bracings and shear walls

Assuming fixed parameters S = H, L = 6.5 m and H = 3.2 m

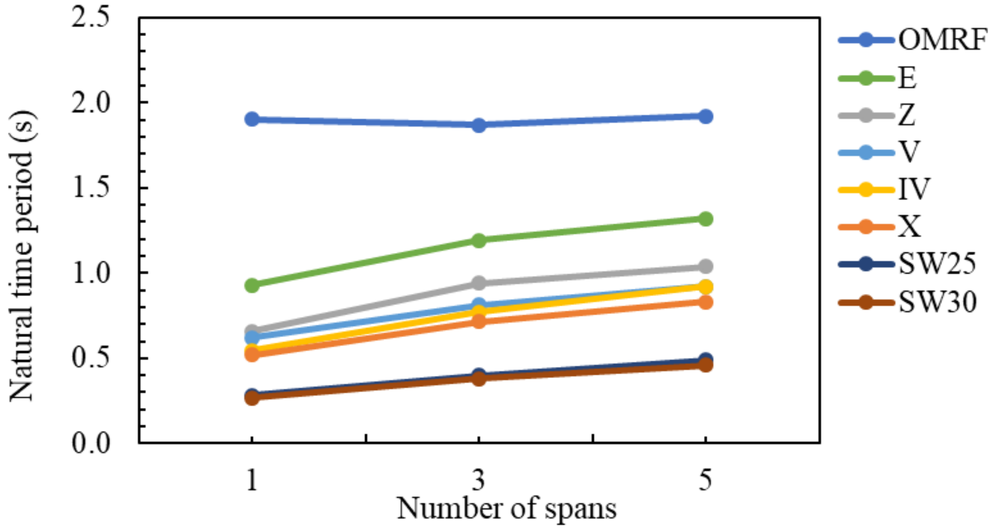

Figure A.1.79: The natural time period of the frames versus the number of spans for different types of bracings and shear walls

A.1.2.4 The effect of story height change on the natural time period of the steel frames for different types of bracings and shear walls

Assuming fixed parameters S= L, L = 5.5 m and N = 3

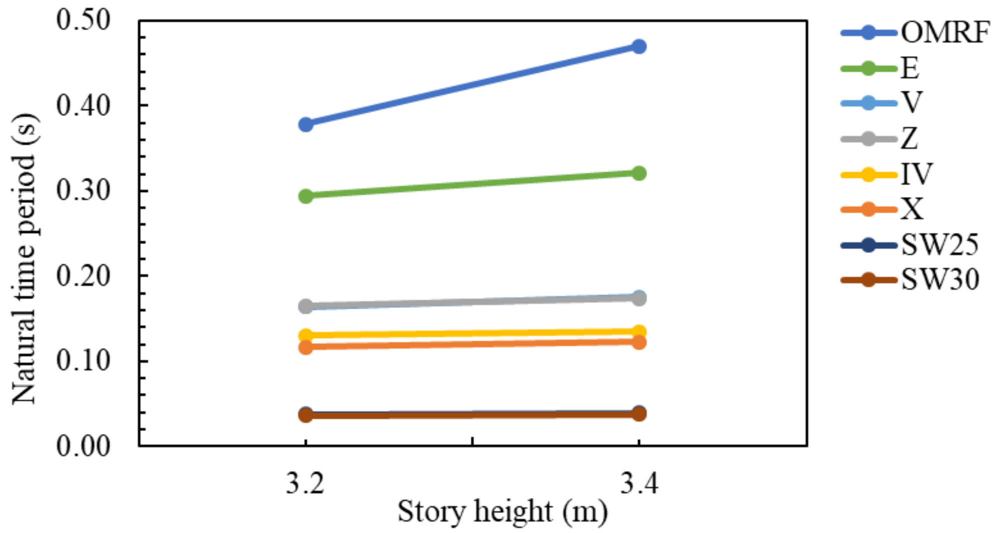

Figure A.1.80: The natural time period of the frames versus the story height change for different types of bracings and shear walls

Assuming fixed parameters S= L, L = 6.5 m and N = 5

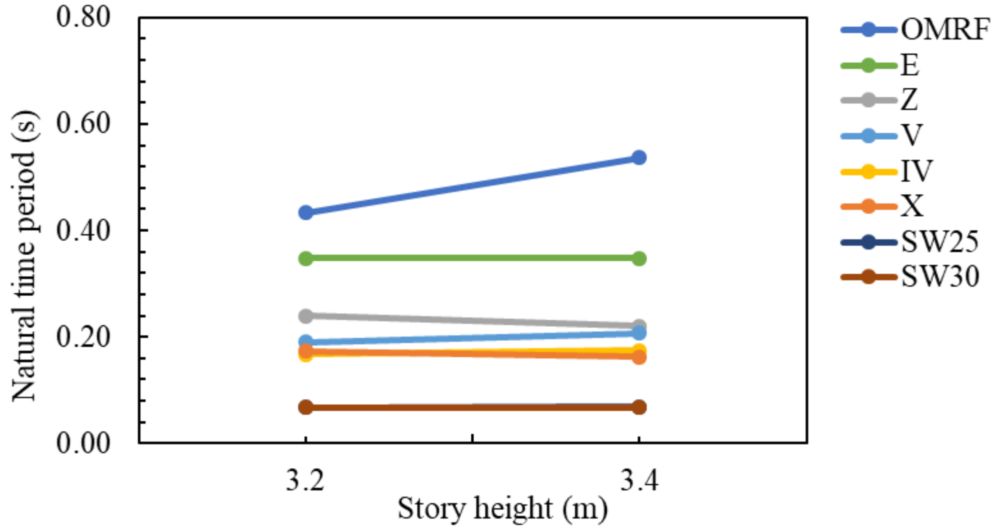

Figure A.1.81: The natural time period of the frames versus the story height change for different types of bracings and shear walls

Assuming fixed parameters S= M, L = 4.5 m and N = 1

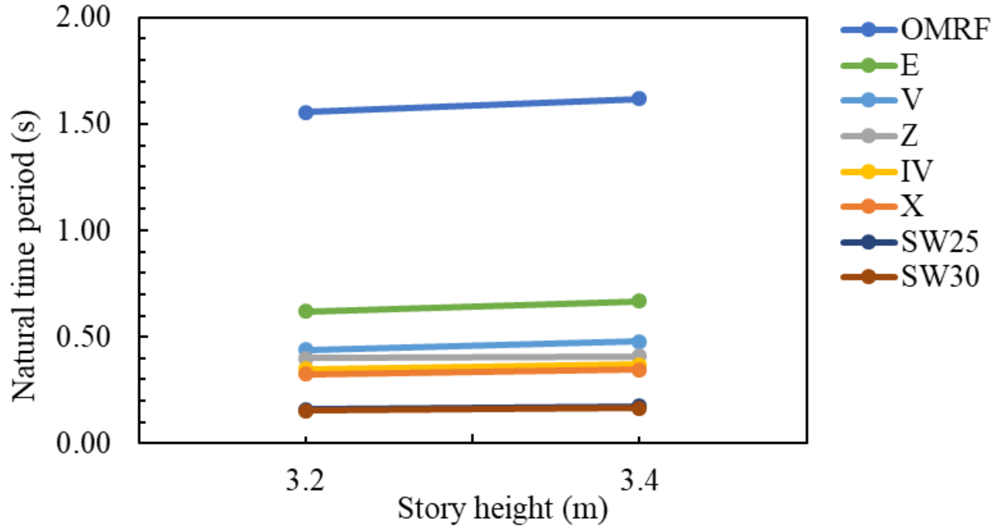

Figure A.1.82: The natural time period of the frames versus the story height change for different types of bracings and shear walls

Assuming fixed parameters S= M, L = 5.5 m and N = 3

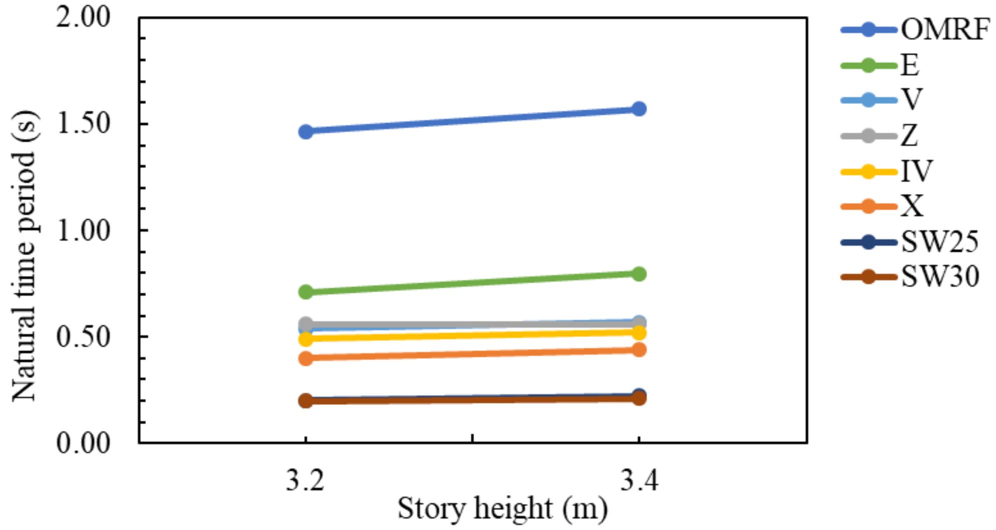

Figure A.1.83: The natural time period of the frames versus the story height change for different types of bracings and shear walls

Assuming fixed parameters S= M, L = 6.5 m and N = 5

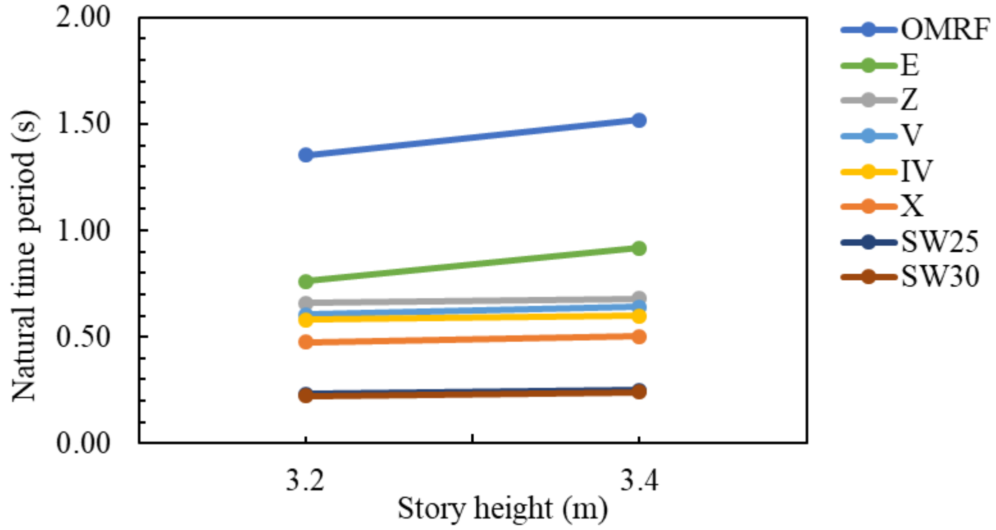

Figure A.1.84: The natural time period of the frames versus the story height change for different types of bracings and shear walls

Assuming fixed parameters S= H, L = 4.5 m and N = 1

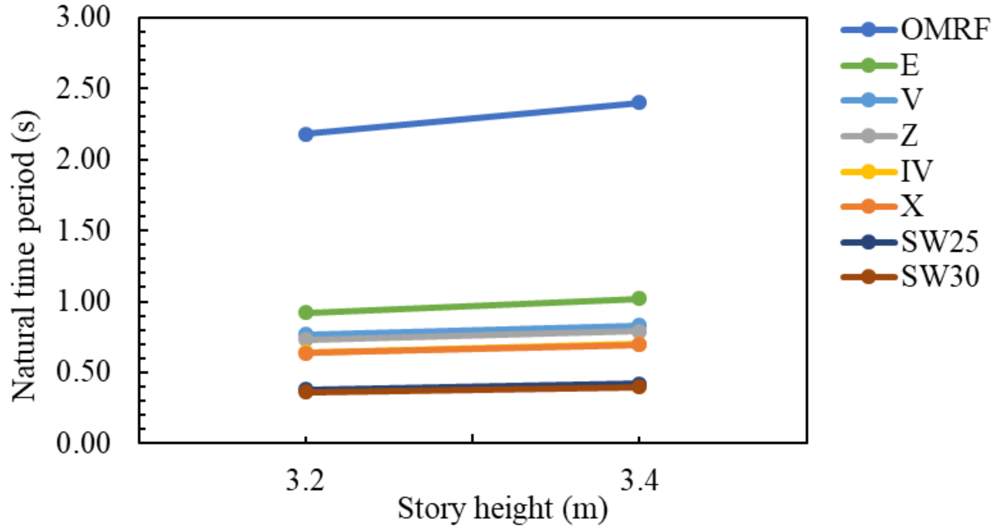

Figure A.1.85: The natural time period of the frames versus the story height change for different types of bracings and shear walls

Assuming fixed parameters S= H, L = 5.5 m and N = 3

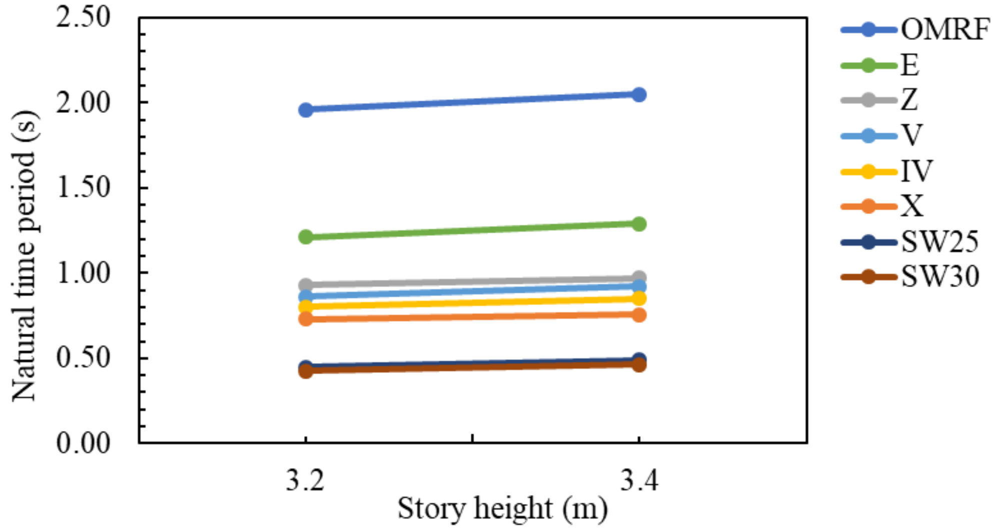

Figure A.1.86: The natural time period of the frames versus the story height change for different types of bracings and shear walls

Assuming fixed parameters S= H, L = 6.5 m and N = 5

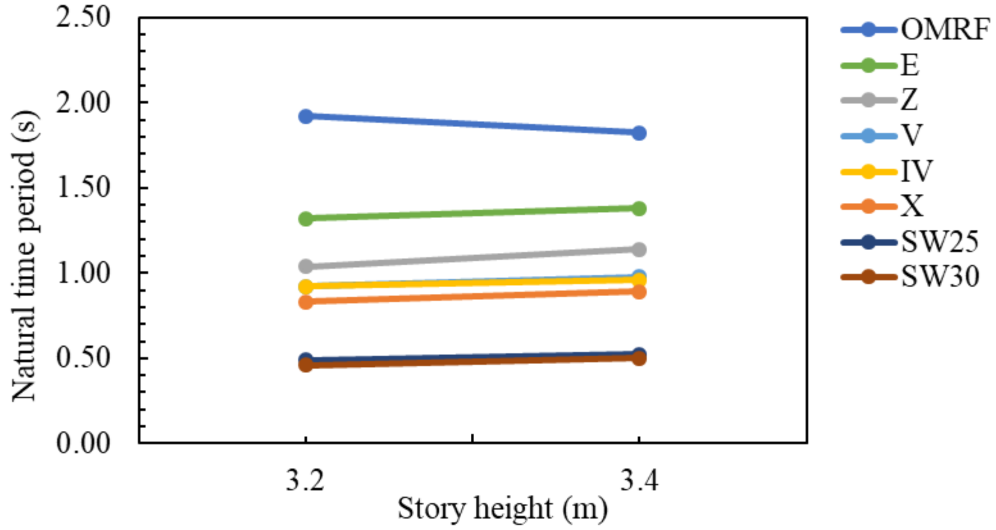

Figure A.1.87: The natural time period of the frames versus the story height change for different types of bracings and shear walls

A.1.3 Maximum Base Shear (Maximum Capacity)

A.1.3.1 The effect of span length on the maximum base shear strength of the steel frames for different types of bracing and shear walls

The parameters fixed for this figure are N = 3, S = L, H = 3.2 m

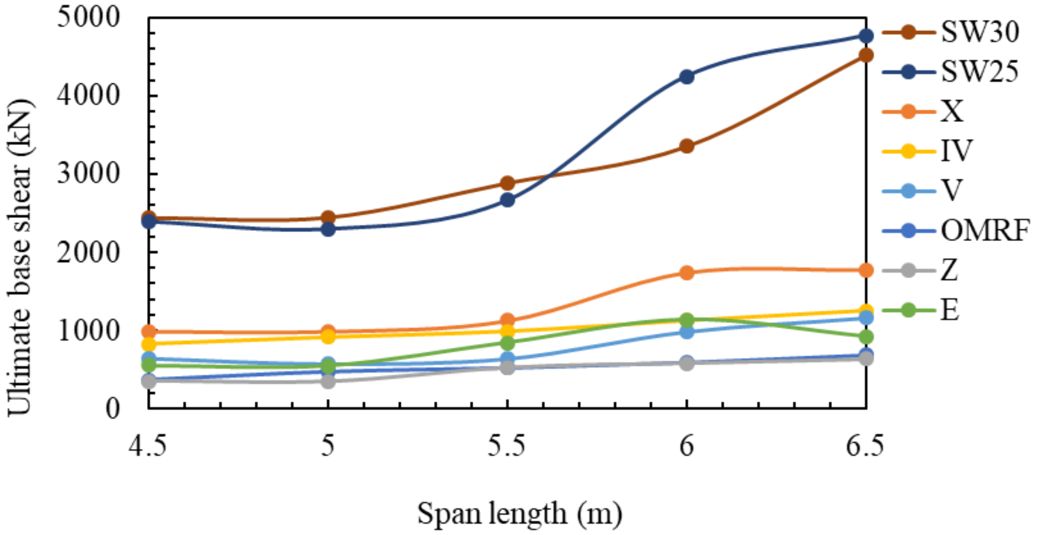

Figure A.1.88: The maximum base shear strength of the frames versus span length for different types of bracings and shear wall

The parameters fixed for this figure are N = 5, S = L, H = 3.2 m

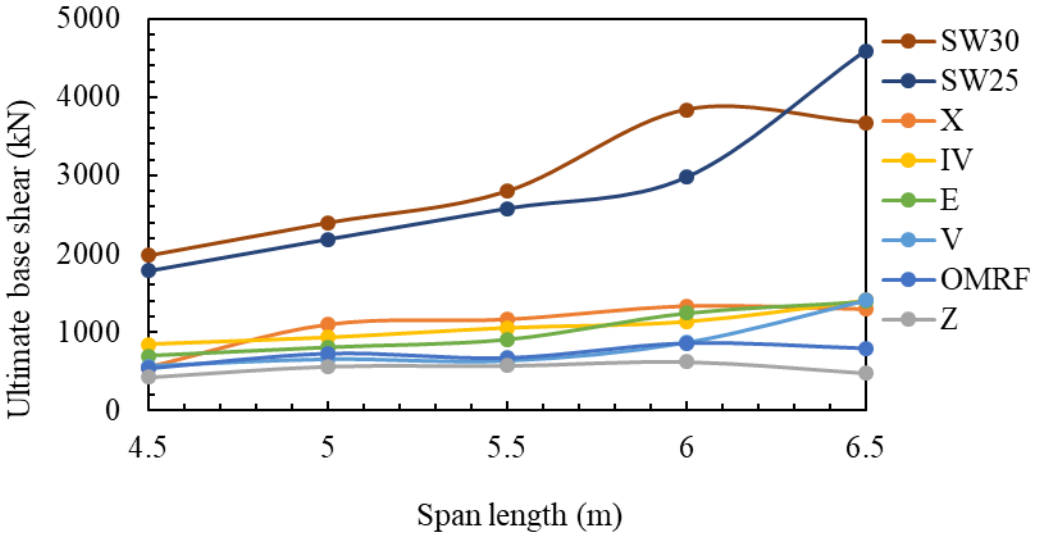

Figure A.1.89: The maximum base shear strength of the frames versus span length for different types of bracings and shear walls

The parameters fixed for this figure are N = 1, S = M, H = 3.2 m

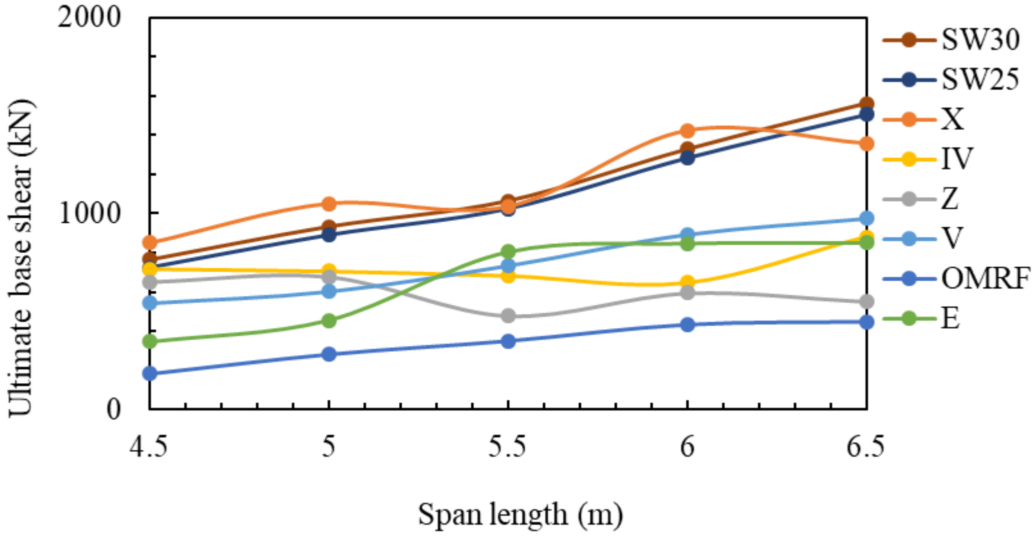

Figure A.1.90: The maximum base shear strength of the frames versus span length for different types of bracings and shear walls

The parameters fixed for this figure are N = 3, S = M, H = 3.2 m

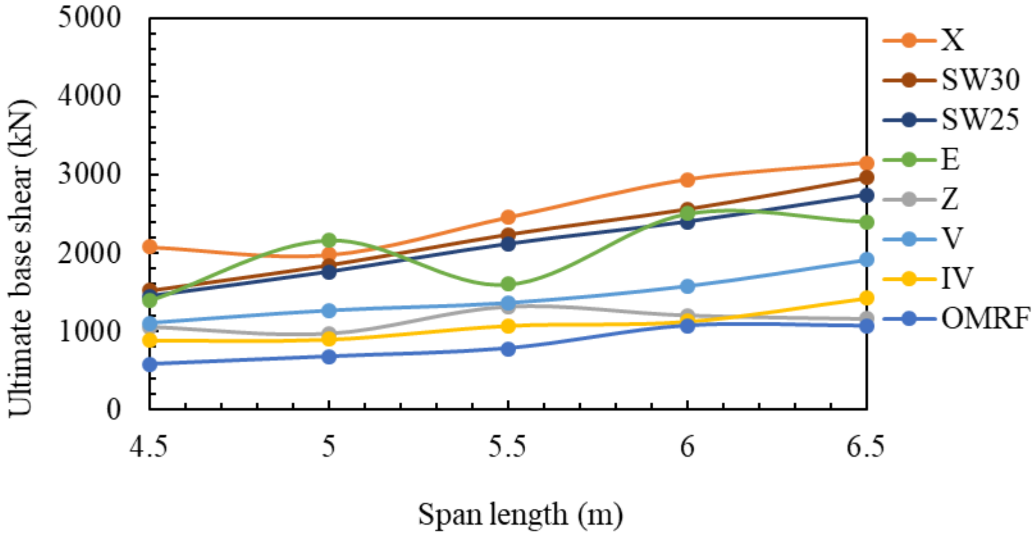

Figure A.1.91: The maximum base shear strength of the frames versus span length for different types of bracings and shear walls

The parameters fixed for this figure are N = 5, S = M, H = 3.2 m

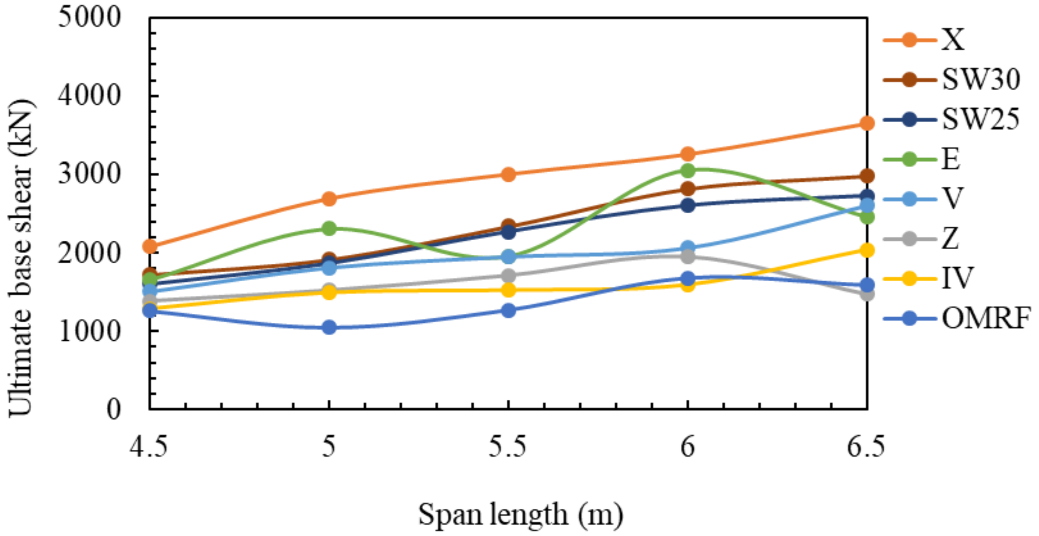

Figure A.1.92: The maximum base shear strength of the frames versus span length for different types of bracings and shear walls

The parameters fixed for this figure are N = 1, S = H, H = 3.2 m

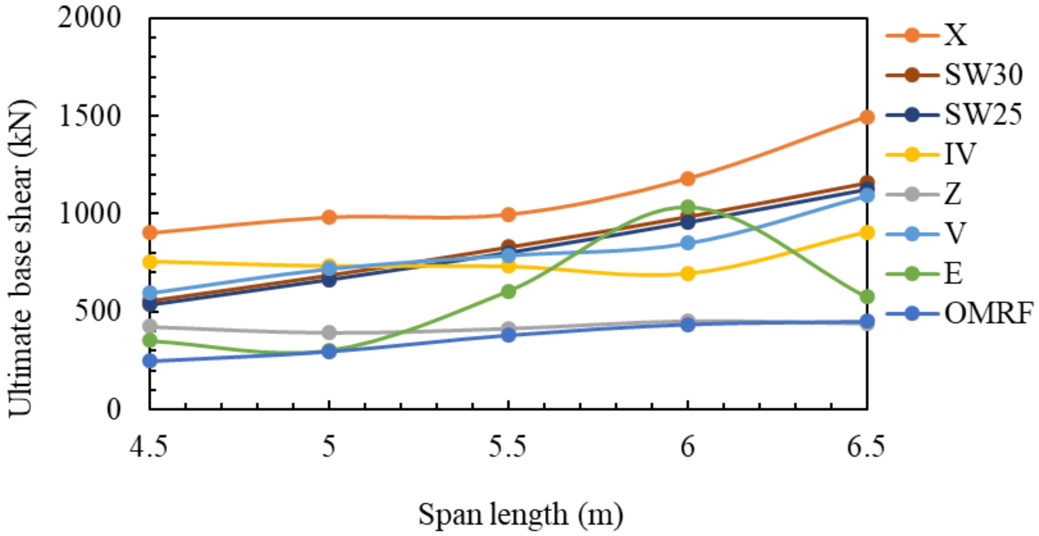

Figure A.1.93: The maximum base shear strength of the frames versus span length for different types of bracings and shear walls

The parameters fixed for this figure are N = 3, S = H, H = 3.2 m

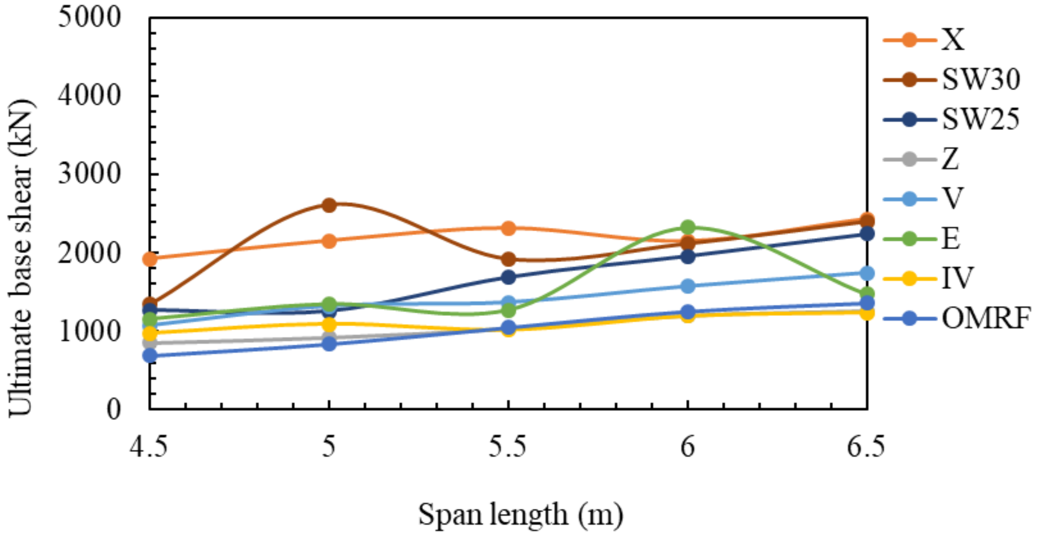

Figure A.1.94: The maximum base shear strength of the frames versus span length for different types of bracings and shear walls

The parameters fixed for this figure are N = 5, S = H, H = 3.2 m

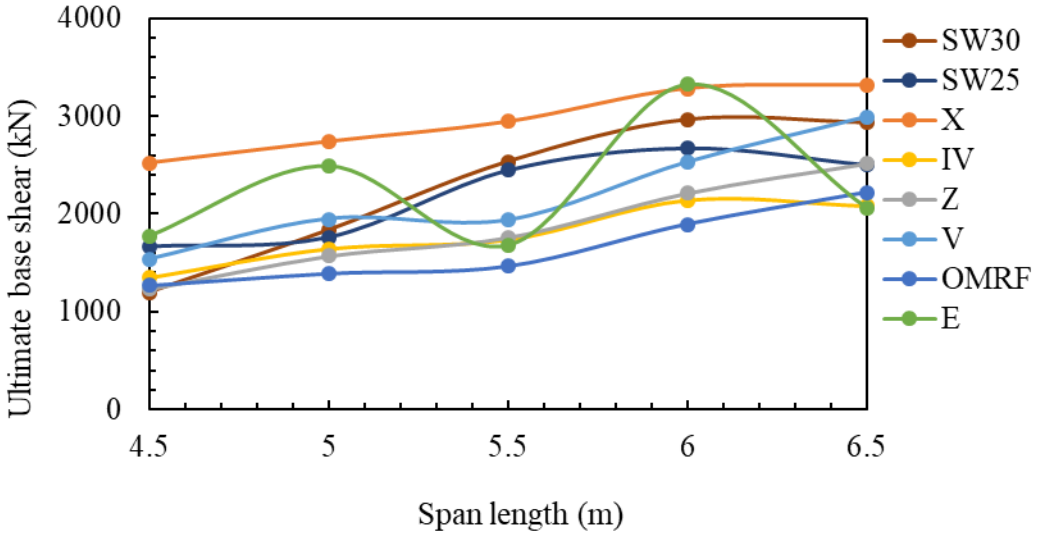

Figure A.1.95: The maximum base shear strength of the frames versus span length for different types of bracings and shear walls

A.1.3.2 The effect of number of stories on the maximum base shear of the steel frames for different types of bracings and shear walls

Assuming fixed parameters N = 3, L = 4.5 m and H =3.2 m

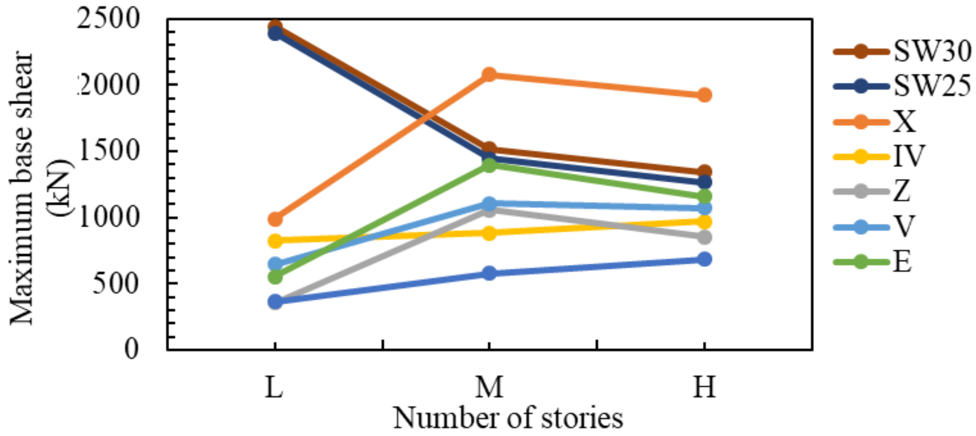

Figure A.1.96: The maximum base shear of steel frames versus number of stories for different types of bracings and shear walls

Assuming fixed parameters N = 5, L = 4.5 m and H = 3.2 m

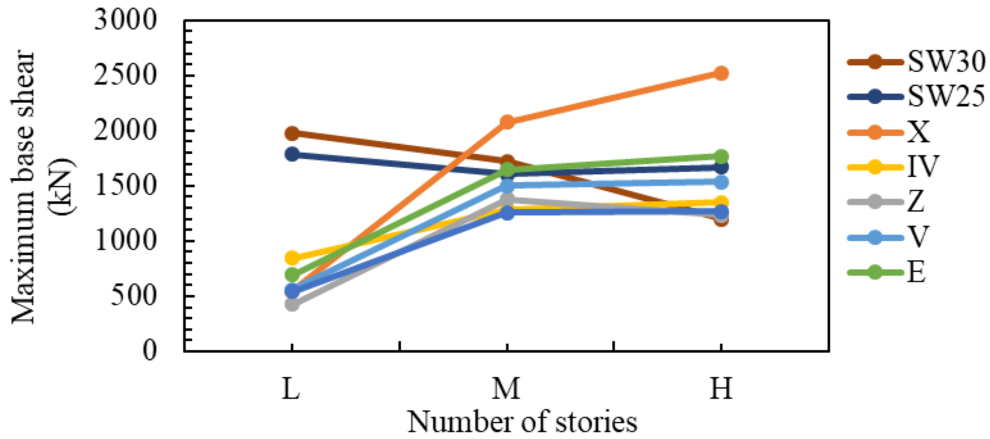

Figure A.1.97: The maximum base shear of steel frames versus number of stories for different types of bracings and shear walls

4.3.3 The effect of number of spans on the maximum base shear of the steel frames for different types of bracings and shear walls

Assuming fixed parameters S = M, L = 4.5 m and H = 3.2 m

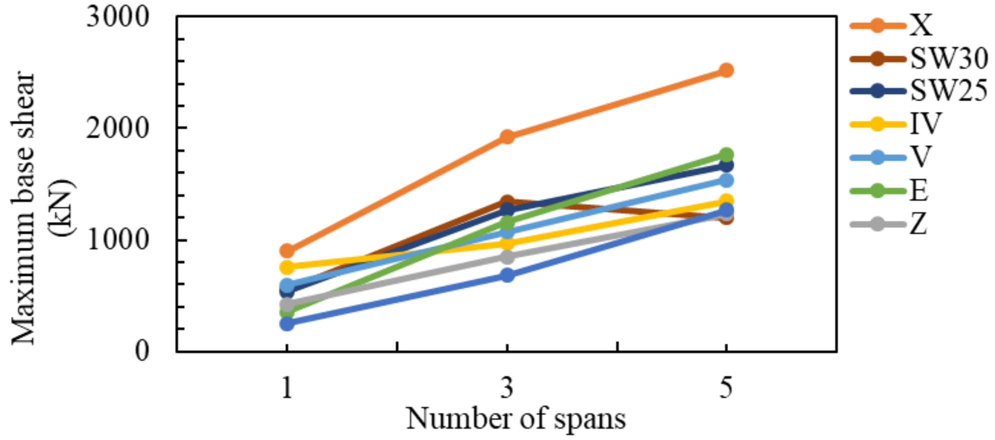

Figure A.1.98: The maximum base shear of steel frames versus number of spans for different types of bracings and shear walls

Assuming fixed parameters S = H, L = 4.5 m and H = 3.2 m

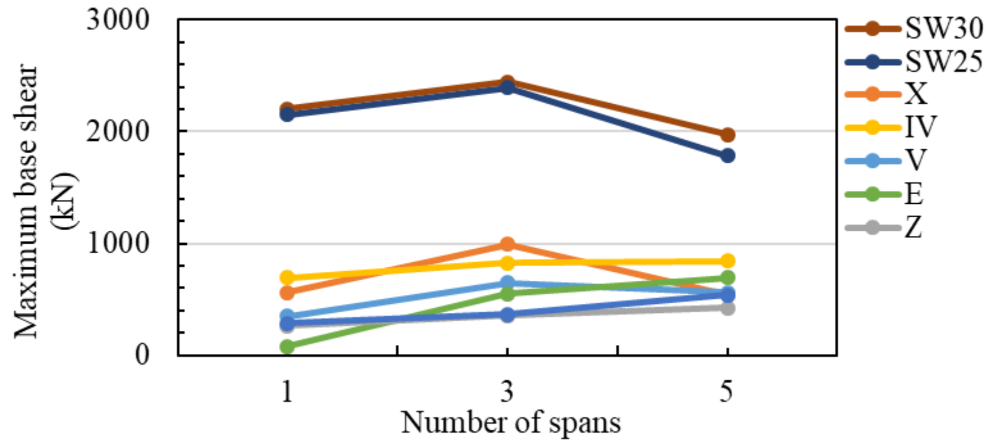

Figure A.1.99: The maximum base shear of steel frames versus number of spans for different types of bracings and shear walls

4.3.4 The effect of story height change on the maximum base shear of the steel frames for different types of bracings and shear walls

Assuming fixed parameters S= M, L = 4.5 m and N = 1

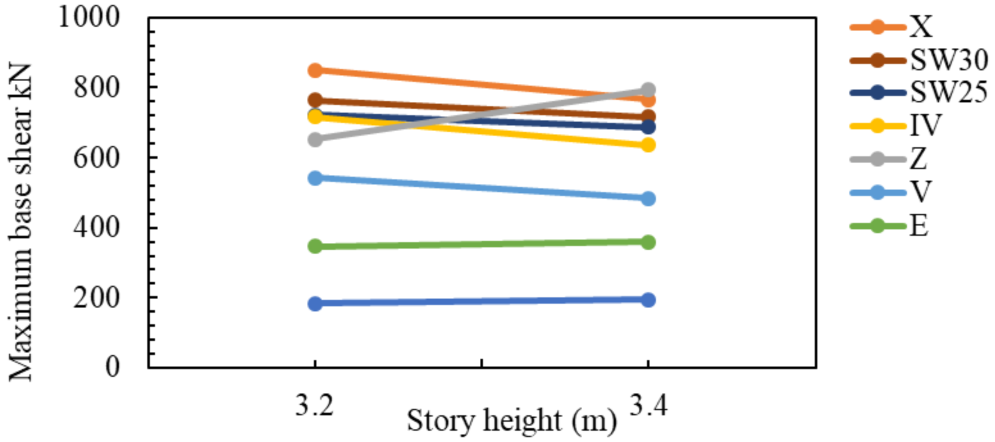

Figure A.1.100: The maximum base shear of steel frames versus story height change for different types of bracings and shear walls

Assuming fixed parameters for the figure S= L, L = 4.5 m and N = 1

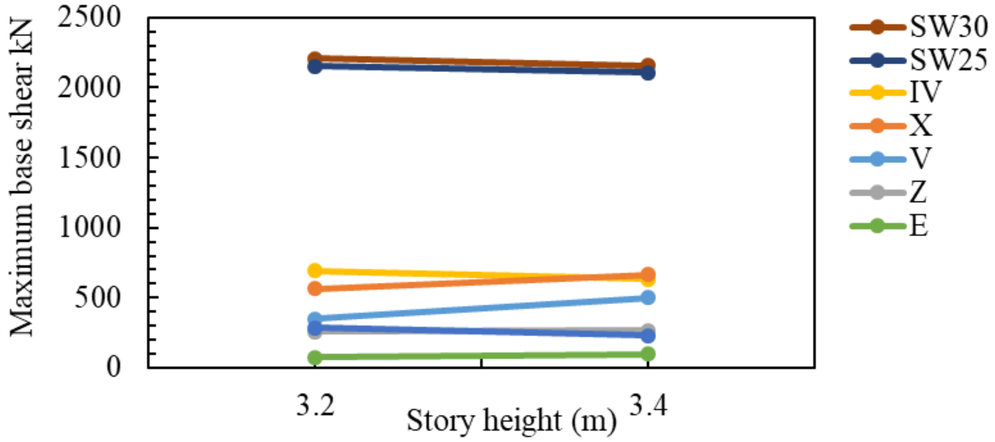

Figure A.1.101: The maximum base shear of steel frames versus story height change for different types of bracings and shear walls

APPENDIX 2

In this appendix, the results of all models are listed in tables and these results are used to draw graphs in chapter 4. For better understanding of the results of each model in the listed tables, all the symbols are defined below:

S defines the type of structure which is classified into low (1 storey), medium (5 storeys) and high (8 storeys)
P symbol means moment resisting frame
B means type of lateral load resisting systems
N is the number of spans
L is the length of span
H is the height of storey

Each table is dedicated to one type of LLRS and other parameters are changed. Each type of LLRS has 90 models considering different parameters which have been defined above.

Table A.2.1: Results of the models of ordinary moment resisting frames

SPN, L, H	K (kN/mm)	Time Period(s)	Vu (kN)
LP1,4.5,3.2	4.509	0.328	285
LP1,5,3.2	5.921	0.302	346
LP1,5.5,3.2	6.410	0.305	383
LP1,6,3.2	6.807	0.309	476
LP1,6.5,3.2	8.639	0.286	519
LP3,4.5,3.2	6.723	0.465	364
LP3,5,3.2	8.148	0.446	471
LP3,5.5,3.2	12.426	0.378	520
LP3,6,3.2	9.971	0.442	591
LP3,6.5,3.2	11.330	0.432	687
LP5,4.5,3.2	6.798	0.596	540

LP5,5,3.2	10.070	0.517	728
LP5,5.5,3.2	15.432	0.438	673
LP5,6,3.2	17.541	0.429	858
LP5,6.5,3.2	18.681	0.433	793
MP1,4.5,3.2	0.694	1.556	183
MP1,5,3.2	1.026	1.287	282
MP1,5.5,3.2	1.299	1.214	350
MP1,6,3.2	1.634	1.140	433
MP1,6.5,3.2	1.553	1.224	448
MP3,4.5,3.2	1.776	1.607	576
MP3,5,3.2	2.213	1.490	674
MP3,5.5,3.2	2.603	1.464	779
MP3,6,3.2	3.392	1.340	1071
MP3,6.5,3.2	3.981	1.285	1068
MP5,4.5,3.2	3.021	1.560	1256
MP5,5,3.2	3.495	1.620	1045
MP5,5.5,3.2	3.881	1.520	1268
MP5,6,3.2	4.753	1.438	1675
MP5,6.5,3.2	6.037	1.354	1586
HP1,4.5,3.2	0.507	2.181	249
HP1,5,3.2	0.642	2.040	298
HP1,5.5,3.2	0.846	1.870	381
HP1,6,3.2	1.052	1.780	436
HP1,6.5,3.2	1.006	1.902	451
HP3,4.5,3.2	1.444	2.253	684
HP3,5,3.2	1.915	2.083	834
HP3,5.5,3.2	2.465	1.960	1046
HP3,6,3.2	2.770	2.858	1253
HP3,6.5,3.2	2.976	1.870	1360
HP5,4.5,3.2	2.147	2.350	1268
HP5,5,3.2	2.801	2.170	1390

HP5,5.5,3.2	3.178	2.150	1465
HP5,6,3.2	3.972	2.000	1893
HP5,6.5,3.2	4.708	1.920	2216
LP1,4.5,3.4	4.518	0.328	233
LP1,5,3.4	4.848	0.330	284.8
LP1,5.5,3.4	5.473	0.333	361.6
LP1,6,3.4	5.850	0.334	446.9
LP1,6.5,3.4	6.469	0.331	385.8
LP3,4.5,3.4	4.553	0.565	340.57
LP3,5,3.4	5.575	0.484	428.44
LP3,5.5,3.4	8.124	0.470	528
LP3,6,3.4	7.479	0.515	465
LP3,6.5,3.4	9.306	0.478	655
LP5,4.5,3.4	6.427	0.614	533.75
LP5,5,3.4	8.320	0.570	568.5
LP5,5.5,3.4	9.270	0.570	672.9
LP5,6,3.4	9.859	0.570	662.53
LP5,6.5,3.4	12.293	0.537	878.68
MP1,4.5,3.4	0.617	1.620	195.3
MP1,5,3.4	0.922	1.360	272.1
MP1,5.5,3.4	1.012	1.410	315.3
MP1,6,3.4	1.419	1.225	404.87
MP1,6.5,3.4	1.350	1.314	418
MP3,4.5,3.4	1.588	1.660	545
MP3,5,3.4	2.167	1.532	635.75
MP3,5.5,3.4	2.222	1.571	779.2
MP3,6,3.4	2.935	1.453	999.3
MP3,6.5,3.4	2.986	1.475	984
MP5,4.5,3.4	2.490	1.705	1249
MP5,5,3.4	2.969	1.651	1005
MP5,5.5,3.4	3.935	1.520	1278.5

MP5,6,3.4	4.422	1.510	1588.4
MP5,6.5,3.4	4.674	1.520	1593.76
HP1,4.5,3.4	0.426	2.400	234.26
HP1,5,3.4	0.590	2.131	290.1
HP1,5.5,3.4	7.404	2.000	355
HP1,6,3.4	0.909	1.920	406.3
HP1,6.5,3.4	0.855	2.070	421.72
HP3,4.5,3.4	1.270	2.415	645
HP3,5,3.4	1.676	2.230	787.4
HP3,5.5,3.4	2.069	2.051	988.369
HP3,6,3.4	2.435	1.980	1222.77
HP3,6.5,3.4	3.166	1.819	1361.94
HP5,4.5,3.4	2.822	2.034	1532
HP5,5,3.4	3.409	1.960	1654.99
HP5,5.5,3.4	3.941	1.920	1835
HP5,6,3.4	4.360	1.910	1997.323
HP5,6.5,3.4	5.096	1.823	1704

Table A.2.2: Results of the models of X type bracing

SXN, L, H	K (kN/mm)	Time Period(s)	Vu (kN)
LX1,4.5,3.2	107.401	0.067	563
LX1,5,3.2	111.417	0.7	1158
LX1,5.5,3.2	130.597	0.067	1131
LX1,6,3.2	138.043	0.069	1529
LX1,6.5,3.2	156.603	0.071	1588
LX3,4.5,3.2	103.286	0.115	989
LX3,5,3.2	103.781	0.12	987.69
LX3,5.5,3.2	119.098	0.117	1125
LX3,6,3.2	121.166	0.121	1740

LX3,6.5,3.2	119.718	0.126	1775.55
LX5,4.5,3.2	82.823	0.16	546.2
LX5,5,3.2	99.302	0.154	1092
LX5,5.5,3.2	100.270	0.159	1159
LX5,6,3.2	101.760	0.165	1328
LX5,6.5,3.2	103.053	0.174	1290
MX1,4.5,3.2	12.292	0.326	849.46
MX1,5,3.2	16.044	0.304	1051
MX1,5.5,3.2	19.882	0.289	1036
MX1,6,3.2	23.797	0.27	1423
MX1,6.5,3.2	28.534	0.269	1358
MX3,4.5,3.2	22.704	0.416	2078
MX3,5,3.2	25.806	0.41	1977.4
MX3,5.5,3.2	31.287	0.4	2453
MX3,6,3.2	36.682	0.39	2936
MX3,6.5,3.2	40.738	0.39	3149
MX5,4.5,3.2	23.960	0.52	2076
MX5,5,3.2	29.408	0.5	2693
MX5,5.5,3.2	34.436	0.49	3006
MX5,6,3.2	38.666	0.486	3261
MX5,6.5,3.2	44.520	0.477	3653.32
HX1,4.5,3.2	4.725	0.636	902
HX1,5,3.2	6.133	0.6	982.6
HX1,5.5,3.2	7.831	0.555	996.3
HX1,6,3.2	9.520	0.533	1179.5
HX1,6.5,3.2	11.237	0.518	1496.1
HX3,4.5,3.2	9.562	0.79	1922.35
HX3,5,3.2	11.589	0.765	2152.2
HX3,5.5,3.2	14.128	0.73	2313
HX3,6,3.2	16.183	0.721	2146.4
HX3,6.5,3.2	18.350	0.714	2429.1

HX5,4.5,3.2	11.632	0.926	2521.13
HX5,5,3.2	13.561	0.915	2740.1
HX5,5.5,3.2	15.897	0.887	2943
HX5,6,3.2	20.233	0.839	3283
HX5,6.5,3.2	22.577	0.832	3318
LX1,4.5,3.4	99.561	0.07	664.4
LX1,5,3.4	113.977	0.069	1186.25
LX1,5.5,3.4	123.167	0.07	991
LX1,6,3.4	130.554	0.071	1466
LX1,6.5,3.4	145.958	0.074	1421.5
LX3,4.5,3.4	95.299	0.12	851.94
LX3,5,3.4	111.924	0.116	888
LX3,5.5,3.4	109.578	0.123	1010
LX3,6,3.4	114.302	0.125	1693
LX3,6.5,3.4	115.764	0.129	1691
LX5,4.5,3.4	84.082	0.16	863.2
LX5,5,3.4	93.777	0.159	921
LX5,5.5,3.4	94.698	0.165	989
LX5,6,3.4	99.885	0.168	1132.5
LX5,6.5,3.4	115.385	0.163	1233
MX1,4.5,3.4	10.808	0.345	766.25
MX1,5,3.4	13.651	0.328	870
MX1,5.5,3.4	16.750	0.32	1204
MX1,6,3.4	20.595	0.3	1194
MX1,6.5,3.4	25.239	0.285	1294.9
MX3,4.5,3.4	19.412	0.45	1584.3
MX3,5,3.4	22.944	0.44	1676.9
MX3,5.5,3.4	25.079	0.44	2098
MX3,6,3.4	32.371	0.41	2681.7
MX3,6.5,3.4	37.693	0.403	3065.1
MX5,4.5,3.4	21.199	0.553	2121.8

MX5,5,3.4	25.885	0.53	2525
MX5,5.5,3.4	28.056	0.54	2129
MX5,6,3.4	35.497	0.51	2689
MX5,6.5,3.4	39.748	0.502	3665
HX1,4.5,3.4	3.921	0.697	655
HX1,5,3.4	5.072	0.654	829
HX1,5.5,3.4	6.607	0.604	975
HX1,6,3.4	8.165	0.573	1111
HX1,6.5,3.4	9.996	0.547	1485.4
HX3,4.5,3.4	8.198	0.849	1788.6
HX3,5,3.4	10.147	0.825	1972
HX3,5.5,3.4	12.373	0.755	2458
HX3,6,3.4	14.625	0.755	2355
HX3,6.5,3.4	17.482	0.723	2887
HX5,4.5,3.4	10.007	0.993	2154.4
HX5,5,3.4	11.847	0.975	2511
HX5,5.5,3.4	14.321	0.931	2886.6
HX5,6,3.4	17.876	0.881	3566.6
HX5,6.5,3.4	19.169	0.89	3737.1

Table A.2.3: Results of the models of Z type (diagonal) bracing

SZN, L, H	K (kN/mm)	Time Period(s)	Vu (kN)
LZ1,4.5,3.2	60.304	0.097	260
LZ1,5,3.2	63.656	0.100	359.2
LZ1,5.5,3.2	76.081	0.096	412
LZ1,6,3.2	78.334	0.098	472.5
LZ1,6.5,3.2	81.111	0.110	658
LZ3,4.5,3.2	68.864	0.162	354.2
LZ3,5,3.2	71.565	0.170	343

LZ3,5.5,3.2	76.733	0.165	525
LZ3,6,3.2	78.918	0.170	582.5
LZ3,6.5,3.2	79.189	0.180	633
LZ5,4.5,3.2	57.344	0.220	424
LZ5,5,3.2	66.922	0.211	561.2
LZ5,5.5,3.2	68.856	0.220	570
LZ5,6,3.2	71.366	0.230	620.3
LZ5,6.5,3.2	72.031	0.240	477.5
MZ1,4.5,3.2	9.408	0.400	652.2
MZ1,5,3.2	11.364	0.380	678.8
MZ1,5.5,3.2	13.536	0.375	478
MZ1,6,3.2	15.388	0.370	594
MZ1,6.5,3.2	17.628	0.360	551
MZ3,4.5,3.2	15.706	0.540	1055.2
MZ3,5,3.2	17.919	0.540	967
MZ3,5.5,3.2	18.530	0.560	1315.7
MZ3,6,3.2	22.687	0.538	1202
MZ3,6.5,3.2	24.991	0.538	1157
MZ5,4.5,3.2	16.803	0.665	1381
MZ5,5,3.2	19.891	0.650	1520
MZ5,5.5,3.2	22.493	0.654	1713
MZ5,6,3.2	24.488	0.660	1959
MZ5,6.5,3.2	27.073	0.660	1468
HZ1,4.5,3.2	4.002	0.733	424
HZ1,5,3.2	5.000	0.706	393
HZ1,5.5,3.2	6.154	0.674	414.5
HZ1,6,3.2	7.265	0.661	454
HZ1,6.5,3.2	8.267	0.660	441.6
HZ3,4.5,3.2	7.744	0.970	850.7
HZ3,5,3.2	9.180	0.950	918.5
HZ3,5.5,3.2	8.061	0.930	1029

HZ3,6,3.2	12.061	0.930	1196
HZ3,6.5,3.2	13.370	0.940	1254
HZ5,4.5,3.2	9.273	1.130	1232
HZ5,5,3.2	10.762	1.120	1570
HZ5,5.5,3.2	12.228	1.110	1759
HZ5,6,3.2	15.767	1.020	2213
HZ5,6.5,3.2	17.578	1.040	2515
LZ1,4.5,3.4	56.604	0.100	266
LZ1,5,3.4	64.178	0.100	332
LZ1,5.5,3.4	71.564	0.100	374
LZ1,6,3.4	73.957	0.100	455
LZ1,6.5,3.4	74.650	0.104	487.5
LZ3,4.5,3.4	64.759	0.170	333
LZ3,5,3.4	71.035	0.165	458
LZ3,5.5,3.4	0.727	0.174	437
LZ3,6,3.4	74.621	0.180	563.7
LZ3,6.5,3.4	77.183	0.180	658
LZ5,4.5,3.4	58.759	0.220	421
LZ5,5,3.4	63.432	0.220	513
LZ5,5.5,3.4	65.579	0.219	525.3
LZ5,6,3.4	71.478	0.230	697
LZ5,6.5,3.4	81.672	0.220	694
MZ1,4.5,3.4	8.431	0.410	794
MZ1,5,3.4	10.123	0.400	563
MZ1,5.5,3.4	12.204	0.390	595
MZ1,6,3.4	13.897	0.390	527
MZ1,6.5,3.4	16.270	0.380	520
MZ3,4.5,3.4	14.707	0.550	1086
MZ3,5,3.4	16.316	0.560	796
MZ3,5.5,3.4	18.148	0.560	1126
MZ3,6,3.4	20.939	0.562	1092

MZ3,6.5,3.4	24.043	0.548	1136
MZ5,4.5,3.4	15.260	0.700	1282
MZ5,5,3.4	17.989	0.690	1382
MZ5,5.5,3.4	20.500	0.690	1347
MZ5,6,3.4	23.013	0.680	1936
MZ5,6.5,3.4	25.235	0.680	1840
HZ1,4.5,3.4	3.427	0.790	408
HZ1,5,3.4	4.272	0.760	352
HZ1,5.5,3.4	5.359	0.720	384
HZ1,6,3.4	6.450	0.700	433
HZ1,6.5,3.4	7.575	0.684	461
HZ3,4.5,3.4	6.868	1.020	775.5
HZ3,5,3.4	8.348	1.000	823
HZ,5.5,3.4	9.782	0.970	932.4
HZ3,6,3.4	11.344	0.970	1010
HZ3,6.5,3.4	12.659	0.940	1445.7
HZ5,4.5,3.4	8.246	1.200	1102
HZ5,5,3.4	9.655	1.200	1349
HZ5,5.5,3.4	12.843	1.065	2319
HZ5,6,3.4	13.069	1.120	2024
HZ5,6.5,3.4	13.796	1.140	2236

Table A.2.4: Results of the models of IV type bracing

SIVN, L, H	K (kN/mm)	Time Period(s)	Vu (kN)
LIV1,4.5,3.2	84.524	0.076	692
LIV1,5,3.2	89.136	0.078	704
LIV1,5.5,3.2	96.623	0.078	685.6
LIV1,6,3.2	102.953	0.079	637
LIV1,6.5,3.2	122.631	0.076	853

LIV3,4.5,3.2	81.889	0.130	826
LIV3,5,3.2	90.106	0.130	912.7
LIV3,5.5,3.2	100.814	0.130	988
LIV3,6,3.2	103.703	0.130	1126
LIV3,6.5,3.2	124.416	0.125	1252.7
LIV5,4.5,3.2	75.680	0.171	843.6
LIV5,5,3.2	79.654	0.174	931
LIV5,5.5,3.2	86.292	0.175	1054
LIV5,6,3.2	93.663	0.176	1133
LIV5,6.5,3.2	111.264	0.167	1405
MIV1,4.5,3.2	11.400	0.350	717.2
MIV1,5,3.2	13.522	0.340	706
MIV1,5.5,3.2	15.543	0.336	683.1
MIV1,6,3.2	18.176	0.330	648.4
MIV1,6.5,3.2	22.365	0.310	881
MIV3,4.5,3.2	17.875	0.500	884
MIV3,5,3.2	20.820	0.490	896
MIV3,5.5,3.2	23.937	0.490	1072
MIV3,6,3.2	26.482	0.490	1126
MIV3,6.5,3.2	31.897	0.464	1425
MIV5,4.5,3.2	19.619	0.615	1284
MIV5,5,3.2	22.699	0.606	1490
MIV5,5.5,3.2	25.767	0.600	1521
MIV5,6,3.2	28.518	0.600	1590
MIV5,6.5,3.2	34.793	0.580	2038.4
HIV1,4.5,3.2	4.669	0.642	757
HIV1,5,3.2	5.845	0.610	734
HIV1,5.5,3.2	7.166	0.590	732.3
HIV1,6,3.2	8.325	0.580	696
HIV1,6.5,3.2	10.228	0.544	905.3
HIV3,4.5,3.2	9.145	0.840	969

HIV3,5,3.2	11.072	0.810	1091
HIV3,5.5,3.2	12.941	0.803	1011
HIV3,6,3.2	14.938	0.790	1192
HIV3,6.5,3.2	17.082	0.770	1238
HIV5,4.5,3.2	10.567	1.000	1348
HIV5,5,3.2	12.899	0.970	1643
HIV5,5.5,3.2	15.116	0.940	1736
HIV5,6,3.2	17.712	0.910	2139.6
HIV5,6.5,3.2	19.422	0.920	2081
LIV1,4.5,3.4	76.163	0.080	631.6
LIV1,5,3.4	81.282	0.080	621
LIV1,5.5,3.4	88.561	0.082	608
LIV1,6,3.4	110.084	0.077	814
LIV1,6.5,3.4	114.526	0.078	773
LIV3,4.5,3.4	74.342	0.136	729
LIV3,5,3.4	82.260	0.136	825.6
LIV3,5.5,3.4	91.892	0.135	910
LIV3,6,3.4	106.442	0.130	1065
LIV3,6.5,3.4	116.104	0.130	1124
LIV5,4.5,3.4	67.723	0.181	786.2
LIV5,5,3.4	74.473	0.181	839
LIV5,5.5,3.4	83.412	0.179	935
LIV5,6,3.4	98.073	0.172	1212
LIV5,6.5,3.4	102.782	0.175	1244.5
MIV1,4.5,3.4	9.915	0.370	636
MIV1,5,3.4	11.824	0.362	626
MIV1,5.5,3.4	14.079	0.350	616
MIV1,6,3.4	17.369	0.330	798.3
MIV1,6.5,3.4	20.093	0.326	797.5
MIV3,4.5,3.4	15.556	0.540	766.9
MIV3,5,3.4	18.680	0.520	878

MIV3,5.5,3.4	21.218	0.520	912.6
MIV3,6,3.4	26.074	0.490	1134
MIV3,6.5,3.4	29.214	0.480	1178
MIV5,4.5,3.4	17.542	0.645	1227
MIV5,5,3.4	19.685	0.650	1002
MIV5,5.5,3.4	23.127	0.630	1393
MIV5,6,3.4	27.916	0.610	1594
MIV5,6.5,3.4	31.443	0.600	2024
HIV1,4.5,3.4	3.974	0.700	647.9
HIV1,5,3.4	5.007	0.660	649
HIV1,5.5,3.4	6.179	0.630	650
HIV1,6,3.4	7.789	0.600	738
HIV1,6.5,3.4	9.032	0.580	818.6
HIV3,4.5,3.4	7.857	0.902	852
HIV3,5,3.4	9.619	0.867	999
HIV,5.5,3.4	11.420	0.850	950
HIV3,6,3.4	14.333	0.800	1206
HIV3,6.5,3.4	15.268	0.810	1028
HIV5,4.5,3.4	9.283	1.100	1263
HIV5,5,3.4	11.068	1.050	1425.8
HIV5,5.5,3.4	12.795	1.020	1406.6
HIV5,6,3.4	15.742	0.980	1672.6
HIV5,6.5,3.4	17.787	0.960	2147

Table A.2.5: Results of the models of V type bracing

SVN, L, H	K (kN/mm)	Time Period(s)	Vu (kN)
LV1,4.5,3.2	40.987	0.12	351.2
LV1,5,3.2	45.676	0.11	582
LV1,5.5,3.2	52.734	0.11	717

LV1,6,3.2	58.552	0.1	840
LV1,6.5,3.2	66.132	0.1	804.4
LV3,4.5,3.2	38.536	0.19	646
LV3,5,3.2	53.568	0.17	576
LV3,5.5,3.2	62.671	0.164	641
LV3,6,3.2	68.537	0.16	982
LV3,6.5,3.2	74.688	0.16	1156.9
LV5,4.5,3.2	45.068	0.222	562.6
LV5,5,3.2	51.331	0.218	650.3
LV5,5.5,3.2	56.665	0.217	629
LV5,6,3.2	74.603	0.2	864
LV5,6.5,3.2	85.353	0.19	1408.7
MV1,4.5,3.2	7.187	0.44	543.4
MV1,5,3.2	8.924	0.42	603
MV1,5.5,3.2	10.764	0.41	735
MV1,6,3.2	12.922	0.39	894.4
MV1,6.5,3.2	15.688	0.37	976.2
MV3,4.5,3.2	13.003	0.59	1107
MV3,5,3.2	15.928	0.56	1267
MV3,5.5,3.2	19.104	0.54	1363
MV3,6,3.2	22.105	0.53	1577.8
MV3,6.5,3.2	26.384	0.5	1911.4
MV5,4.5,3.2	15.282	0.69	1502
MV5,5,3.2	18.168	0.67	1809
MV5,5.5,3.2	21.651	0.65	1948.7
MV5,6,3.2	24.409	0.64	2061
MV5,6.5,3.2	29.554	0.604	2601
HV1,4.5,3.2	3.381	0.77	596.4
HV1,5,3.2	4.231	0.732	720
HV1,5.5,3.2	5.106	0.705	788.4
HV1,6,3.2	6.406	0.66	850

HV1,6.5,3.2	8.004	0.62	1091.6
HV3,4.5,3.2	7.266	0.934	1070
HV3,5,3.2	8.880	0.9	1319.4
HV3,5.5,3.2	10.982	0.86	1366.4
HV3,6,3.2	12.953	0.83	1571.9
HV3,6.5,3.2	14.762	0.814	1744
HV5,4.5,3.2	8.963	1.1	1534.9
HV5,5,3.2	10.913	1.05	1946.82
HV5,5.5,3.2	12.853	1.01	1934
HV5,6,3.2	16.030	0.957	2526.76
HV5,6.5,3.2	18.594	0.92	2988.6
LV1,4.5,3.4	35.940	0.12	500
LV1,5,3.4	42.514	0.122	445.1
LV1,5.5,3.4	44.467	0.115	568.4
LV1,6,3.4	52.489	0.11	558.87
LV1,6.5,3.4	79.590	0.1	884.51
LV3,4.5,3.4	41.819	0.183	484.8
LV3,5,3.4	47.080	0.183	492.2
LV3,5.5,3.4	55.580	0.175	513.4
LV3,6,3.4	69.213	0.166	578.72
LV3,6.5,3.4	75.420	0.165	637
LV5,4.5,3.4	40.170	0.238	495.53
LV5,5,3.4	47.332	0.23	532.75
LV5,5.5,3.4	54.051	0.22	531.94
LV5,6,3.4	68.438	0.21	585.1
LV5,6.5,3.4	74.271	0.207	981.4
MV1,4.5,3.4	6.173	0.48	485
MV1,5,3.4	7.856	0.45	605.5
MV1,5.5,3.4	9.778	0.43	745.92
MV1,6,3.4	11.689	0.41	794
MV1,6.5,3.4	13.986	0.39	919.2

MV3,4.5,3.4	11.383	0.625	1081
MV3,5,3.4	13.875	0.6	1126.1
MV3,5.5,3.4	16.960	0.57	1348.6
MV3,6,3.4	20.682	0.54	1546.4
MV3,6.5,3.4	23.342	0.535	1720.3
MV5,4.5,3.4	13.127	0.75	1336.4
MV5,5,3.4	16.096	0.71	1639
MV5,5.5,3.4	18.806	0.69	1752.5
MV5,6,3.4	22.595	0.66	2176.6
MV5,6.5,3.4	26.050	0.64	2445.7
HV1,4.5,3.4	2.904	0.83	548.6
HV1,5,3.4	3.619	0.79	675.83
HV1,5.5,3.4	4.578	0.74	759.7
HV1,6,3.4	5.770	0.7	1030
HV1,6.5,3.4	6.925	0.67	1012.3
HV3,4.5,3.4	6.272	1	1019.63
HV3,5,3.4	7.848	0.949	1427
HV,5.5,3.4	9.414	0.92	1192
HV3,6,3.4	12.033	0.86	1565.6
HV3,6.5,3.4	13.001	0.86	1562.1
HV5,4.5,3.4	7.730	1.17	1379.75
HV5,5,3.4	9.977	1.109	1872.5
HV5,5.5,3.4	11.163	1.08	1746
HV5,6,3.4	2.184	1.02	2337.69
HV5,6.5,3.4	16.274	0.98	2726

Table A.2.6: Results of the models of E (K-shaped) type bracing

SEN,L,H	K (kN/mm)	Time Period(s)	Vu (kN)
LE1,4.5,3.2	4.539	0.325	77.15
LE1,5,3.2	6.718	0.282	116.6
LE1,5.5,3.2	7.298	0.285	245.3
LE1,6,3.2	9.184	0.265	345
LE1,6.5,3.2	9.621	0.27	394
LE3,4.5,3.2	13.069	0.332	551.7
LE3,5,3.2	17.982	0.294	550
LE3,5.5,3.2	20.454	0.294	844.7
LE3,6,3.2	22.148	0.296	1137.52
LE3,6.5,3.2	33.061	0.251	924.2
LE5,4.5,3.2	13.867	0.413	695.2
LE5,5,3.2	18.374	0.379	803.2
LE5,5.5,3.2	20.520	0.378	903
LE5,6,3.2	22.804	0.375	1240.5
LE5,6.5,3.2	28.623	0.348	1389.1
ME1,4.5,3.2	3.742	0.619	346
ME1,5,3.2	4.550	0.61	456
ME1,5.5,3.2	4.581	0.651	804.48
ME1,6,3.2	4.562	0.94	848
ME1,6.5,3.2	5.165	0.66	8512
ME3,4.5,3.2	8.258	0.72	1393.5
ME3,5,3.2	9.475	0.719	2164.98
ME3,5.5,3.2	10.816	0.709	1602
ME3,6,3.2	11.199	0.733	2500
ME3,6.5,3.2	11.925	0.734	2400
ME5,4.5,3.2	11.415	0.781	1649
ME5,5,3.2	11.449	0.76	2304
ME5,5.5,3.2	12.872	0.841	1953

ME5,6,3.2	12.907	0.875	3054
ME5,6.5,3.2	15.075	0.762	2460
HE1,4.5,3.2	2.804	0.923	351.46
HE1,5,3.2	3.079	0.942	302.1
HE1,5.5,3.2	3.394	0.98	604.5
HE1,6,3.2	4.005	0.924	1035
HE1,6.5,3.2	4.234	0.93	576
HE3,4.5,3.2	5.368	1.143	1155.97
HE3,5,3.2	5.396	1.207	1347
HE3,5.5,3.2	6.247	1.21	1269.44
HE3,6,3.2	6.650	1.223	2322
HE3,6.5,3.2	7.588	1.194	1470.5
HE5,4.5,3.2	7.598	1.239	1769.6
HE5,5,3.2	8.093	1.26	2488.3
HE5,5.5,3.2	9.300	1.251	1674.1
HE5,6,3.2	9.629	1.285	3324
HE5,6.5,3.2	10.173	1.322	2061
LE1,4.5,3.4	4.123	0.342	99.5
LE1,5,3.4	5.908	0.301	96.9
LE1,5.5,3.4	5.020	0.343	241
LE1,6,3.4	8.136	0.282	324.6
LE1,6.5,3.4	8.268	0.291	320.54
LE3,4.5,3.4	13.463	0.327	534.76
LE3,5,3.4	15.315	0.316	512.5
LE3,5.5,3.4	17.146	0.321	783.89
LE3,6,3.4	21.157	0.303	1147.63
LE3,6.5,3.4	29.592	0.265	1097.9
LE5,4.5,3.4	12.057	0.442	650.2
LE5,5,3.4	16.295	0.403	774.76
LE5,5.5,3.4	16.774	0.42	758.88
LE5,6,3.4	21.432	0.387	1192.5

LE5,6.5,3.4	28.347	0.348	1197.6
ME1,4.5,3.4	3.198	0.667	360
ME1,5,3.4	3.686	0.654	407
ME1,5.5,3.4	4.862	0.612	911.6
ME1,6,3.4	5.059	0.98	810.82
ME1,6.5,3.4	5.270	0.651	481
ME3,4.5,3.4	8.408	0.704	1326
ME3,5,3.4	9.168	0.739	1534
ME3,5.5,3.4	8.127	0.799	2008
ME3,6,3.4	10.226	0.763	2443.8
ME3,6.5,3.4	10.422	0.785	1635.2
ME5,4.5,3.4	10.288	0.832	1686.1
ME5,5,3.4	9.670	0.902	1639
ME5,5.5,3.4	10.659	0.924	1674.23
ME5,6,3.4	11.993	0.904	3141.54
ME5,6.5,3.4	12.579	0.917	2422.45
HE1,4.5,3.4	2.347	1.02	249.6
HE1,5,3.4	2.628	1.03	388.57
HE1,5.5,3.4	2.369	1.127	731.9
HE1,6,3.4	3.513	0.99	954
HE1,6.5,3.4	3.447	1.02	579.3
HE3,4.5,3.4	4.855	1.213	1134
HE3,5,3.4	4.940	1.26	1228
HE,5.5,3.4	5.522	1.29	1113
HE3,6,3.4	5.954	1.3	2143
HE3,6.5,3.4	6.756	1.27	1164.5
HE5,4.5,3.4	6.525	1.346	2011
HE5,5,3.4	7.350	1.336	2345.3
HE5,5.5,3.4	8.335	1.326	1502
HE5,6,3.4	8.478	1.39	2945.3
HE5,6.5,3.4	9.337	1.38	1678.9

Table A.2.7: Results of the models of SW25 (shear wall with compressive strength of concrete 25 MPa)

SWN,L,H,F25	K(kN/mm)	Time Period(s)	Vu (kN)
LW1,4.5,3.2	1892.303	0.018	2152
LW1,5,3.2	2125.141	0.018	2657
LW1,5.5,3.2	2294.76	0.018	3220.9
LW1,6,3.2	2450.195	0.018	3834
LW1,6.5,3.2	2579.658	0.018	4501
LW3,4.5,3.2	732.738	0.037	2395
LW3,5,3.2	760.937	0.038	2300
LW3,5.5,3.2	802.1	0.038	2664
LW3,6,3.2	871.519	0.038	4250
LW3,6.5,3.2	872.425	0.039	4772
LW5,4.5,3.2	381.629	0.064	1784
LW5,5,3.2	399.235	0.065	2187
LW5,5.5,3.2	418.18	0.066	2580
LW5,6,3.2	451.658	0.067	2977
LW5,6.5,3.2	470.711	0.068	4591
MW1,4.5,3.2	82.147	0.16	723
MW1,5,3.2	109.029	0.147	890
MW1,5.5,3.2	110.17	0.156	1025
MW1,6,3.2	161.554	0.133	1285
MW1,6.5,3.2	204.302	0.124	1507
MW3,4.5,3.2	79.323	0.242	1446
MW3,5,3.2	104.125	0.223	1757
MW3,5.5,3.2	138.93	0.202	2113.52
MW3,6,3.2	170.285	0.192	2397
MW3,6.5,3.2	204.092	0.182	2741
MW5,4.5,3.2	78.798	0.294	1605
MW5,5,3.2	99.091	0.282	1868

MW5,5.5,3.2	129.431	0.255	2270
MW5,6,3.2	155.695	0.246	2600
MW5,6.5,3.2	182.453	0.233	2723
HW1,4.5,3.2	22.043	0.378	537
HW1,5,3.2	29.524	0.345	665
HW1,5.5,3.2	38.48	0.32	803
HW1,6,3.2	47.751	0.299	957
HW1,6.5,3.2	58.928	0.281	1125
HW3,4.5,3.2	25.532	0.523	1265
HW3,5,3.2	31.944	0.493	1253
HW3,5.5,3.2	42.6	0.447	1686
HW3,6,3.2	52.451	0.423	1958
HW3,6.5,3.2	64.61	0.398	2240
HW5,4.5,3.2	26.419	0.638	1667
HW5,5,3.2	35.501	0.586	1761
HW5,5.5,3.2	47.264	0.531	2446
HW5,6,3.2	56.418	0.5	2674
HW5,6.5,3.2	65.498	0.489	2503
LW1,4.5,3.4	1770.275	0.019	2106
LW1,5,3.4	2008.061	0.019	2596
LW1,5.5,3.4	2188.13	0.019	3146
LW1,6,3.4	2353.079	0.019	3746
LW1,6.5,3.4	2492.186	0.019	4397
LW3,4.5,3.4	650.999	0.04	2040
LW3,5,3.4	752.495	0.038	2292
LW3,5.5,3.4	794.53	0.039	3565
LW3,6,3.4	863.663	0.039	4222
LW3,6.5,3.4	870.087	0.04	4761
LW5,4.5,3.4	362.874	0.066	2137
LW5,5,3.4	395.866	0.066	2239
LW5,5.5,3.4	366.275	0.066	2851

LW5,6,3.4	449.01	0.068	3080
LW5,6.5,3.4	466.056	0.069	3536
MW1,4.5,3.4	69.872	0.175	688
MW1,5,3.4	93.092	0.16	852
MW1,5.5,3.4	109.625	0.156	1055.5
MW1,6,3.4	139.033	0.145	1206
MW1,6.5,3.4	181.567	0.132	1440
MW3,4.5,3.4	71.185	0.257	1481
MW3,5,3.4	94.121	0.234	1795
MW3,5.5,3.4	114.74	0.224	2094
MW3,6,3.4	143.529	0.21	2486
MW3,6.5,3.4	190.616	0.189	2832
MW5,4.5,3.4	67.848	0.319	1699
MW5,5,3.4	88.49	0.294	1959
MW5,5.5,3.4	113.613	0.274	2432
MW5,6,3.4	134.292	0.268	2734
ME5,6.5,3.4	160.361	0.25	2868
HW1,4.5,3.4	18.373	0.419	514
HW1,5,3.4	23.377	0.395	627
HW1,5.5,3.4	32.421	0.35	767.63
HW1,6,3.4	40.827	0.327	914
HW1,6.5,3.4	50.312	0.307	1074
HW3,4.5,3.4	21.206	0.574	1306
HW3,5,3.4	27.66	0.532	1554
HW3,5.5,3.4	36.337	0.491	1774
HW3,6,3.4	45.073	0.457	2001
HW3,6.5,3.4	55.015	0.433	2278
HW5,4.5,3.4	23.406	0.679	1729
HW5,5,3.4	30.549	0.63	1959
HW5,5.5,3.4	40.047	0.58	2497
HW5,6,3.4	48.225	0.548	2832

HW5,6.5,3.4	57.02	0.525	2861

Table A.2.8: Results of the models of SW30 (shear wall with compressive strength of concrete 30 MPa)

SWN,L,H,F30	K (kN/mm)	Time Period(s)	Vu (kN)
LW1,4.5,3.2	2071.39	0.017	2206
LW1,5,3.2	2326.591	0.017	2726
LW1,5.5,3.2	2512.52	0.017	3298
LW1,6,3.2	2682.858	0.017	3926
LW1,6.5,3.2	2824.643	0.017	4604
LW3,4.5,3.2	714.904	0.037	2443
LW3,5,3.2	766.285	0.036	2446
LW3,5.5,3.2	837.49	0.037	2885
LW3,6,3.2	889.645	0.037	3352
LW3,6.5,3.2	863.379	0.039	4517
LW5,4.5,3.2	385.236	0.062	1976
LW5,5,3.2	408.307	0.064	2394
LW5,5.5,3.2	427.37	0.065	2796
LW5,6,3.2	423.612	0.069	3838
LW5,6.5,3.2	451.259	0.068	3672
MW1,4.5,3.2	89.841	0.153	764
MW1,5,3.2	119.299	0.14	933
MW1,5.5,3.2	130.446	0.014	1064.3
MW1,6,3.2	189.166	0.123	1328
MW1,6.5,3.2	222.817	0.118	1562
MW3,4.5,3.2	90.589	0.225	1517
MW3,5,3.2	119.025	0.207	1844
MW3,5.5,3.2	145.0267	0.198	2235.98
MW3,6,3.2	180.554	0.186	2562

MW3,6.5,3.2	222.975	0.174	2961
MW5,4.5,3.2	85.369	0.282	1721
MW5,5,3.2	109.972	0.262	1914
MW5,5.5,3.2	136.92	0.251	2337
MW5,6,3.2	168.987	0.232	2816
MW5,6.5,3.2	201.081	0.223	2981
HW1,4.5,3.2	24.151	0.36	556
HW1,5,3.2	32.41	0.33	687
HW1,5.5,3.2	41.845	0.305	831
HW1,6,3.2	52.578	0.285	987
HW1,6.5,3.2	64.694	0.268	1160
HW3,4.5,3.2	26.411	0.512	1344
HW3,5,3.2	36.877	0.457	2613
HW3,5.5,3.2	47.33	0.424	1926
HW3,6,3.2	58.361	0.4	2119
HW3,6.5,3.2	70.7	0.38	2399
HW5,4.5,3.2	27.559	0.622	1197
HW5,5,3.2	37.732	0.557	1834
HW5,5.5,3.2	51.56	0.505	2533
HW5,6,3.2	62.498	0.478	2963
HW5,6.5,3.2	73.092	0.46	2932
LW1,4.5,3.4	1937.786	0.018	2157
LW1,5,3.4	2198.148	0.018	2661
LW1,5.5,3.4	23956.38	0.018	3223
LW1,6,3.4	2576.395	0.018	3840
LW1,6.5,3.4	2728.754	0.018	4502
LW3,4.5,3.4	689.647	0.039	4589
LW3,5,3.4	785.146	0.037	3038
LW3,5.5,3.4	825.57	0.038	3645
LW3,6,3.4	900.371	0.38	3514
LW3,6.5,3.4	900.716	0.039	4010

LW5,4.5,3.4	378.847	0.063	2020
LW5,5,3.4	408.326	0.064	2450
LW5,5.5,3.4	426.452	0.066	2851
LW5,6,3.4	423.046	0.068	3345
LW5,6.5,3.4	450.806	0.068	3825
MW1,4.5,3.4	76.414	0.167	717
MW1,5,3.4	101.854	0.153	894
MW1,5.5,3.4	130.625	0.142	1071.75
MW1,6,3.4	162.95	0.134	1271
MW1,6.5,3.4	198.918	0.126	1494
MW3,4.5,3.4	72.893	0.254	1603
MW3,5,3.4	93.46	0.237	1895
MW3,5.5,3.4	129.84	0.21	2201.7
MW3,6,3.4	156.508	0.201	2652
MW3,6.5,3.4	189.692	0.19	2995
MW5,4.5,3.4	70.559	0.318	1891
MW5,5,3.4	95.021	0.283	2078
MW5,5.5,3.4	122.083	0.264	2522
MW5,6,3.4	145.902	0.255	2829
ME5,6.5,3.4	174.296	0.242	3196
HW1,4.5,3.4	20.407	0.397	531
HW1,5,3.4	27.466	0.362	656
HW1,5.5,3.4	35.605	0.334	793
HW1,6,3.4	44.913	0.312	944
HW1,6.5,3.4	55.426	0.293	1109
HW3,4.5,3.4	23.126	0.55	1394
HW3,5,3.4	31.348	0.503	1657
HW,5.5,3.4	40.158	0.465	1973
HW3,6,3.4	49.942	0.432	2229
HW3,6.5,3.4	61.003	0.41	2482
HW5,4.5,3.4	24.57	0.66	1818

HW5,5,3.4	32.278	0.609	1994
HW5,5.5,3.4	41.596	0.59	2684.4
HW5,6,3.4	52.995	0.52	2919
HW5,6.5,3.4	62.385	0.5	3082