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Understanding the structural behavior of the minaret of Al-Umayyad Mosque of Aleppo, Syria and providing strengthening interventions against seismic excitations

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Contents

Abstract

Masonry structures had a great importance in the past in the construction field. A lot of masonry structures have been constructed around the globe like churches, mosques, museums and colosseum etc. Experimental studies and surveys have suggested that most of the masonry structures have failed structurally when they are subjected to high intensity Earthquakes. One of the types of masonry structures which have been give great attention are 'Minarets'. Minaret is a slender structure which is built near the mosque surrounding.

In this thesis, the main concern is to study the structural behavior of the Minaret of 'Al-Umayyad Mosque' against seismic actions located in the city of Aleppo, Syria. At present, the minaret is damaged. The minaret is to be built again with the same materials and geometrical characteristics but there are some restrictions over it. Thus, the main scope is to understand the structural behavior of minaret against seismic actions and on the basis of behavior of the minaret, the task is to provide strengthening interventions to make it safe against seismic actions that may act upon it in real life scenarios.

For this purpose, numerical modelling of the structure is done. The software use for the modelling is Abacus. Finite element method (FEM) tool is used for the analysis using solid and shell elements. The analysis is performed on two types of models. 1) Complete minaret structure including outer boundary, central pillar and stairs. 2) Outer boundary only. The seismic analysis is performed for the combination of self-weight and earthquake excitation. The Response spectrum analysis is used for the purpose and the analysis is performed considering linear elastic material behavior. After carrying out the seismic response, the purpose is to find out the critical regions against seismic excitation and provide strengthening interventions against those critical regions.

Moreover, the contribution provided by central pillar and stairs is also discussed by comparing the results from two models. Different types of strengthening interventions are defined which can increase response of the minaret against seismic excitations. The increase in the seismic response of the minaret is discussed in term of increase in the value of the peak ground acceleration (PGA).

1. Introduction

Masonry is the most significant building material which is known to mankind since the beginning of civilization. It has been used a lot in the past for the construction for cathedrals, mosques and cities which have last long and are present today in stable condition in spite of experiencing high intensity of Earthquakes. Masonry is also being used nowadays for construction because of simplicity of its design, construction and the attracted feature it contain.

Masonry structures have high vulnerability to seismic actions. They are prone to failure when they acted upon by seismic forces. For two decades, emphasize is being given to experimental study of the behavior of masonry structures. However, in spite of experimental studies the response of the masonry structures to Earthquake actions is still questionable. Moreover, the large scale dynamic testing is very costly which makes it difficult to perform experiments quite often. For this purpose, the numerical modelling and analysis of the structure has been proven an important tool to character the response of the masonry structures. This is because of the reason that nowadays the computer software are efficient enough to provide proficient and reliable results.

1.1. History of Masonry architecture

The first used masonry structure was made up of stone masonry. Before 9000-8000 BC stone masonry houses were found through archeological surveys in near Hullen Lake by Lourenco and Oliveira. Stone masonry is difficult to shape and due to its weight it is difficult to transport also.

Then the legacy of masonry structures shifted to mud brick houses. Before 8350-7350 BC, mud brick structures were being made in the area close to Palestine. These structures were used to be construct in dry climate regions where clay mud is available.

Considering the Egypt, from Dynastic times (5000 BC) up to the Roman occupation of the land (50 AD), the structures were primarily made from sun dried bricks. This structure use to shrink a lot which produces cracking in the structure. To overcome this thing, the sand and straw were mixed which reduces the formation of cracks. Later on, the concept of sun-dried bricks were taken over by the burnt bricks because of higher strength.

The need to understand the structural behavior of the masonry structures gained importance when stone lintels were being made to support the weight of masonry above it. This was significantly used when temples were being made. The arch concept for underground structures was first used by the Indus valley civilization, then by Greece, Egypt and Mesopotamia. But Romans were first one who used Arch for the structures made above the ground. The Greeks were the first one to use columns and beams for the structural support. While Romans has contributed a lot to the masonry structures by building roads, buildings. One of the finest example of Roman masonry structure is Roman Colosseum.

After 6th century AD, Persian and Ottoman Empire invested a lot to build domes and minarets for the mosques. These structural elements were constructed in abundance in these parts of worlds. They were primarily made up of stone masonry. Domes were especially made to overcome large span problems. Moreover, they have magnificent architectural view which has made them an integral part of the construction of mosques in the Muslim world. Minarets were built in mosque, at least one because of religious importance. They are slender structure with vulnerability to seismic actions. The examples of such structures is Blue mosque in Turkey which have four minarets. There are several minarets having beautiful architectural view in the Isfahan city of Iran. Moreover, there are minarets in Syria, Afghanistan and other Muslim populated countries. Similarly, among Christianity the church towers have huge importance which are structurally more or less as same as minarets.

Following are some famous masonry structures around the globe which have significant reputation.

Figure.1. Famous Masonry Structures

The structural behavior of all above mentioned masonry structures is mandatory to understand especially to seismic actions. Most of the masonry structures present today have significant importance from heritage and cultural point of view. So in order to preserve these structures against natural hazards we need to have study of these structures in detail.

1.2. Scope and Objectives

The following study deals with understanding the structural behavior of the masonry minarets against seismic actions. For this purpose, a minaret of 'Al-Umayyad' mosque present in Aleppo city of Syria, is selected. The minaret is destroyed because of explosion due to bombing, but the actual reason is still not clear. So there is kind of uncertainty about failure of the minaret whether it is fallen because of its weight, some seismic actions or explosion due to bombing. The minaret has enormous significance in Muslim world and it is also a world's heritage site of UNESCO. So there is need to rebuild the minaret again but there are some requirements which are needed to be fulfilled. These requirement will be discussed in detail later but one of the main requirement is to build the minaret with the same material to restore the cultural and heritage importance of the minaret and mosque.

Figure.1.1. Minaret of Al-Umayyad Mosque

Following are the objectives of the work to be performed;

1) To make the numerical model of the minaret. The model should be such that it should show as close as possible the actual performance of the minaret. In this way, it is possible to figure out the response of the minaret in an accurate way. For this purpose modal analysis is carried out to know about the frequency, time period and mode shape of the minaret.
2) To carry out the linear elastic analysis of the model to figure out the seismic capacity of the minaret. Response spectrum analysis is used for this purpose.

3) On the basis of the seismic response obtained in Step#2, the design interventions are provided (Hoop and flexural reinforcement) for the minaret to increase the resistance of the minaret against seismic actions.

4) Keeping the model in Linear Elastic regime, the model obtained in step#3 is again subjected to Response Spectrum analysis to compare the results so that increase in seismic response can be quantified.

5) After carrying out linear analysis, the minaret is taken into non-linear regime, and is subjected to real life scenarios. In this step, the non-linear model with and without design interventions is acted by an Earthquake so that the response of the structure can be measured and compared in non-linear regime also.

1.3. Contents Outline

The content in this thesis is divided into six chapters.

In first chapter, the overview of the masonry structures, scope and objectives of the thesis is explained.

In second chapter, the literature review relating to minaret, geometrical configuration of minaret, types of minaret, construction techniques of minaret are mechanical properties of the minaret are discussed in detail. Moreover, the response of different minarets to seismic actions is also discussed in detail. Then, the date relating to Minaret of Al-Umayyad mosque is also discussed in detail. It includes geometrical configuration, construction material properties, historical events on minaret and the issues relating to reconstruction are also elaborated in detail.

In third chapter, the procedure for making numerical model is described in detail. The type of material used, the type of mesh used and all things relating to modelling is described.

In fourth chapter, the response of the minaret to linear seismic analysis is carried about. The results are displayed and discussed. The comparison is made between the two types of models.

In fifth chapter, the strengthening interventions are defined. The results are discussed and compared.

In sixth chapter, the conclusion are made based on the analysis results and points are prescribed for further research on this topic.

2. Literature Review

In this chapter, the content about minaret, geometrical configuration of minaret and the previous studies relating to seismic analysis of the minaret are described in detail.

2.1. Minaret

Minaret is an Arabic word and in Persian language it is called as Goldaste which means bucket of flowers. Minaret is a tall slender structure made from masonry and is a distinctive architectural structure akin to a tower. It was very common to be constructed in the Muslim world in the past years. Minarets are either separate or attached to the portico of the mosque.

2.1.1. Functions of Minaret

In the past, in Eastern regions having warm weather, the purpose of minarets was to provide ventilation. Usually the minarets have an opening in the ceiling which helps to remove the warm air through the cupola. Thus it leaves the cool air at the lower part of the minaret. Similarly for mosques with domes, the purpose of this construction was merely to provide ventilation.

The function of the minaret gradually changed to religious purposes. The speaker is set on the top balcony of the minaret and is used for 'Adhan' which happens for five times a day.

2.1.2. History

The use of Minarets have long history. They were notably in Middle East and Mesopotamia even before the advent of Islam.

For one mosque, one minaret used to be enough. But later on keeping in view the aesthetics and symmetric requirements, the number of minarets for a mosque were increased. In the Islamic world, for centuries, the Holy mosque of Mecca only had six minarets. Then the blue mosque of Turkey was the first one after mosque of Mecca to have more than one minaret.

2.1.3. Structural Typology of Minarets

On the basis of the structural typology, the minarets have three different types.

1) Cylindrical
2) Cone
3) Polygonal

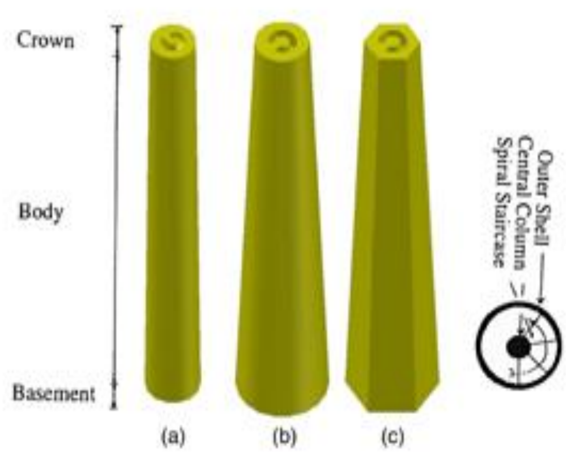


Figure 2.1. Different typologies of Minarets

Generally, the minaret consist of three main parts. One is basement which is constructed inside the earth on rocky or solid layer. The second is main body which comprises of outer shell, central pillar and stair case which can be spiral or straight. The third part is called 'crown' which is constructed on top of the body. It may or may not have a balcony. The height of the minarets are generally 20 m to 50 m. Their outer diameter of the shell is 2.5 m to 6 m at the top and it can vary in some cases and the top it can get to 2 m to 5 m. For the central pillar, the diameter may vary from 0.3 m to 1.6 m. (Hejazi 1997)

The structural typology of the minarets also vary based on the region, cultural and heritage values. The minarets of Turkey, Iran, Egypt and Syria are more or less same but they are different in terms of segments configuration and construction techniques.

Below is the Turkish minaret, named as 'Iskenderpas' minaret, it contains all the segments that we may see in all the minarets of the world.

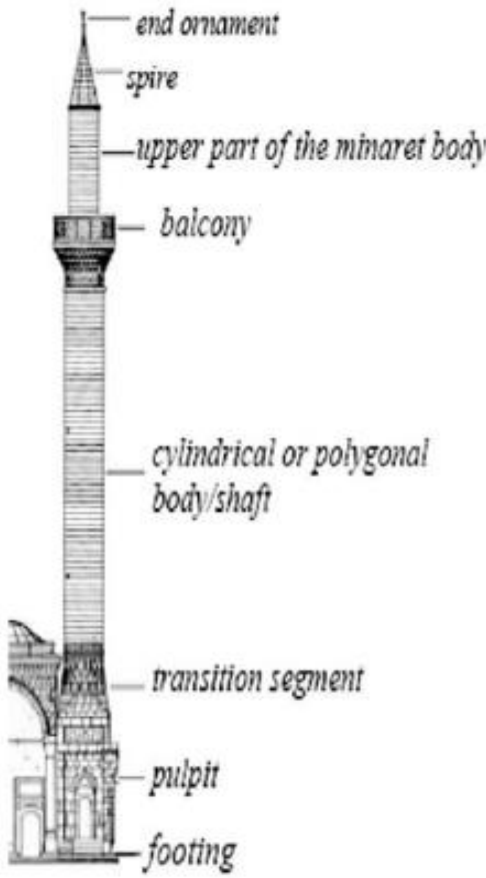


Figure 2.2. Iskenderpas Minaret

The description of the segments is explained in detail; as follows,

1) Footing

The footing is made up of thick rigidly connected stone blocks. The footing is sometimes attached to the adjacent walls of the mosque.

2) Pulpit

The pulpit is a small portion of the minaret having height (5 m-10 m). It is constructed above the minaret and is generally used for entrance to minaret. In Turkish minarets, it have square or octagonal shape. But sometimes it may have also polygonal shape with 10, 12 or 16 faces.

3) Transition Segment

It is usually of the height of (2-3) m. It provides transition from pulpit to cylindrical or polygonal body. The transition is smooth which is ensured by using cut stones, pyramids, planes or inverted triangular shaped elements. It is also of varying size along the height.

4) Cylindrical or polygonal body/shaft

It is the longest part of the minaret structure. It covers almost two-third of the height of the minaret. It is usually of the cylindrical shape. Square and orthogonal shapes were also very common. During Seljuk period (One of the ruler in Ottoman Empire), the tapered cross-sections were widespread. Since then, the minarets with constant cross-section were fabricated.

5) Stairs

They are an important part of structural system of the minaret. They were made with steel, masonry or timber materials. But in recent years, reinforced concrete is in demand for use as stairs material. In some minarets, the presence of the stairs strongly affect the performance of the minarets against horizontal forces such as Earthquakes or Wind.

6) Balcony

During old times, the balcony was used for call of prayer. But with the invention of loudspeakers, the use of balcony has only architectural and visual purpose because of its aesthetics.

7) Upper part of the cylindrical/ Polygonal Body

This part is between the balcony and the spire. It has same purpose as lower cylindrical body. It has lower cross-sectional area as compared to the lower part.

8) Spire

The spire is the top most part of the minaret which serves as a roof. It is typical among Muslim world to make such structure at top of minaret. It is usually of the conical shape or dome. It may or may not be made up of same material as upper cylindrical body.

9) End Ornaments

It is a small piece which does not have the structural importance. It is just a symbol to show the end of the minaret. It is placed on top of the minaret.

Finally, the minarets present in other regions can be briefly classified as follows;

a) Minarets of Egypt have several balconies. Their size decreases as the height is increased. The top is usually covered with rounded dome and bottom cross-section is commonly rounded.

b) The minarets of Morocco and Spain are large slender structures. Their cross-section is square mostly and richly decorated. The material is brick which is made with several patterns to improve the beauty of the minaret.

c) Persian minarets are high, slender and tapered structures. They have balcony which is at bigger height and a small dome which caps the minaret.

d) Minarets of Syria have usually square cross-section for main body of the minaret. The cross-section is square for the main body but it varies for the pulpit. The top is covered with dome and it does not have balcony in most cases. The call for prayer was used to make through the roof.

A few of prominent minarets present around the world are shown in the subsequent figures;

Figure 2.2. Different Minarets present around the World

2.1.4. Construction of Minarets

In the following, the general scheme used for the construction of the minarets in explained.

The minarets were mainly constructed by using stone blocks, bricks or with combination of both. The mortar was also used as a binding agent for the stone blocks and bricks. The bricks used were normally clay bricks. The usual material which was used for the construction of the Turkish minarets was Limestone. Oguzmert (2002) performed non-destructive testing on different Turkish minarets to figure out the mechanical properties of the minarets constructed by high quality limestone. The following table shows the mechanical properties of the Limestone material;

High Quality Limestone	
Dry Density	23.9 KN/m^3
Fully Saturated Density	24.5 KN/m^3
Compressive Strength	16 Mpa
Tensile Strength	0.9 Mpa
Modulus of Elasticity	5860 Mpa
Poisson Ratio	0.25

Table 2.1. Mechanical Properties of High Quality Limestone

The modulus of elasticity and compressive strength for low quality Limestone can drop down to 3000 Mpa and 5 Mpa respectively.

The stone blocks and bricks have high compressive strength which is enough to resist compressive forces produced in the minaret under condition of high seismicity. But mortar have very low tensile strength due to which, under earthquakes, minarets are prone to failure. This problem was encountered by Turkish during earthquake of 1509. Ottoman's tackled the problem by using special technique. The idea was to make the structure earthquake resistant. They started to use iron pieces and clamps to lock stone blocks as shown in the following figure which increases their seismic response. The iron reinforcement was used in horizontal and vertical directions. The vertical iron bars or clamps were injected into the stone blocks by making anchorage holes in the masonry. The melted lead was also poured into these holes to provide bond between bars and stone masonry. About 2000 kg of this heavy metal lead was used for this purpose. The idea was to get high seismic response of the minaret due to this additional weight of lead which have proven successful because of the stability of the minaret after so many years. This idea was implemented in the minarets of the famous Blue mosque. The mortar used to bind stone blocks was of special type known as 'Horasan' mortar. This mortar along with lime was filtered and it left for 10-15 years underground to yeast for the purpose of gaining more strength. The physical interpretation of this construction technique is depicted in the following figure;

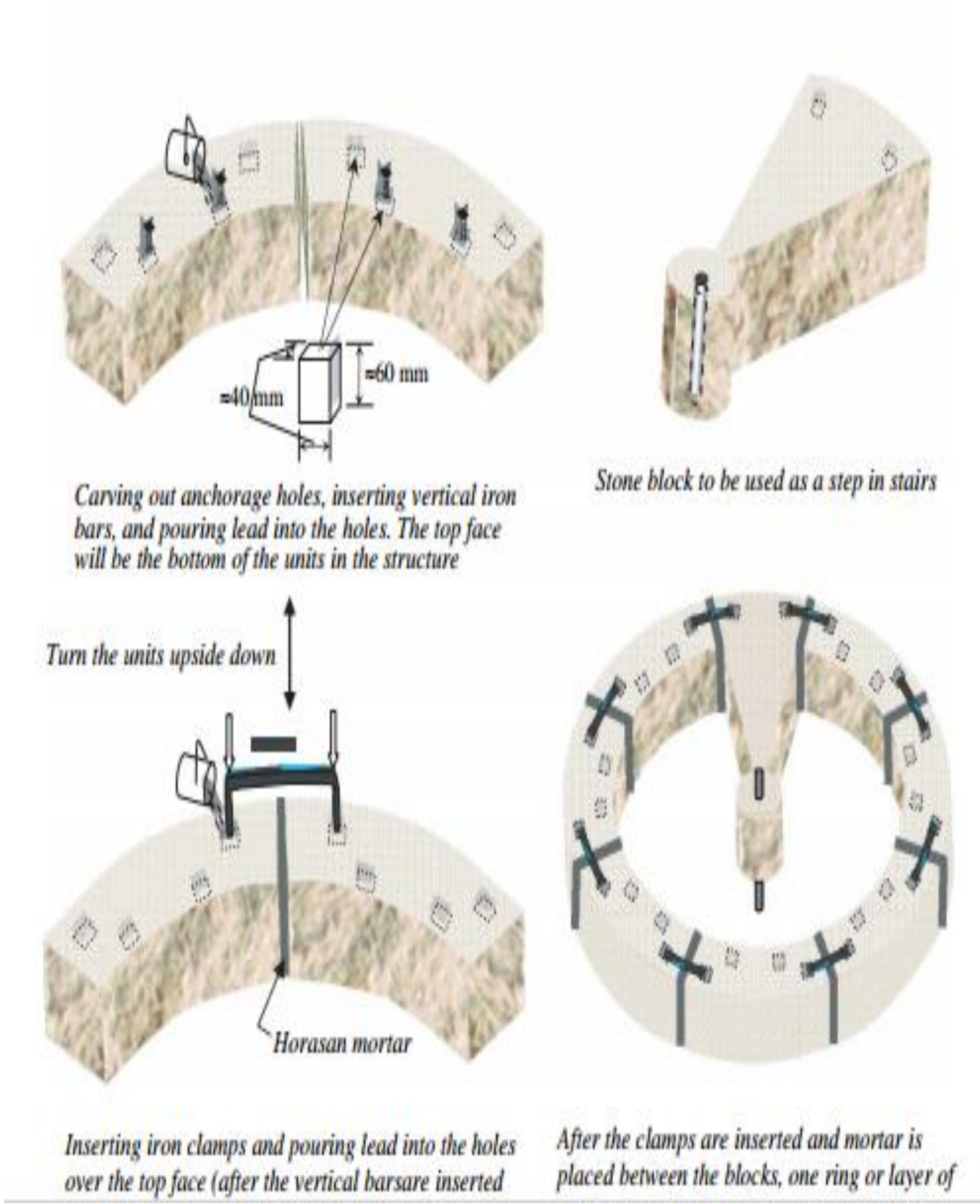


Figure 2.3. Physical Interpretation of Construction Technique

2.2. Minaret of the Al-Umayyad Mosques of Aleppo

The minaret under study whose seismic response is to be evaluated and new design interventions are to be proposed is the minaret in the famous city of Aleppo of Syria.

The Al-Umayyad Mosque of Aleppo is the oldest and biggest mosques of the Aleppo. It is located at the old city of Aleppo in a district which is as known as al-Jalloum. It is considered as the world's heritage site by UNESCO since 1986. It was constructed during the reign of Umayyad in 8th century (715). That's why it is called as Al-Umayyad mosque. The site on which was built was the core and an important place for the millenaries. The site of the Great Mosque was once the agora of the Hellenistic period, which later became the garden for the Cathedral of Saint Helena during the Christian era of Roman rule in Syria.

Figure 2.4. Al-Umayyad Mosque

The mosque suffered a lot of damage since it was built in 8th century. In the following centuries after 8th, it was attacked by Eastern Roman Empire due to which it was badly demolished. In 1090, when Muslims again got the control of the area, they built it again under the ruler 'Abu'l Hasan Muhammad'. He also gave orders to make a minaret to symbolize the mosque as a property of the Muslims. The 40 m long minaret was constructed in the northeast corner of the mosque which construction was completed in five years from 1090 to 1094. It was one of the most decorated and significant minaret in the Muslim world. On the artistic level, it was considered to be the best from architectural point of view. It had great amount of design work over it and many verses of Quran were written over it.

2.2.1. Location of the Minaret

The minaret is constructed at the northeast corner of the mosque. The entrance to the minaret is on the southern side and the northern side of the minaret can be seen from the main gate of the mosque. The west side of the minarets have connection to the walls of the mosque while east side is free and can be seen from the road adjacent to the mosque.

The location of the mosque on the map can be seen in the following figure;

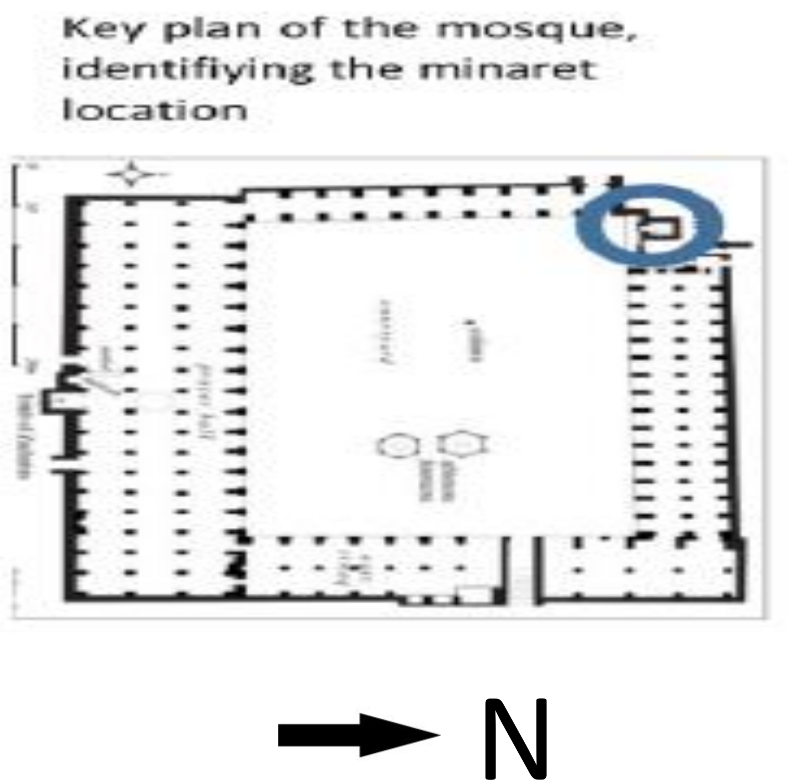


Figure 2.5. Location of Minaret in Mosque

2.2.2. Geometrical Configuration of minaret

a) Height

The height of the minaret is 37.540 m. It consist of five floors of almost a square cross-section. The dimensions of the cross-sections are constant along the five floors. The distribution of height among the components of the minaret is as follows;

- Footing: The footing of the minaret is about 209 m.
- Pulpit: The pulpit is 73.80 m long. Its cross section is different as compared to the rest of the minaret. It provides the entrance to the minaret.
- Transition Segment: The transition segment is 307 m long. It has same cross-section as the main body of the minaret.
- Main Body: The main body 2700 m long. It has four floors for which cross-section is as same as transition segment. It has several openings for the ventilation purposes. The size of the openings is not big. It varies from 1.325 m $*$ 0.9 m to 0.530 m $*$ 0.45 m for rectangular openings and have radius of 0.225 m for circular openings.
- Spire: At the top of the minaret there is a dome which is supported by small masonry columns and a roof which was used for the call of a prayers.

The minaret also have a central pillar about which stairs revolve and goes up to the five floors, and at the top we get to the veranda where the minaret have hemispherical dome. The façade of each direction is shown in the following figure;

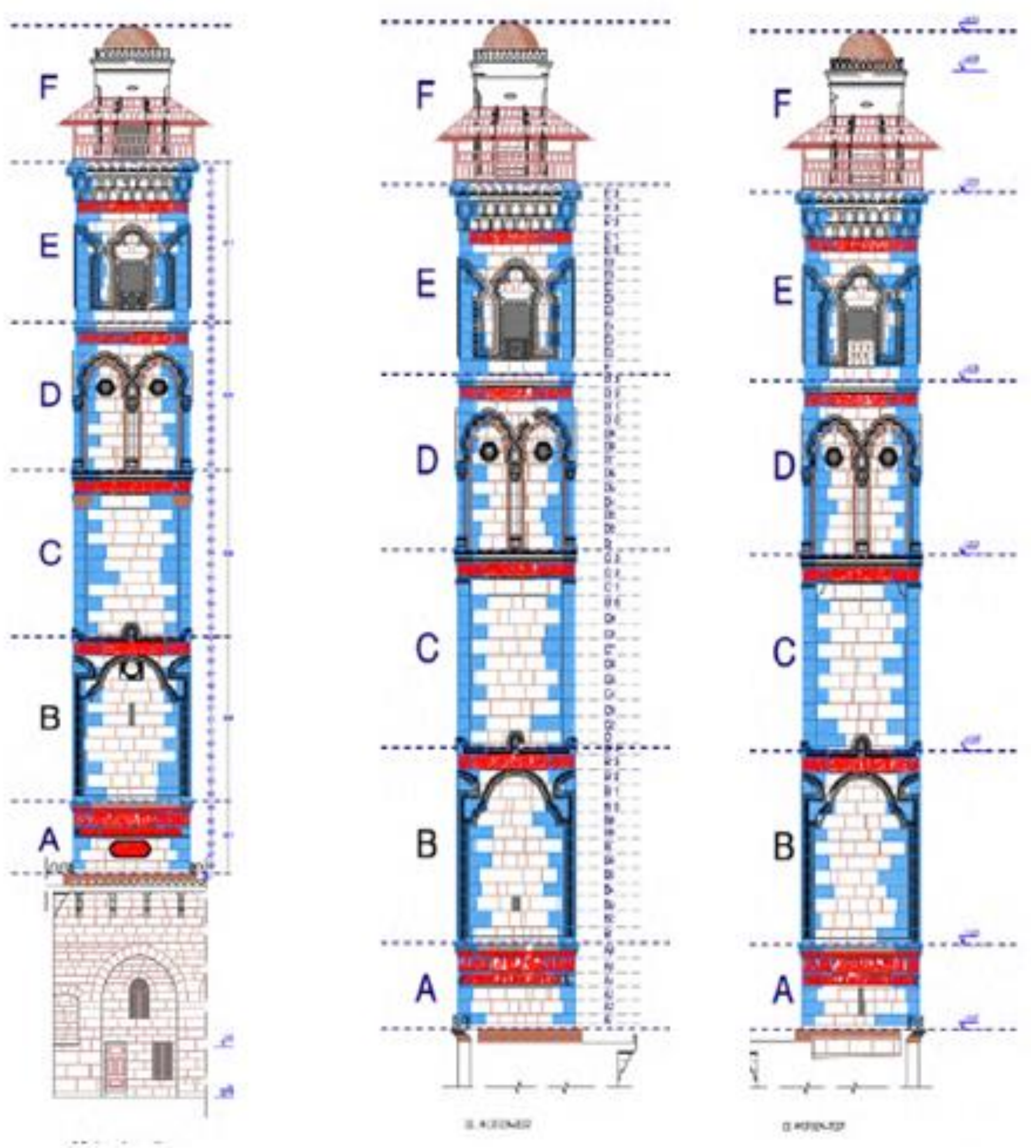


Figure 2.2.6. Elevation View of The minaret in different directions

The left end shows the South facade, middle shows the West façade and the right shows the East faced of the minaret.

b) Cross-sectional Dimensions

The cross section of minaret is of two different typologies.

- **From base to 9.470 m height**

The cross-section is shown as follows;

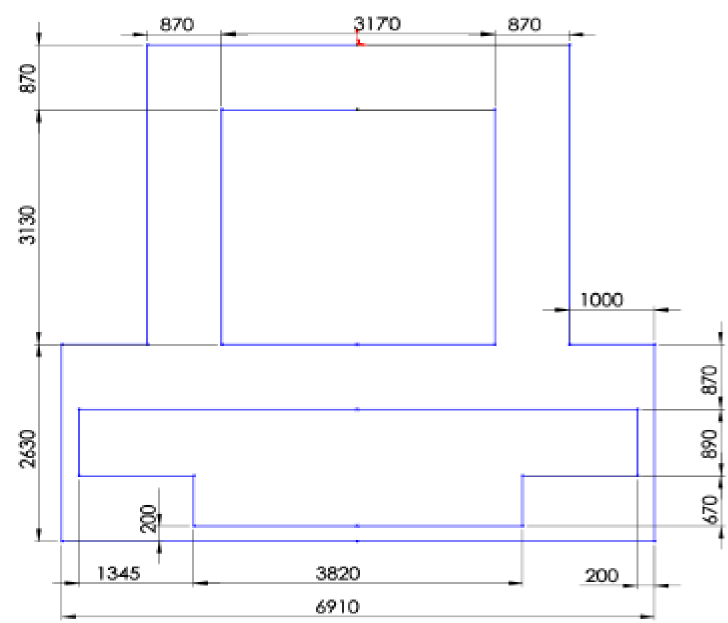


Figure 2.8. Cross-Section at Base

- **From 9.470 m to 3.9540 m height**

The cross-section is square and is shown in the following figure;

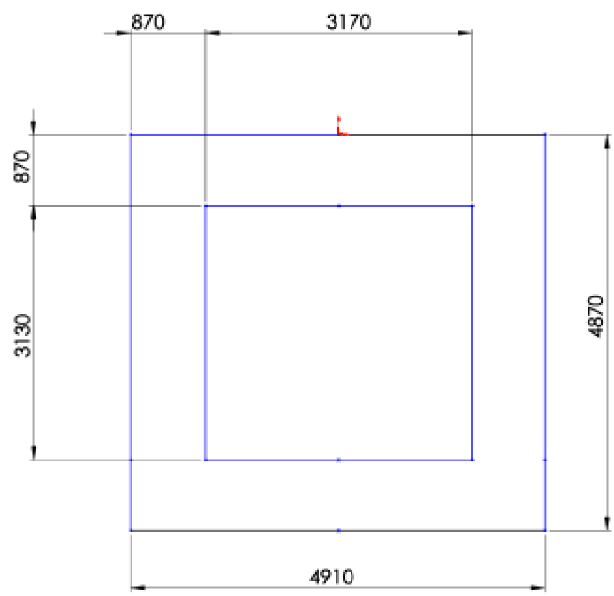


Figure 2.8. Cross-Section at Junction

2.2.3. General Aspects in the design of the minarets

Minarets are tall, slender structures having vertical development along their height. They almost have symmetry long vertical axis. So the main forces for which a minaret can be designed are;

a) Self-weight of the structure

b) Lateral or Horizontal loading (Earthquake or Wind loading)

The gravity loading acting on the minaret is not so high so the minaret can withstand their self-weight without losing stability. However, the main concern in design of minarets is related to lateral loading. The minarets are vulnerable to earthquake or wind loading. There are no specific design codes for design of minarets in Turkey, Iran, Syrian or any other country where we have majority of minarets. However, there are some guidelines for towers, bell towers and other structures which have vertical development which we can consider as same as minarets. These guidelines are available in the chapter 5 of the document which is related to the evaluation of the seismic response of the cultural heritage to reduce the seismic risk with reference to the technical Standards for Construction set forth in the Decree of the Ministry of Infrastructure and Transport on 14 January 2008, which illustrates the different possibilities of modeling the structural behavior of a historic construction in masonry for the assessment of seismic safety.

Since the minaret is a slender structure with vertical development, hence the 5.4.4 is the section which is assigned to calculate the seismic response of such structures. The Simple Mechanical Model (LV1) is used for these type of structures.

The seismic design or response of these type of structures depend heavily on the slenderness which is highly variable parameter and presence of adjacent structures. The presence of structures provide stiffness to the minaret and change the seismic response. The response of these structures can be easily extracted by using linear models and running out dynamic analysis i.e. Modal analysis. Such analysis provides basic data to understand completely about the response of the structure. Due to the difficulty of obtaining models which completely describe the behavior of these structures, it is recommended to follow simple mechanical models. To know about the limit situation of the minaret, the simplified mechanical model can be considered to be acted by its own weight and by horizontal forces such that bending is produced in the structure. Hence, because of bending there will be crushing in the compression regions of the minaret and failure due to non-tensile strength. The seismic resistance is compared by calculating the applied and resisting moments based on the model having no traction and non-linear distribution of masonry. For this purpose, the checks are performed for two main directions as moment of inertia along two directions might be different. Also the checks are performed for different points along the height minaret because of the unknown nature of the critical section. The resisting moment is calculated by the following formula;

$$M_{ui} = \frac{\sigma_{0i} A_i}{2}(b_i - \frac{\sigma_{0i} A_i}{0.85 f_d a_i s_i})$$

Where;

σ_{0i}= Average Normal stress in the section (W/A);

W = Weight of the structure above the considered section;

A = Cross-sectional area of the considered section

a_i = Dimension of the side perpendicular to the direction of Earthquake considered purified by an opening

b_i= Dimension of the side parallel to the direction of Earthquake considered purified by an opening

f_d= Compressive strength of the material used.

Whereas, the formula used for calculating the applied moment is given in the following;

$$F_i = \frac{W_i z_i}{\sum_{k=1}^{n} W_k z_k} F_h$$

Where;

F_i = It is the force applied at the body-center of the considered portion;

W_i and W_k = It is the self-weight of the considered portion and portion above respectively;

Z_i and Z_k = are the body centers of the considered portion and above it repectively;

F_h= 0.85S_E (T_1) W/qg = (Period T_1 is always considered to be greater than the T_B which is Th period corresponding to the start of constant portion of elastic response spectrum.

The bending moment can be calculated by using following formula;

$$M_{si} = F_{hi}\ z_{Fi}$$

Where;

$$z_{Fi} = \frac{\sum_{k=i}^{n} W_k z_k^2}{\sum_{k=i}^{n} W_k z_k} - z_{i*}$$

2.2.4. Construction Material of the Minaret

The minaret was constructed with stone blocks of ‘Limestone material’. As usually happens in traditional masonry structures, the stone blocks were bound together by using clay mortar. The properties of the material used for the construction is unknown due to unavailability of the data of the mechanical tests performed on minaret. However, based on the material used and carrying some literature work on its characteristics and performance, the mechanical properties can be estimated and can be used for the thesis project.

The mechanical properties of the material used in this project are based on the Italian guidelines present in the Italian codes for the analysis of masonry structures.

Mechanical Properties of the material	
Material Type	**Limestone**
Dry Density	**1600 kg/m^3**
Shear Stress	**0.028 Mpa**
Compressive Strength	**2.4 Mpa**
Tensile Strength	**0 Mpa**
Modulus of Elasticity	**900 Mpa**
Poisson Ratio	**0.3**
Shear Modulus	**300 Mpa**

Table 2.2. Mechanical Properties of the Material used for the Project

The original design of the minaret was based on the self-weight of the structure and no provisions were made for earthquake or wind loading. At that time, there were no design codes to be taken into account. So the mere idea was that the compressive stresses will be resisted by the stone blocks because they are considered to be best in business when acted upon by compressive forces and ,in order to tackle tensile forces in the structure, the mortar was used in between the stone blocks. Strength of mortar in tension is not very high so the performance of the minaret under horizontal loading was not guaranteed. However, the minaret still managed to survive the horizontal loading.

2.3. Historical Events

Since the construction of the minaret in 1094, there has been no major damage done to the minaret. There has been many deadliest earthquakes experienced by the city of Aleppo since the construction of minaret. Out of which, the earthquake of 11 October 1130 which is considered to be the third deadliest earthquake of the world. The magnitude of the earthquake was 7.6 and it had damage level of XI according to Mercalli intensity scale. There is another which happened in 1202 but it was not as intense as the 1130 earthquake. In spite of all these events, the minaret did not suffer any major damage during its lifetime. Hence the minaret did not have any specific interventions in the past. However, some renovation works were performed on minaret during 2003 by Syrian Government.

The minaret was destroyed and reduced to rubble in April 2013. During the war in Syria which started in 2011 and is still going on, the mosque was occupied by rebel forces. During a fight in April 2013, the fight broke out between rebel forces and Syrian army, the minaret was attacked by unspecified number of artillery strokes which was the reason behind devour of minaret. But it is still not clear which group fired the bullets on minaret. Concerning media reports from different groups, it has been said that the rebel forces were behind destroy of minaret because Syrian armed forces were few yards away from the minaret. After the fight, the Syrian Government took control of the area and the Syrian President ‘Assad’ passed on orders to reconstruct the minaret.

The following figures shows the minaret was destroyed badly and it was crushed into rubble masonry.

2.9. Destroyed Parts of the Minaret

2.4. Issues with Reconstruction

Reconstruction of a cultural heritage site is always a debatable issue. It creates confusion between professionals and communities. The cultural sites which are to needed to be constructed again are very important for the integrity of the culture, area and emotions of the people. However, some international norms and principles consider not to reconstruct a cultural site because of danger of loss of integrity of the site. Both of these states have their own justifications;

The justifications given for reconstruction might be;

- The reconstructed site can improve tourism and economic development of the country.
- It can be the only option to preserve the site for longer time and keep the emotions of the people intact.
- The site can continue to serve its function in the future if it was an important site to the people who were attached to it.

While on the other hand, the defenses given for other scenarios are;

- The reconstruction may produce something which may not show completely the aspects of the site and in this way the integrity of the site may be lost.
- The cost of reconstruction is very high.
- The romantic value of the destroyed building can be more attractive as compared to the reconstructed one.

- The difficulty of achieving authenticity because reconstruction involve conjecture to a greater or less degree.

There are many international norms and principles relating to reconstruction of the cultural sites, some of them are in favor and others are not. Every norm has its own valid reason and debate.

The reconstruction has been happening since Roman Antiquity and in 19th century it takes its serious form. After the demolitions caused by the French Revolution and subsequent social changes brought awareness in authorities to start considering about the reconstruction of heritage.

The restoration practice developed during the19th century represented by a number of very different individual approaches and criteria in Europe. One of the prominent architect in the past Viollet le Duc said that 'Reconstruction converts a building into a new state that has never existed before'. John Ruskin spoke out in opposite and insisted on aesthetical value of the ruin of the site.

In 1964, the Venice Charter, published a document which recognized the value of reconstructing lost buildings if they were properly documented but it recommended the replacement of missing parts "to integrate harmoniously to the whole, but at the same time must be distinguishable from the original so that restoration does not falsify the artistic or historic evidence".

In 1982, the Dresden Declaration accepted and it stated that all the cultural sites and monuments which are destroyed by war should be reconstructed. The Soviet Union East Germany and Poland were very concerned about their history and integrity of their cultural sites. So they made everything possible to preserve the structures.

The Riga charter on authenticity and historical construction was taken place in 2000 and it stated that all the sites which were destroyed wither because of natural disasters or human being interventions should be reconstructed.

Lastly, Paris in 2011, considered the reconstruction as main ingredient and driver force behind the sustainable development. In the meanwhile, the charter of Venice was renewed and precision of the conditions was increased under which the reconstruction is to be proposed.

The suggested guidelines relating to reconstruction of a site depends on many factors. It involves social, economic and cultural factors and these guidelines cannot be general and accepted worldwide. However, in the present case of minaret of Al-Umayyad mosque, which is an important integral part of the famous mosque in Syria which is city's landmark place. This thing makes an important mark for the reconstruction of the minaret and its reconstruction will also act as a sign of city's recovery. Moreover, the minaret was being used for the call of prayer which is a religious activity and its reconstruction can restore this function. Overall, its reconstruction makes an important cause.

However, there are some specific and strict guidelines have to be followed to preserve the integrity of the minaret which are as follows;

a) Reconstruction should make use of the material debris as much as possible so that material authenticity can be make assured.

b) Reconstruction should be on the basis of authentic surveys and historical, material evidences.

c) Principles of engineering and physics which are applicable presently or in modern era can be applied for reconstruction activities.

d) Existing significant historic fabric shall not be damaged because of the reconstruction.

e) Training and teaching about technical knowledge to local craftsmen and engineers is key to assure sustainability of local community.

f) The issue of reconstruction shall be discussed with local and national authorities and with the concerned community.

3. Numerical Modelling of Minaret

In this chapter, the modelling of the minaret is explained. The model considered for the analysis is numerical model. The model is made using well known commercial software ABAQUS. The version of the software is DS Simulia Abaqus/CAE 6.15-5. It is developed by Dassault Systems which is a part of Dassault group which among other things, also produces airplanes including the formidable Mirage.

3.1. Types of Model

Two types of models for the minaret are made for the analysis. The minaret types are following.

a) Model A: Complete Minaret with central pillar and stairs inside.

b) Model B: Minaret with outer shell only.

The reason to study two types of models was to check out the effect of central pillar and stairs on the outer boundary of the minaret. Moreover, it will also show on the stiffness of the model with and without the presence of the central core.

3.1.1. Components of Model A

This type of model is named as ‘Complete model’. It has been given this name because in order to figure out the response of the whole minaret, the model is made with outer shell having central pillar and stairs inside. The information about the geometry of the minaret is taken from the document which cannot be disclosed because of privacy of Authorities involved in the reconstruction of the minaret. Following are the components of this model;

a) Outer Boundary

This is the outer boundary/shell of the minaret. Its height is almost as same se height of the minaret i.e. 37.45 m. It has two types of cross-sections, one is square with some rectangular portion adjacent to it, which runs from 0 m to 7.380 m height. Other one is almost a square which runs from 7.380 m to 37.45 m height. These two cross-sections modeled in Abaqus are shown in the following figures;

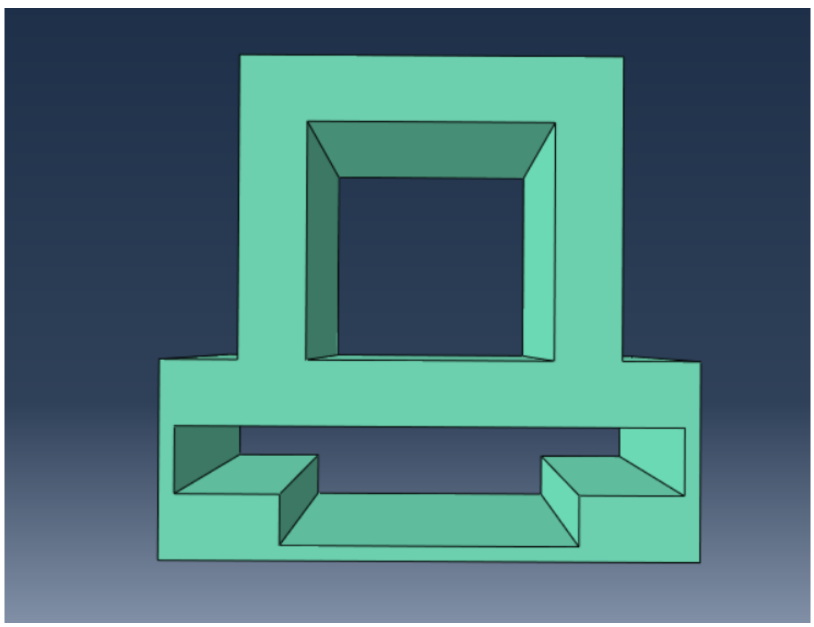


Figure 3.1. Modelling of Base-Cross-section

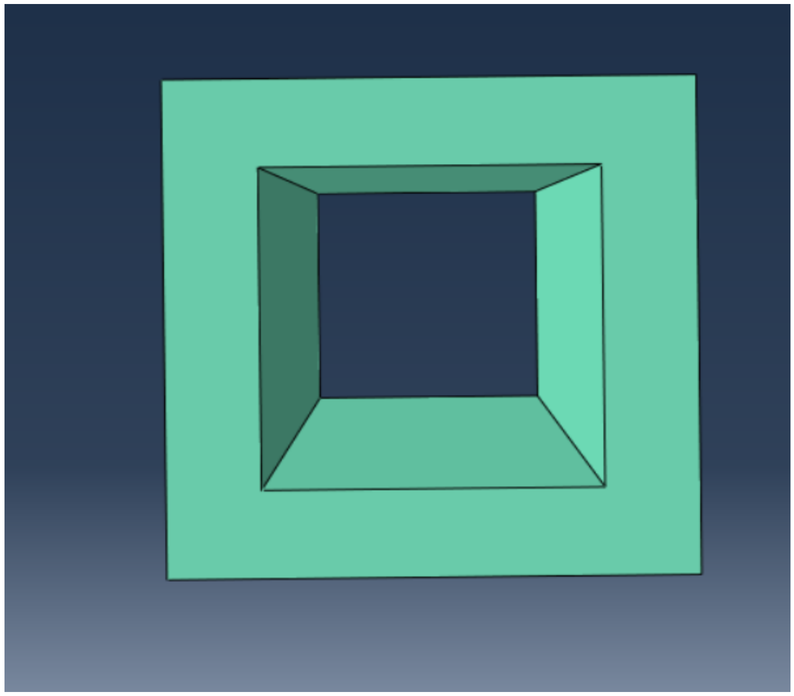


Figure 3.2. Modelling of Cross-Section at Junction

This part of the minaret is modelled as ‘3D Solid Deformable Part’ in Abaqus having 3 DOF’S per node. The overall typology modelled in Abaqus is shown in the following figure;

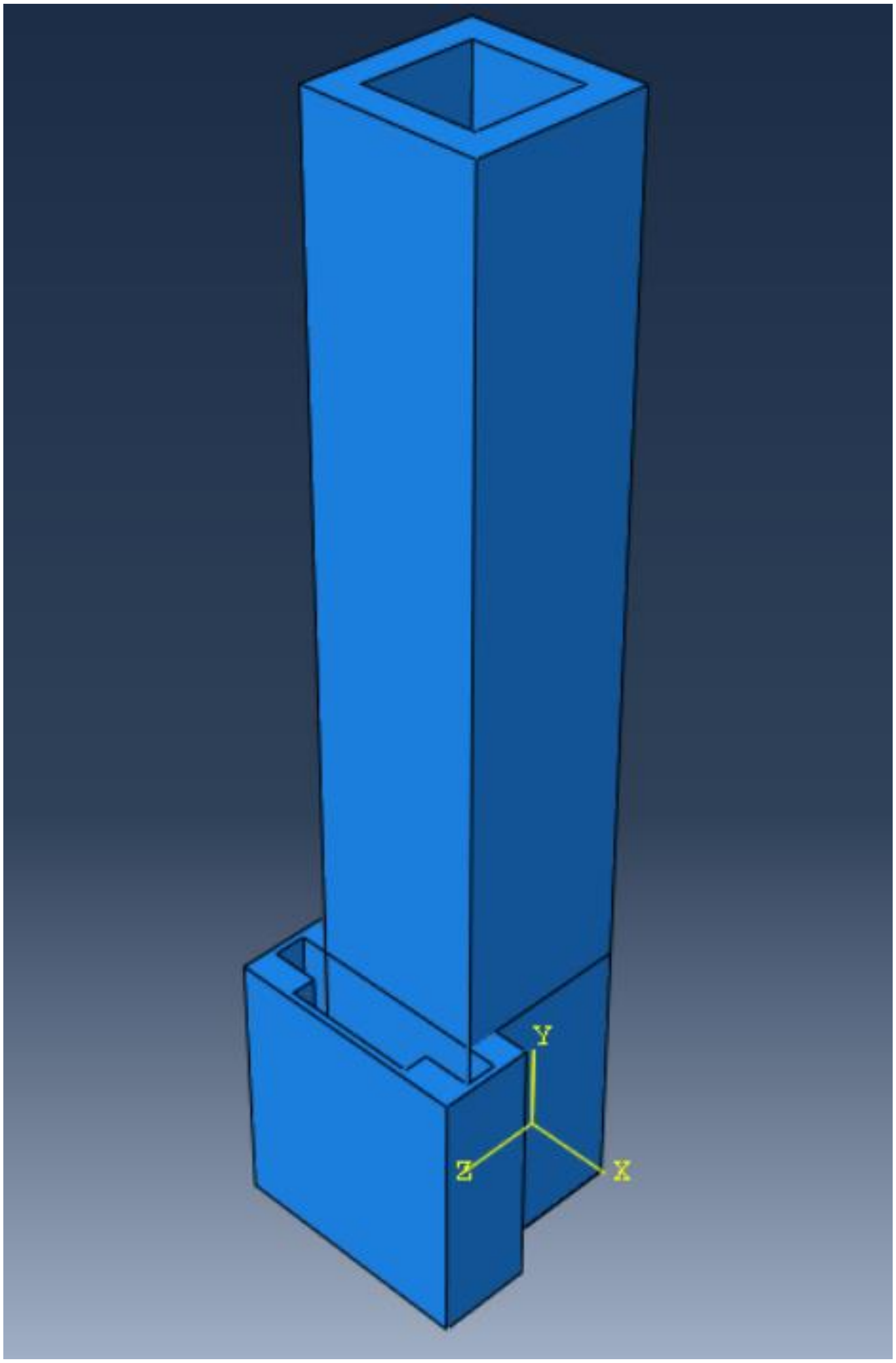


Figure 3.3. Modelling of Outer Boundary

b) Central Pillar

This part of the minaret is enclosed inside the outer boundary. The stairs run along the central pillar. Its height is equal to the outer boundary. It has same cross-section along its height. The cross-sectional dimensions 1220 mm * 1250 mm. It is also modelled as '3D Solid Deformable Part'. The cross-section and longitudinal view of the model is shown in the following figure;

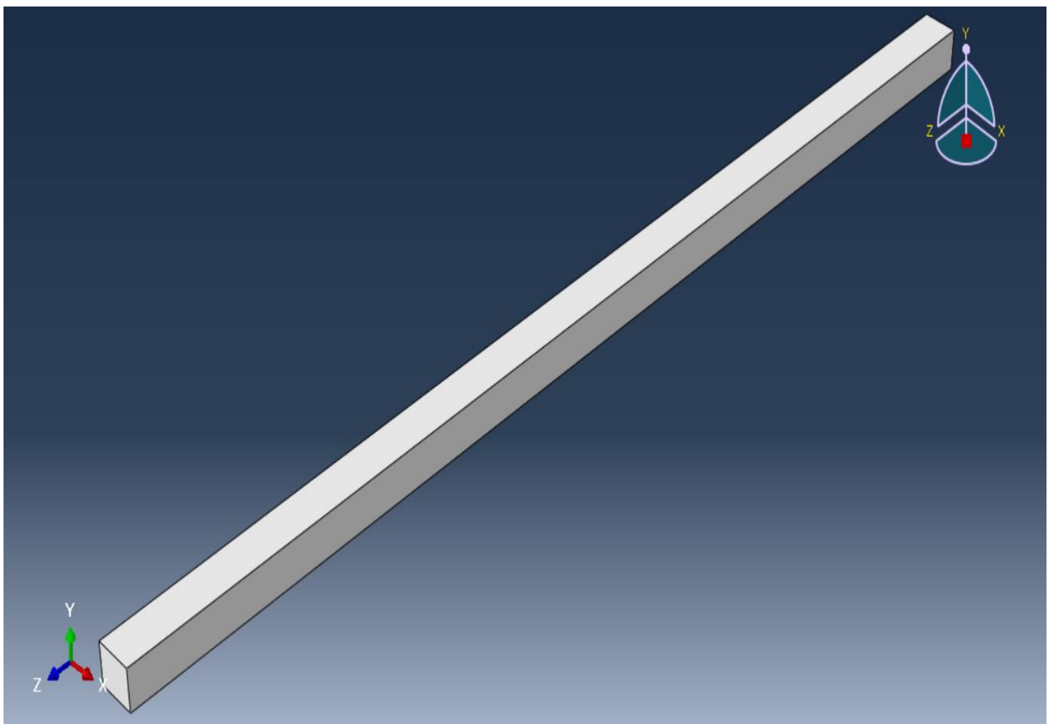


Figure 3.4. Modelling of Central Pillar

c) Landing & Stairs

Stairs are modelled using three different components. All the components are modelled as '3D Solid Shell Parts'. The depth of the shell is considered to be same for all the parts. The depth is 200 mm. The description of these parts is given below;

- **Landing**

The landing is modelled as square shell element. The landing for both directions i.e. X and Y have same geometrical configuration;

Figure 3.5. Modelling of Landing

- **Stairs X & Y**

Both of these components have same geometrical features. The depth for stairs along both directions have depth of 1872.5 mm. However, when modelling in Abaqus along two different directions, the draft angle used is different for stairs of different direction. The angle used for stairs along x-direction have value equal to 33.735359 degrees while for y-direction the angle used is 33.085666 degrees. This part is shown in the following figure;

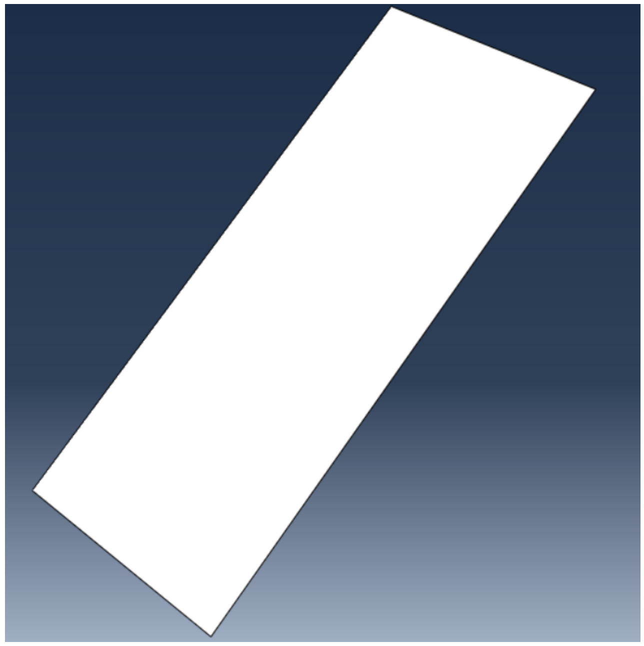


Figure 3.6. Modelling of Stair Steps

3.1.2. Material Properties of Components of Model A

The data relating to the material properties of the minaret has not been found in the literature. However, the reference is made to the Italian document which is referred to the analysis of the masonry structures as defined above. The scope of this thesis is to carry out the analysis in linear regime of the material. Hence, we finally opted the modulus of Elasticity equal to (E=900 Mpa), density equal to (D= 16 KN/m^3) and Poisson ratio equal to (0.2).

3.1.3. Meshing of Model A

For the 3D Solid deformable parts, the meshing is done using hexahedral element. Meshing done with this type of element is easy to modify and we need lesser amount of elements to solve the problem. However, for shell elements Quadrilateral 4-noded element is used. It has certain advantages over triangular elements which is also termed as

CST (constant strain triangle). One of the advantage is that; Quad elements are always accurate compared to triangular elements because displacements are interpolated to a higher degree in quadrilateral elements than in triangular elements. Moreover, triangular elements does not work under conditions of high stress or strain gradient. Finally, the model with mesh is shown in the figure;

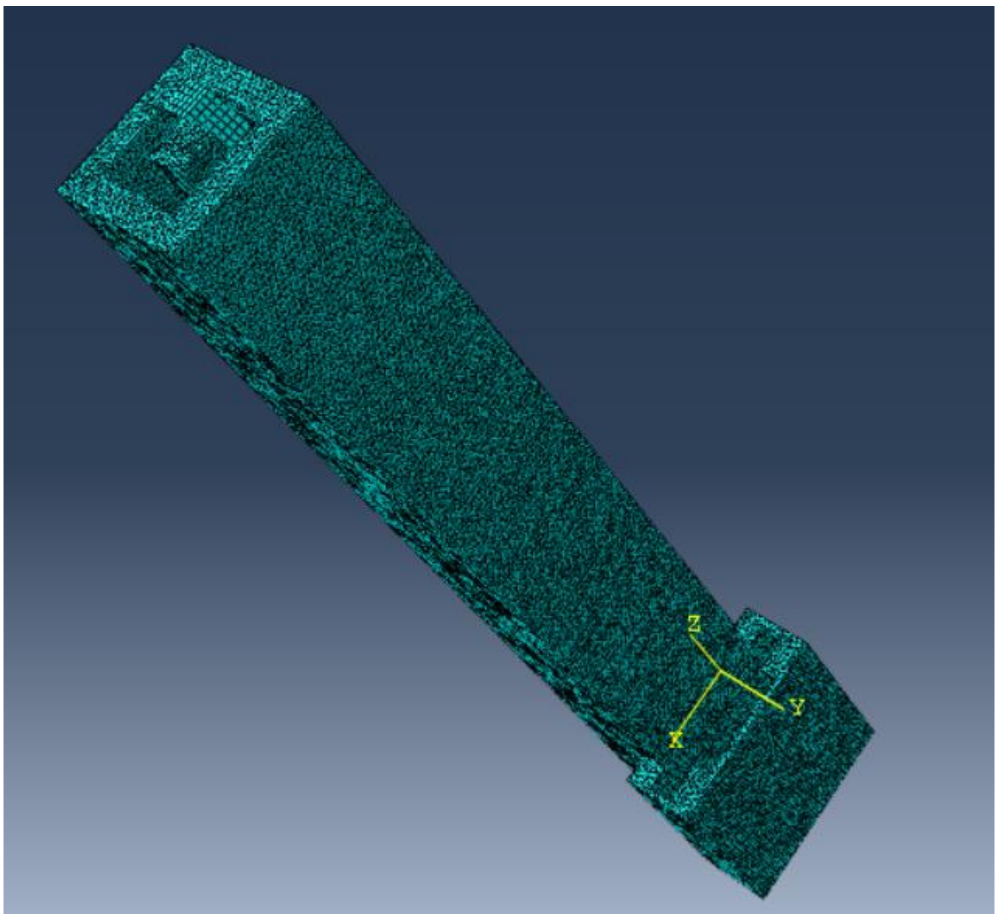


Figure 3.5. Meshing of Model A

The meshing is uniform all over the model which shows that the results will also be accurate. The meshing produces 10324 elements and 732498 nodes.

3.1.4. Components of Model B

In this model, only the outer boundary of the minaret is considered. The central pillar and stairs are not considered for this model. This model also incorporates the window or ventilators openings. The reason for all these considerations is to compare the response of the two different models as it can be stated that the Model B have openings and do not have contribution from stairs and central pillar which makes its mass contribution lesser and hence this effect can be very useful to study the comparison of seismic behavior of two different models.

The outer boundary is modelled as 3D solid having homogenous material. All other features of this model are as same as the Model A. It is modelled with tetrahedral element because of presence of openings in the model. Tetrahedral elements perform better when we have complex geometry and hence it does not produced distortion of mesh. The meshing produces 51405 elements and 411320 nodes.

The Size of openings is explained as follows;

The circular openings have radius of about 225 mm. The bigger rectangular opening have size of 1325 mm $*$ 900 mm, while the smaller rectangular opening is 530 $*$ 430 mm.

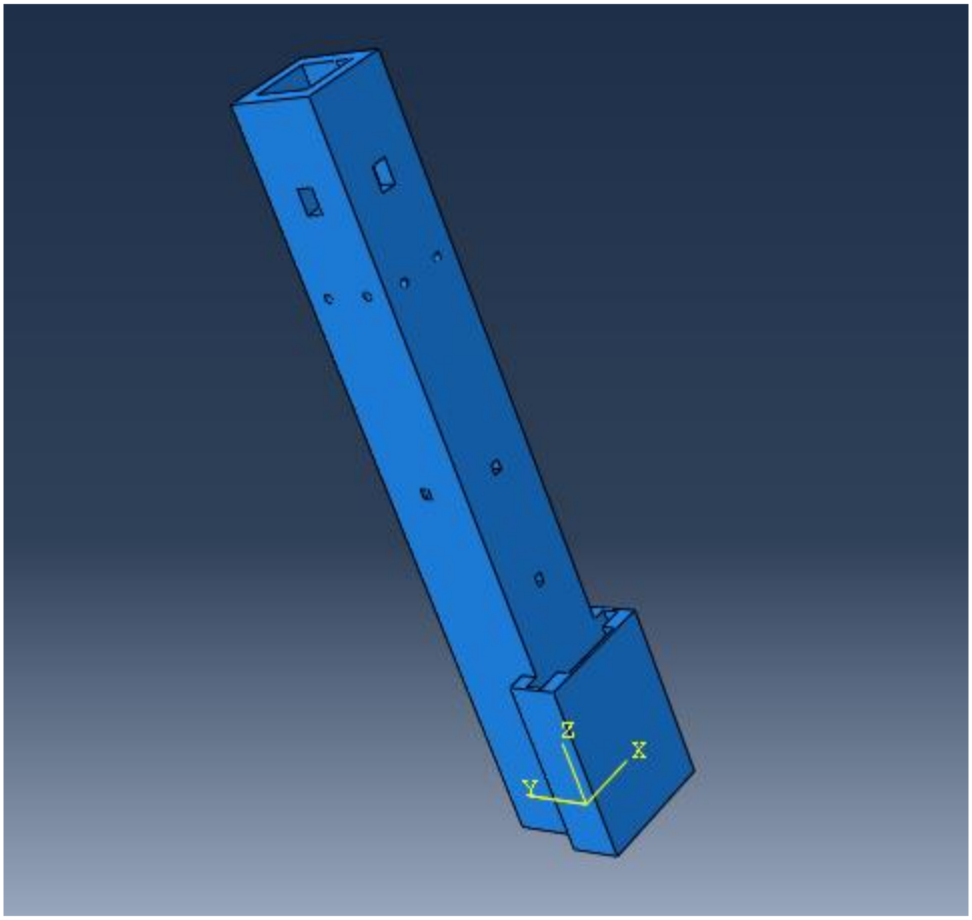


Figure 3.6. Modelling of Model B

3.2. Model Analysis

Model analysis is the study of those properties of a structure which are relating to dynamics of a structure. It measures the response of the structures or fluids in the frequency domain during excitations. The goal of modal analysis is to figure out the modal shapes and frequencies or time periods corresponding to modal shapes. It measures the response under free vibrations of the system. Moreover, it uses mass and stiffness of the system to calculate to find out the natural periods of the structure. The natural period of the system is very important to know in earthquake engineering because it gives information about the resonance condition. Additionally, it also gives idea about the expected earthquake that may act upon the structure.

3.2.1. Model Analysis of Model B

The model analysis of the model A is performed to check out its model shapes, frequencies etc. For this model, the boundary conditions at the base of the model is considered to be fixed i.e. free from displacements and rotations in all directions. The mass and other geometrical properties like moment of Inertia are shown in the following table,

Boundary Condition	Fixed At base
Total Mass of model	958 TON
I_{xx}	4.60E+11 mm^4
I_{yy}	4.56E+11 mm^4
Izz	9.96E+09 mm^4

Table 3.1. Model Characteristics

Modal analysis results are shown in the following table; the table shows the frequency, time period and generalized mass corresponding to each mode;

Mode No.	Frequency(Cycles/Time)	Time Period(s)	Direction	Generalized Mass
1	0.50153	1.99389867	Y	209.78
2	0.51446	1.943785717	X	208.73
3	2.5939	0.385519874	Y	251.05
4	2.6312	0.380054728	X	231.47
5	2.8267	0.353769413	TORSION	342.18
6	4.9175	0.203355363	Y	371.22
7	5.7835	0.17290568	Y	16.859
8	6.0353	0.165691846	X	281.18
9	6.6571	0.150215559	Y	4.5659
10	7.0709	0.141424713	TORSION	14.991
11	8.4294	0.118632406	TORSION	49.763
12	9.7756	0.102295511	Y	2.7169
13	9.8696	0.101321229	Y	5.3216
14	9.894	0.101071356	X	162.27
15	11.293	0.088550429	TORSION	53.842

Table 3.2. Time Period and Direction of Modes

The corresponding modal shapes are;

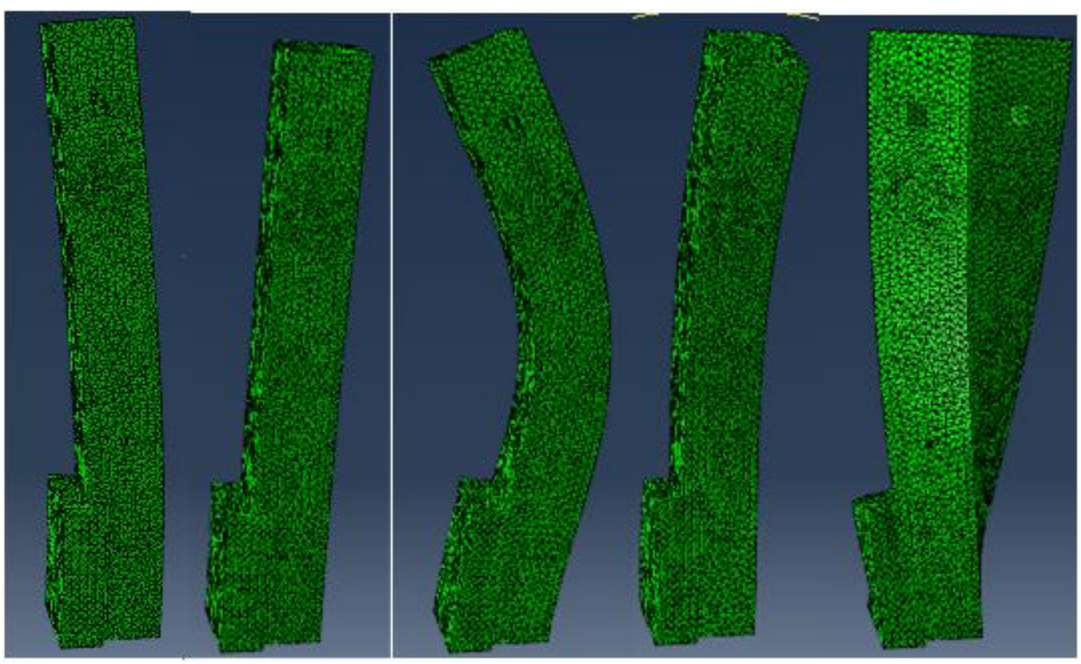


Figure 3.7. Mode Shapes of 1-5 Modes

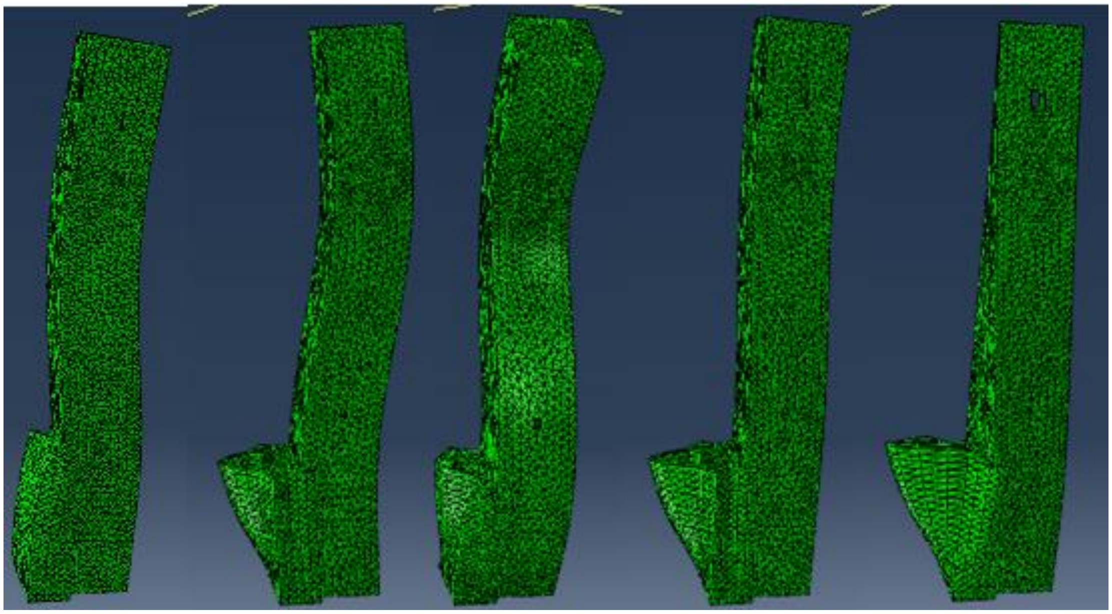
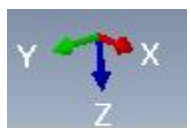


Figure 3.8. Mode Shapes of 5-10 Modes



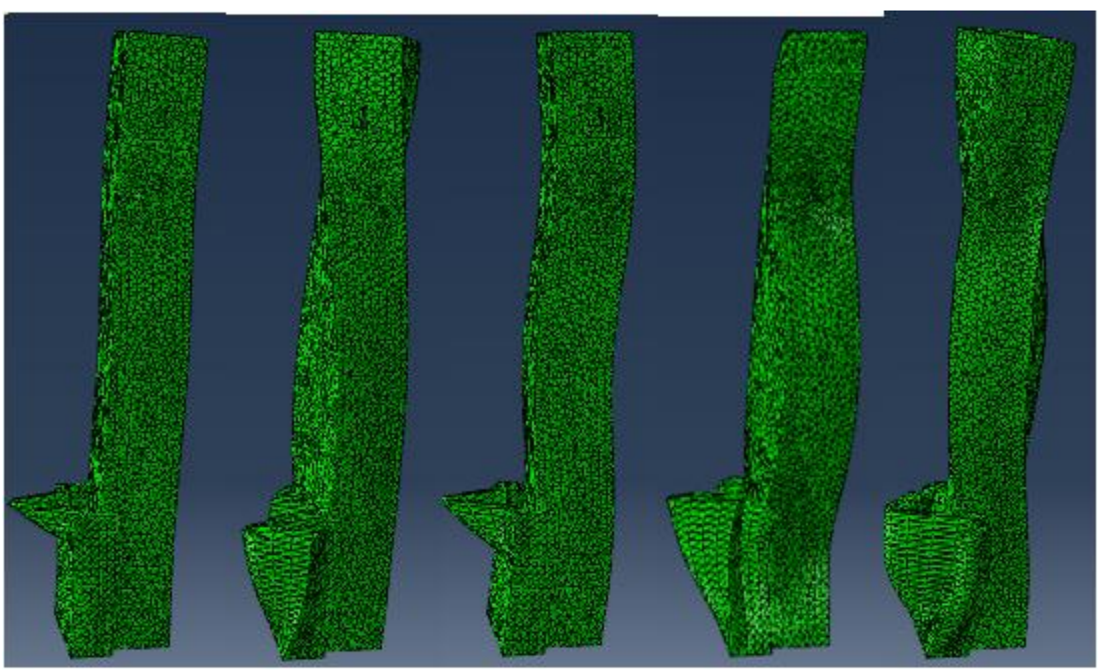


Figure 3.9. Mode Shapes of 10-15 Modes

Unfortunately, there is no experimental data available to compare the model analysis results. However, the main time period which is the first period of the minaret can be compared with an analytical expression which is defined below. This formula calculates the first Eigen value corresponding to first mode shape of the structure. This analytical expression considers a beam which is fixed at one end for its formulation so this case is quite relevant to the case of minaret which is also fixed at one end. It is taken from the book titled as "Advanced Dynamics of Mechanical Systems" by Federico Cheli and Giorgio Diana.

$$\omega_n = K_n \left(\frac{n^2\pi^2}{L^2}\right)\sqrt{\frac{EJ}{\rho A}}$$

Where;

n = Number of mode considered in our case its value is equal to 1.

L = Length of the Minaret

E = Elastic Modulus of the material

J = Moment of Inertia of the cross-section of the minaret

A= Cross-sectional area of the Minaret

ρ = Density of the material

K_n= A coefficient that take into account boundary conditions, in our case value is equal to 0.36.

The values of all these factors are shown in the table;

Area of Cross-section	**13.9896 m^2**
Elastic Modulus of material	**9.00E+08 N/m^2**
Density of Material	**1.60E+03 N/m^3**
Moment of Inertia of Cross-section	**4.00E+01 m^4**
Boundary Condition Coefficient (K)	**0.36**
Number of mode (n)	**1**

The formula gives value of the Eigen value corresponding to first mode shape. From the Eigen value we can find frequency and time period through the following formulation;

$$\omega = \frac{2\pi}{T} = 2\pi f$$

$$f = \frac{1}{T}$$

Hence, using all these formulation, the following results are obtained corresponding to first mode shape of the minaret;

Eigen Value	**2.87923 cycle/second**
Frequency	**0.45848 cycle/second**
Time(s)	**2.18114 seconds**

Table 3.3. Analytical Value of the Time Corresponding to 1st mode

The difference between the time period from the analytical formula and numerical modelling is 8% so the model analysis is considered to be authentic.

Another check which can be applied on the model analysis to check its authenticity. It is related to the number of modes considered for the model analysis. The total number of modes which are considered for modal analysis are fifteen (15). As per the Literature, the minimum number of modes require to completely incorporate the modal response of the structure depends upon the percentage effective masses of the structure along each direction i.e. x, y or z. The percentage effective masses of the model along each direction should be more than 90%.

The effective masses corresponding to all directions are shown in the following table;

Mode Number	X-COMPONENT	Y-COMPONENT	Z-COMPONENT
1	0.0000885	517.54	0.0754
2	515.95	0.0000852	0.00000243
3	0.00304	206.3	2.1366
4	202.54	0.00385	0.000434
5	0.98584	0.00788	0.000113
6	0.001444	0.0769	690.94
7	0.0001611	102.99	13.957
8	87.643	0.0000402	0.005811
9	0.00000643	0.59106	0.0017411
10	3.3298	0.00167	0.0000221
11	7.8244	0.0000854	0.00000339
12	0.0004299	0.88797	0.08217
13	0.39166	37.745	0.00814
14	39.377	0.37938	0.000747
15	0.051	0.00135	0.00035
SUM	858.0978699	866.5252708	867.56739
% of total mass	89.57180271	90.45148965	90.46754
Check	OK	OK	OK

Table 3.4.Effective Mass Contribution Factors

The effective mass contribution corresponding to Moment of Inertia are shown in the following;

Mode Number	X-ROTATION	Y-ROTATION	Z-ROTATION
1	4.37E+11	80536	6.63E+05
2	75697	4.36E+11	2.48E+09
3	1.73E+10	3.06E+05	1.7783
4	3.56E+05	1.69E+10	1.14E+09
5	6.50E+05	7.84E+07	3.93E+09
6	2.52E+09	1.52E+06	2636.3
7	2.00E+09	41254	2.61E+05
8	11715	2.452	2.29E+08
9	4.80E+07	253.45	21652
10	28347	1.4836	2.17E+08
11	1403.3	2.0689	5.66E+08
12	5.68E+07	6050.3	15.765
13	2.94E+08	4.36E+06	6.62E+05
14	2.79E+06	4.28E+06	6.40E+07
15	13640	1.41E+07	3.30E+08
SUM	4.59223E+11	4.52983E+11	8951880356
% of M.O.I	99.83102757	99.33839717	89.87831683
Check	OK	OK	OK

Table 3.5.Effective Mass Contribution Factors

As it can be seen that percentage effective mass contribution along each direction is greater than 90% which shows that 15 number of modes are enough to capture the seismic response of the minaret.

The mass participation of each mode towards total response is shown in the following table;

Mode Number	X-COMPONENT	Y-COMPONENT	Z-COMPONENT	SUM
1	0.000649	1.5707	-0.01896	1.552389
2	1.5722	-0.0006389	0.000108	1.571669
3	-0.00348	-0.9065	0.0922	-0.81778
4	-0.93542	0.00408	-0.001369	-0.93271
5	-0.0536	-0.00408	0.000576	-0.0571
6	-0.00197	-0.01439	1.3643	1.34794
7	-0.0025	2.4716	0.90987	3.37897
8	0.5583	0.000378	0.00454	0.563218
9	0.001437	-0.35979	0.0195	-0.33885
10	-0.4713	-0.01058	-0.00121	-0.48309
11	0.39653	-0.00131	0.0002611	0.395481
12	-0.01257	0.57169	0.17391	0.73303
13	0.27129	-2.6632	0.0391	-2.35281
14	-0.49262	-0.04835	-0.00214	-0.54311
15	0.0307	0.005011	-0.00255	0.033161

Table 3.6. Mass Participation towards each Mode

From the mass participation factor, it can be seen that the mode number 2 contributes more than any other mode to the overall response of the structure.

3.2.2. Model Analysis of Model A

The boundary condition, mass and geometrical properties of the model A are shown in the following table;

Boundary Condition	Fixed At base
Total Mass of model	1014 TON
I_{xx}	5.60E+11 mm^4
I_{yy}	5.56E+11 mm^4
Izz	10.96E+09 mm^4

The results from model analysis are shown which includes frequency, time periods corresponding to first 15 modes of the structure;

Mode No.	Frequency(Cycles/Time)	Time Period(s)	Direction	Generalized Mass
1	0.50091	1.996366613	Y	234.66
2	0.51517	1.941106819	X	233.09
3	2.6803	0.373092564	Y	264.36
4	2.7268	0.366730233	X	264.25
5	2.9543	0.338489659	TORSION	356.64
6	4.9571	0.201730851	Y	437.32
7	6.211	0.161004669	Y	17.702
8	6.3591	0.157254957	X	295.28
9	6.747	0.148214021	Y	2.7474
10	8.0524	0.124186578	TORSION	13.891
11	9.395	0.106439596	TORSION	17.876
12	10.219	0.097856933	Y	91.14
13	10.376	0.096376253	Y	271.28
14	10.6	0.094339623	X	1.1766
15	12.743	0.078474457	TORSION	21.016

Table 3.7. Time period and Direction of Modes

It can been from the table that the time period corresponding to first mode shape is almost as same as the model B. This is due to the fact that mass contribution by stairs and central pillar is not significant. There is difference of 56 ton of mass between two models. Moreover, there is no significant difference between the mode shapes, effective masses and participation factors between the two models.

3.2.3. Model Analysis considering connection with other buildings

The model analysis of the model was also performed considering connection of the model with other parts of the structure. As minaret is part of the mosque, so there are other structures which are connected to minaret up to the height where the main body of the minaret starts. Due to unavailability of information, the connections of these parts is considered to be rigid and fixed. So the boundary conditions were changed for this model and are shown in the figure.

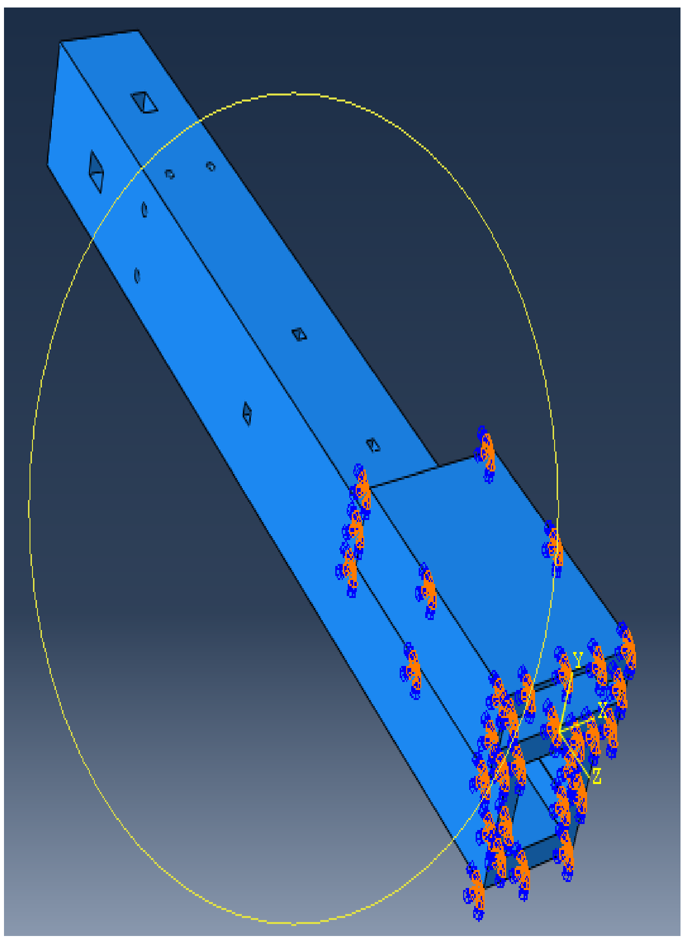


Figure 3.10. Model Having Restrained from surrounding Structure

In the previous models, the boundary conditions were considered only at the base of the model. But for this model, the sides of the minaret were also kept fixed up to the height of the lower body to incorporate the effect of other structures.

Mode No.	Frequency(Cycles/Time)	Time Period(s)	Direction
1	0.57735	1.732051615	Y
2	0.6161	1.623113131	X
3	3.0305	0.329978551	Y
4	3.1056	0.32199897	X
5	3.4085	0.293384187	TORSION
6	5.3689	0.186257893	Y
7	7.2367	0.138184532	Y
8	7.3384	0.136269487	X
9	9.3664	0.106764605	Y
10	9.6673	0.103441499	TORSION
11	10.88	0.091911765	TORSION
12	11.881	0.084167999	Y
13	11.411	0.087634738	Y
14	11.716	0.085353363	X
15	11.967	0.083563132	TORSION

Table 3.8. Time period and Direction of Modes

The time period considering this condition reduces from 1.99 s to 1.73 s and the difference from analytical calculated time also increases i.e. 20 %. Hence it is not recommended to use this model for further calculation because of two reasons. Firstly, the difference increases from 8 % to 20 %, secondly the decrease of time period shows that the stiffness of the model has increased. Considering that the main task is to increase the seismic response of the structure, so it is better to use the model B with fixed condition at base which have lesser stiffness.

The following formula shows this debated that stiffness increases as time period decreases;

$$f_{\mathrm{n}} = \frac{1}{2\pi}\sqrt{\frac{k}{m}}$$

Where;

m = Mass of the structure

k = Stiffness of the structure.

4. Seismic Analysis of Minaret

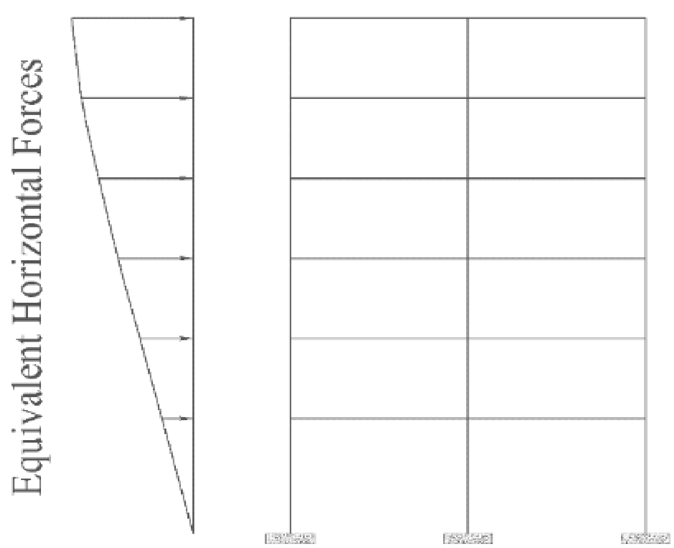
In this chapter, the results and discussions on the seismic analysis of the minaret are explained. Seismic analysis is defined as the analysis which is performed on the structure when it is subjected to earthquake excitations. It is usually performed during structural design, structural assessment and structural retrofitting of buildings which are subjected to or highly prone to earthquake excitations.

4.1. Types of Seismic Analysis

The seismic analysis is categorically divided into five different types;

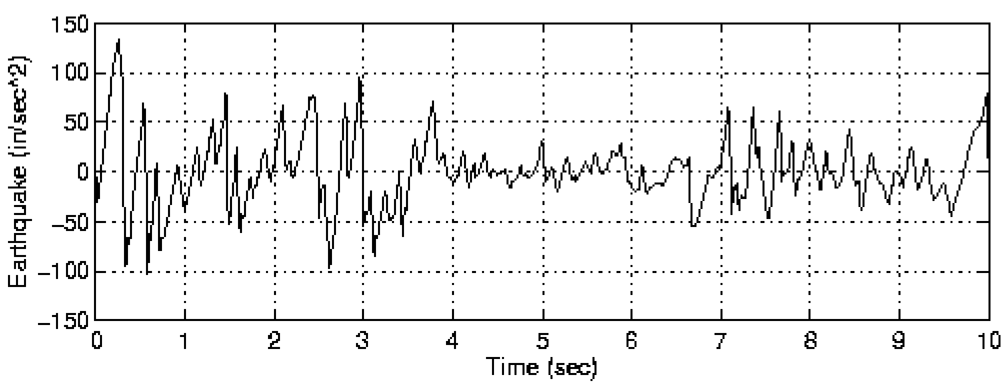
a) Equivalent Static Analysis

This type of analysis considers a series of horizontal forces to be applied on the building which considers to be the earthquake ground motion. It assumes the building to be vibrate in its fundamental mode. For this condition to be satisfied, the building considered should be low rise and does not twist when subjected to ground motion. The input is of the following type;



b) Linear dynamic analysis

Static analysis as defined above is considered to be effective when higher modes are not considered. However, in linear dynamic analysis the higher modes can be considered and hence analysis can be applied on the high rise buildings. In this type of analysis, the building can be considered to Multi degree of freedom system (MDOF) having linear elastic stiffness matrix. The input considered can be of model spectral type or time history. Irrespective of the input, the forces and displacements are calculated using linear elastic analysis. It gives the response of the system in time domain. However, its applicability starts decreasing as the non-linear behavior is considered. The input is of the type;



c) Non-Linear static analysis/Static Push-over

Linear static analysis are taken into account when the structure remain in the elastic regime during the duration of earthquake excitation. But when the structure moves to non-linear regime, we need to consider non-linear static analysis. It considers the same distribution of forces as the linear static analysis but it consider the non-linear properties of the materials. It considers the structure as SD0F system.

d) Non-linear Dynamic Analysis

This type of analysis uses detailed structural model with earthquake ground motion to calculate the response of the structure. In this type of analysis, the structure can be considered as multi degree of freedom system (MDOF). The materials properties are non-linear and are considered in the domain of time history. It calculates response with low uncertainty.

e) Response Spectrum Analysis

In this, multiple modes of the response of the structure are taken into account. The analysis is performed and gives result in frequency domain. It can be used for both linear and non-linear material behavior. The response of the structure can be considered to be the sum of responses from different modal shapes or modes of the structure. The information about the modes of the structure can be taken from the modal analysis of the structure which is performed using computer software. The methods which are used for the combination of the response of the modes are as follows;

1) Absolute; in this peak values of the response of all the modes are added together.
2) Square Root of the sum of Squares (SRSS)
3) Complete Quadratic Equation (CQC); this method is extension of SRSS for closely spaced modes.

The response from the response spectrum using response spectrum input is different as compared to the linear dynamic analysis as phase is lost in describing the response spectrum input. It is not good practice to use this method for irregular structures.

For the seismic analysis of the minaret which is a regular structure, Response spectrum analysis is used and the combination method is square root of sum of squares (SRSS).

4.1.1. Response Spectrum Input

The input considered for the Response Spectrum analysis have regular shape. It have spectral acceleration on y-axis and time period on x-axis. The typical response spectrum input corresponding to Unified Building Code (UBC-1997) is shown,

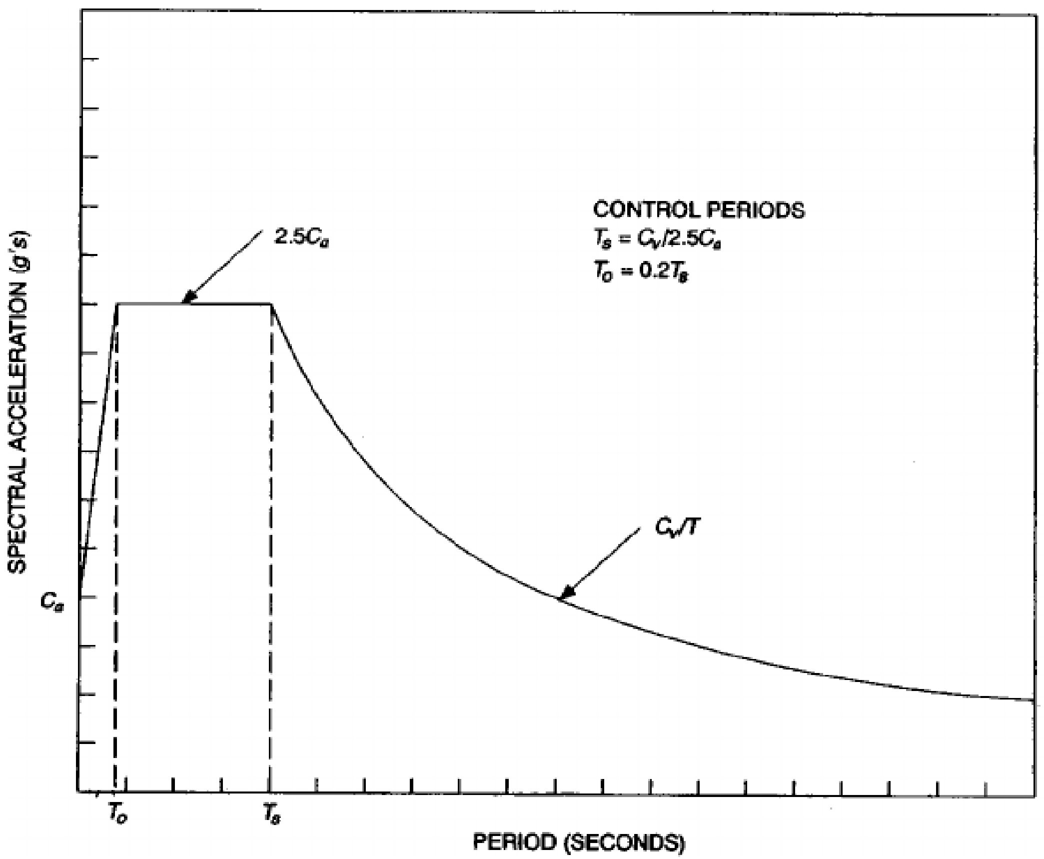


Figure 4.1. Response Spectrum by UBC

Where;

T_o= Time corresponding to start of constant acceleration plateau

T_s= Time corresponding to end of constant acceleration plateau

The input have three different regions; the first region starts from the value of acceleration equal to C_a and remains linear with the time period up to time T_o, the second region corresponds to the constant acceleration having value equal to $2.5C_a$ which starts from time T_o and ends at time T_s. The last region has parabolic change of acceleration with respect to time. It changes with time according to the relation C_v/T.

Due to unavailability of the date conforming to the seismic characteristics of Syria, the reference is made to the document from the U.S geological survey response (USGS) which incorporates the geological and seismic data for all the significant sites of the world. The document gives information about the seismic zone and type of soil for the place considered. Hence from USGS, it is derived that the seismic zone of city of Aleppo is 2A and the type of soil is S_E which is considered to be the soft soil.

This data can be used to generate the design response spectrum according to unified building code (UBC 1997). From the designation of seismic zone, we can find out the seismic zone factor of the site.

Zone	**1**	**2A**	**2B**	**3**
Z	**0.075**	**0.15**	**0.2**	**0.3**

Corresponding to Seismic zone 2A, the zone factor is 0.15. This factor can be used to find out the seismic coefficients C_a and C_v. For the seismic coefficients, the following tables are available in the UBC as well as Syrian seismic code;

Soil Profile Type	Seismic Zone Factor, Z				
	Z = 0.075	Z = 0.15	Z = 0.2	Z = 0.3	Z = 0.4
S_A	0.06	0.12	0.16	0.24	0.32 N_v
S_B	0.08	0.15	0.20	0.30	0.40 N_v
S_C	0.09	0.18	0.24	0.33	0.40 N_v
S_D	0.12	0.22	0.28	0.36	0.44 N_v
S_E	0.19	0.30	0.34	0.36	0.36 N_v
S_F	See Footnote[1]				

[1] Site-specific geotechnical investigation and dynamic site response analysis shall be performed to determine seismic coefficient for Soil Profile Type S_F

Table 4.1. Values of C_a for different values of Seismic Zone Factor

Soil Profile Type	Seismic Zone Factor, Z				
	Z = 0.075	Z = 0.15	Z = 0.2	Z = 0.3	Z = 0.4
S_A	0.06	0.12	0.16	0.24	0.32 N_v
S_B	0.08	0.15	0.20	0.30	0.40 N_v
S_C	0.13	0.25	0.32	0.45	0.56 N_v
S_D	0.18	0.32	0.40	0.54	0.64 N_v
S_E	0.26	0.50	0.64	0.84	0.96 N_v
S_F	See Footnote[1]				

[1] Site-specific geotechnical investigation and dynamic site response analysis shall be performed to determine seismic coefficient for Soil Profile Type S_F

Table 4.2. Values of C_v for different values of Seismic Zone Factor

From the table, the values of co-efficient C_a and C_v are 0.30 g and 0.50 g respectively. These values can be used to get the Elastic response spectrum corresponding to the city of Aleppo. The design response spectrum is shown below;

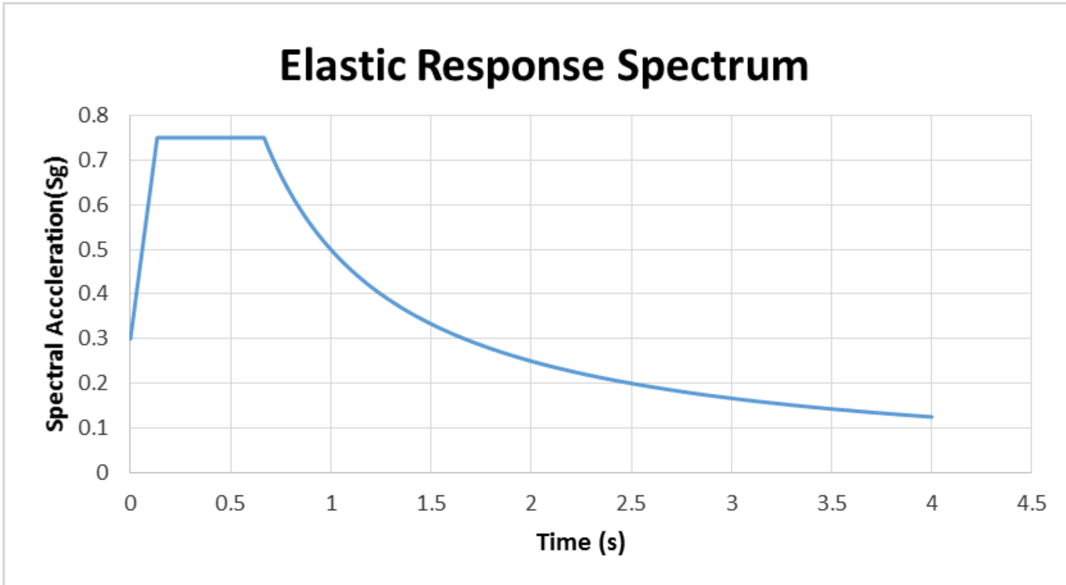
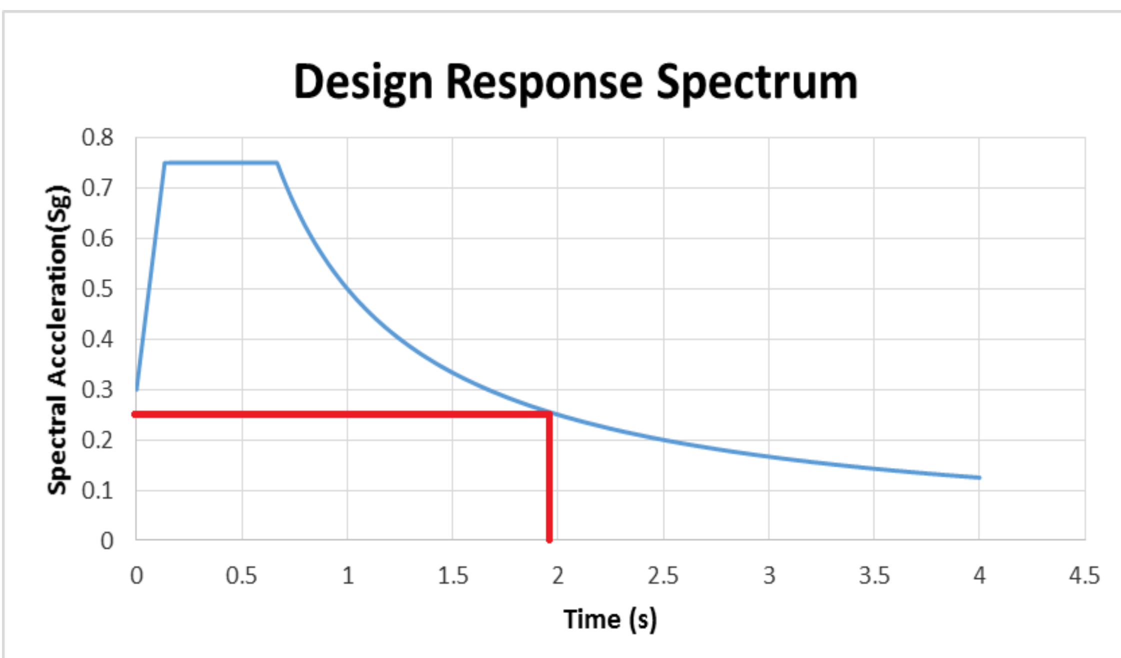


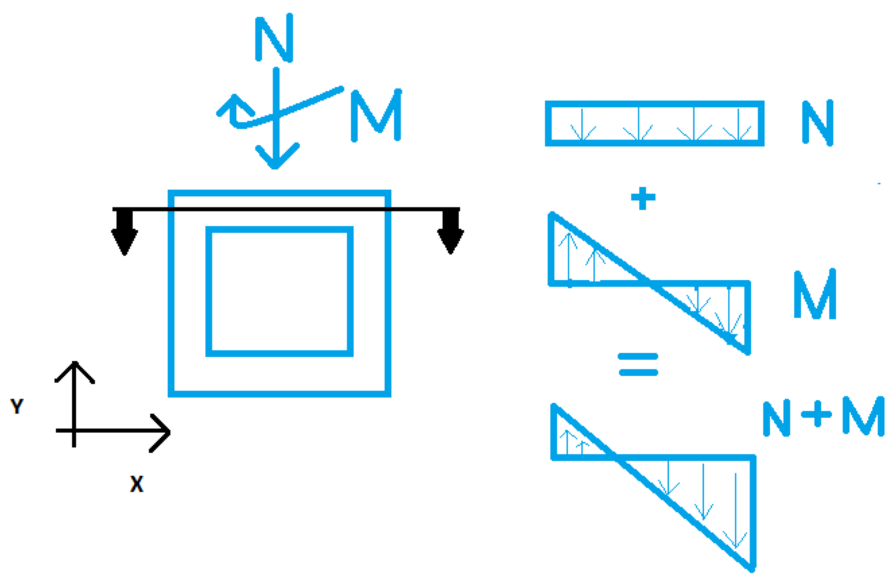
Figure 4.1. Elastic Response Spectrum

Hence, it can be seen in the design response spectrum that the value of PGA is equal to 0.30 g. Moreover, the main period of the model (1.99 seconds) lies in the region of design response spectrum which have value of spectral acceleration lesser than PGA (<0.3g), so it can be said that the model is in safe condition. This condition is shown;



4.2. Analysis of the Model

The main concern of this topic is to study the behavior of minaret under seismic excitations and self-weight of the structure. Therefore, the model under consideration is subjected to self-weight of the structure and design response spectrum mentioned above. This problem is as same as a hollow column is subjected to axial force and bending moment. The cross-section of the column under self-weight of the structure produces constant distribution of normal stresses which are compressive stresses. While, on the other hand, bending moment produces tension on one side and compression on the other side.



Therefore, two types of analysis are performed in order to understand the behavior minaret.

4.2.1. Gravity Analysis

The gravity analysis of the minaret is performed assuming only the self-weight of the minaret acting on it. The distribution of normal compressive stresses is constant throughout the cross-section. However, the distribution of stresses on the minaret starts decreasing as we move from the base of the minaret to the top. There are two types of cross-sections which are considered; one at the base of the minaret (at 0 m level) and other at the point of junction of two different cross-sections (at 7.380 m). The maximum stresses occur the base cross-section. The distribution of normal stresses along the minaret due to gravity analysis is shown in the following figure;

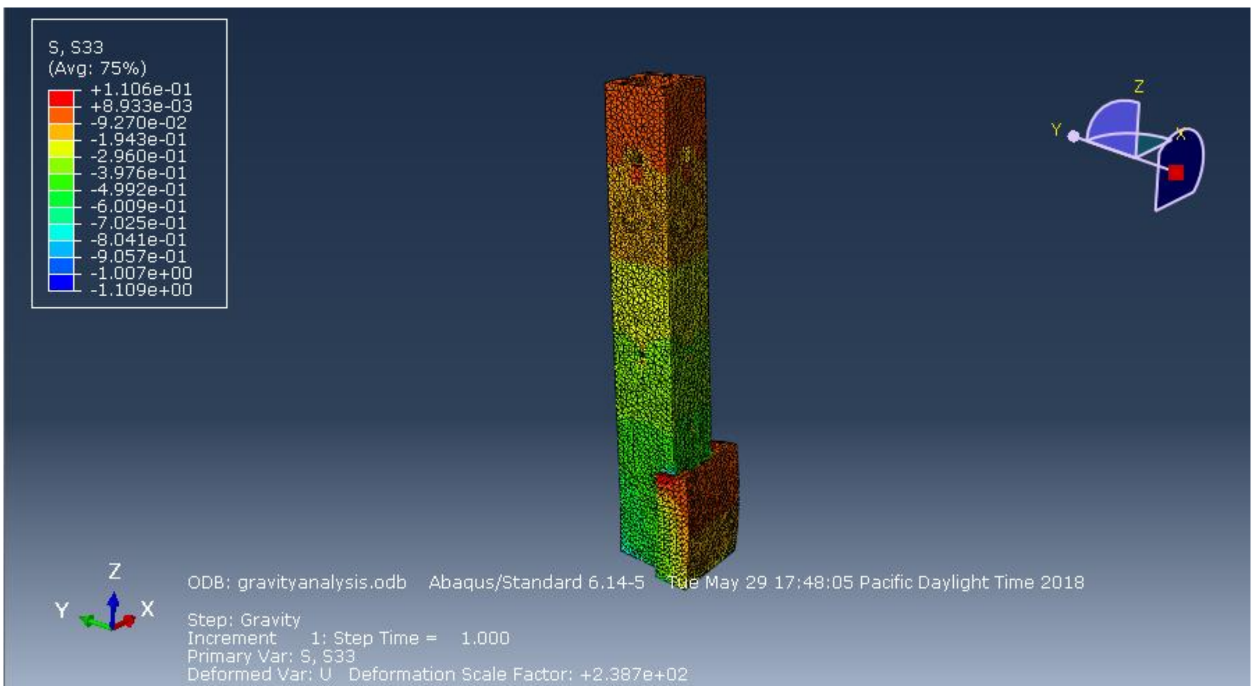


Figure 4.2. Normal Stress Distribution under Gravity Load

The distribution of the stresses at the base and the cross-section at junction is;

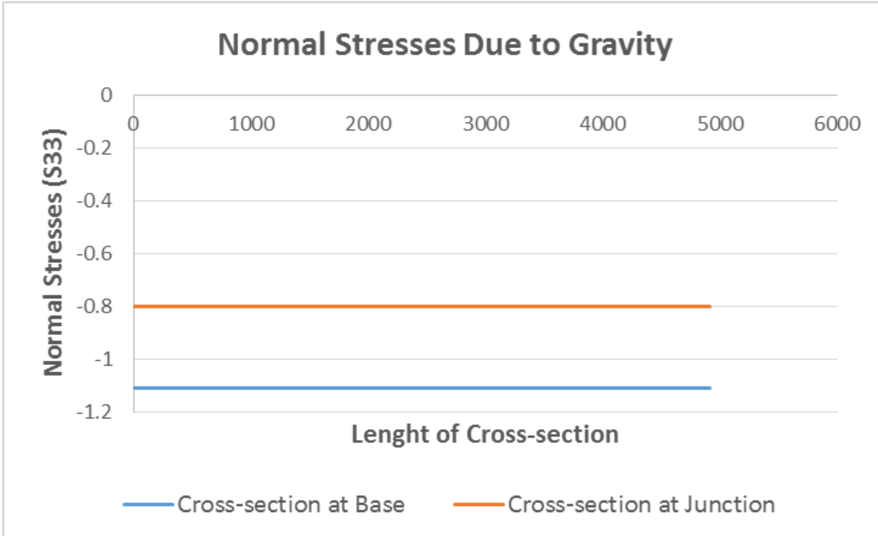


Figure 4.3. Distribution of Normal Stresses Due to Gravity

4.2.2. Response Spectrum Analysis

The response spectrum analysis is performed to find out the distribution of normal stresses. The above mentioned design response spectrum is used for the analysis and it is applied along X-direction. The response spectrum loading produces bending in the minaret. It yields tension on one side and compression on the other. The distribution of stresses along the minaret is presented;

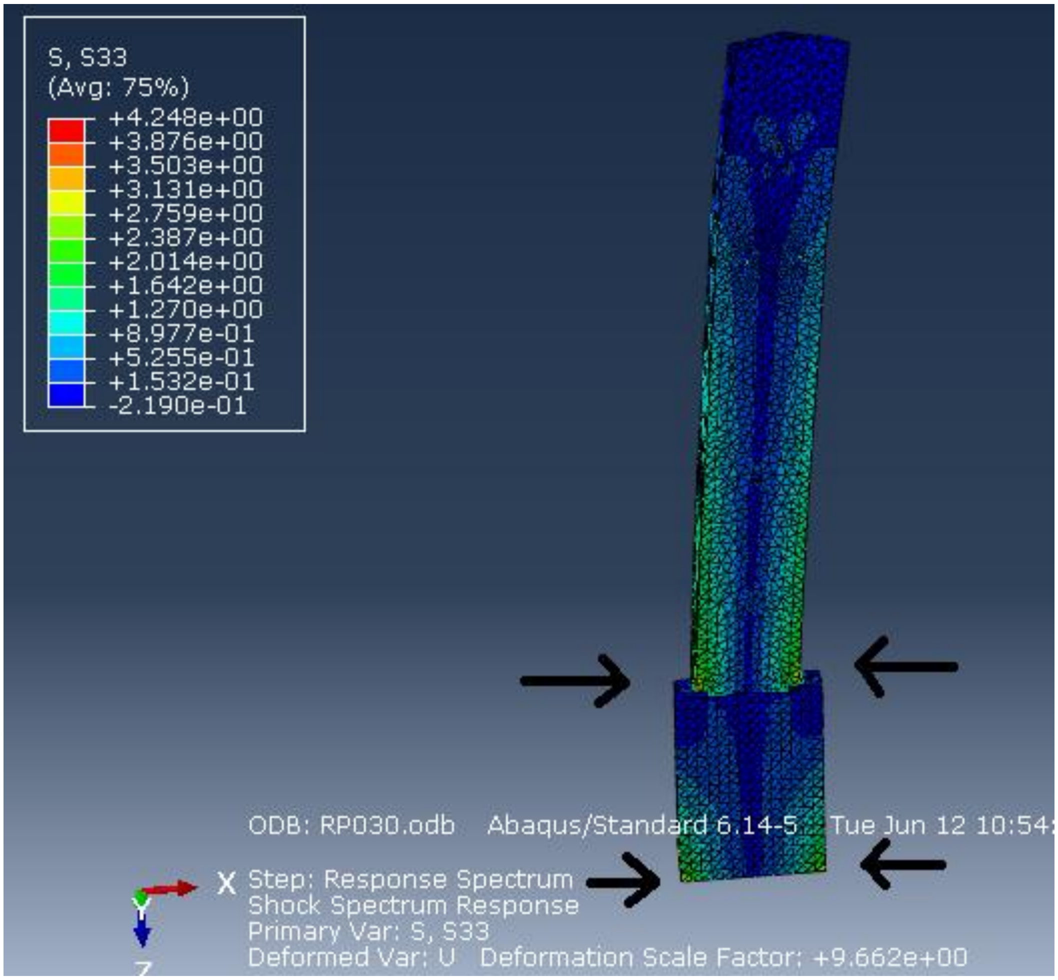


Figure 4.4. Normal Stresses Due to Response Spectrum

As it can be clear from the above figure that the response spectrum produces tension on one side and compression on the other. Moreover, the distribution of stresses starts decreasing as we move from the base of the minaret towards the top. The distribution of bending stresses across the cross-sections at base and junction are shown in the following graphs;

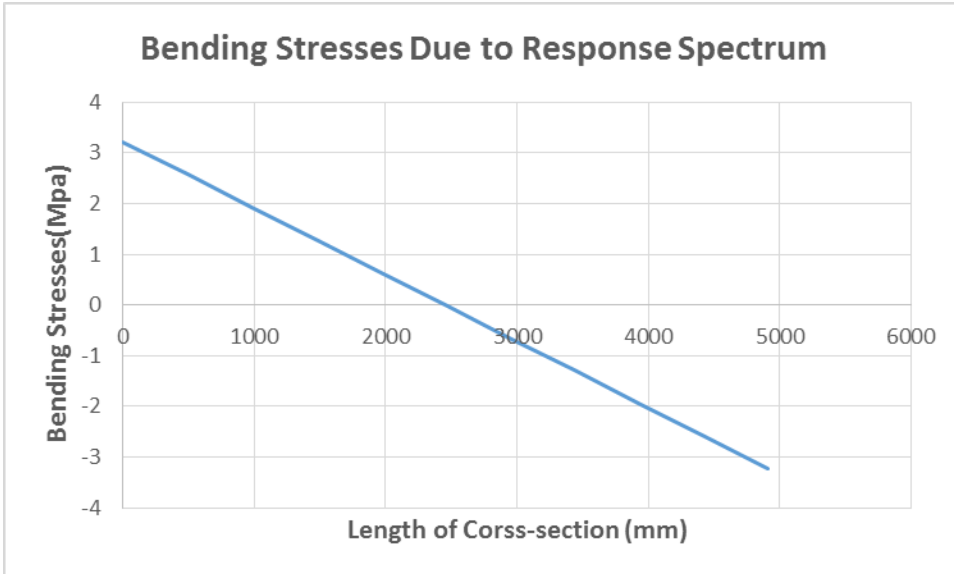


Figure 4.5. Bending Stresses Profile due to Resonse Spectrum at Base Cross-section

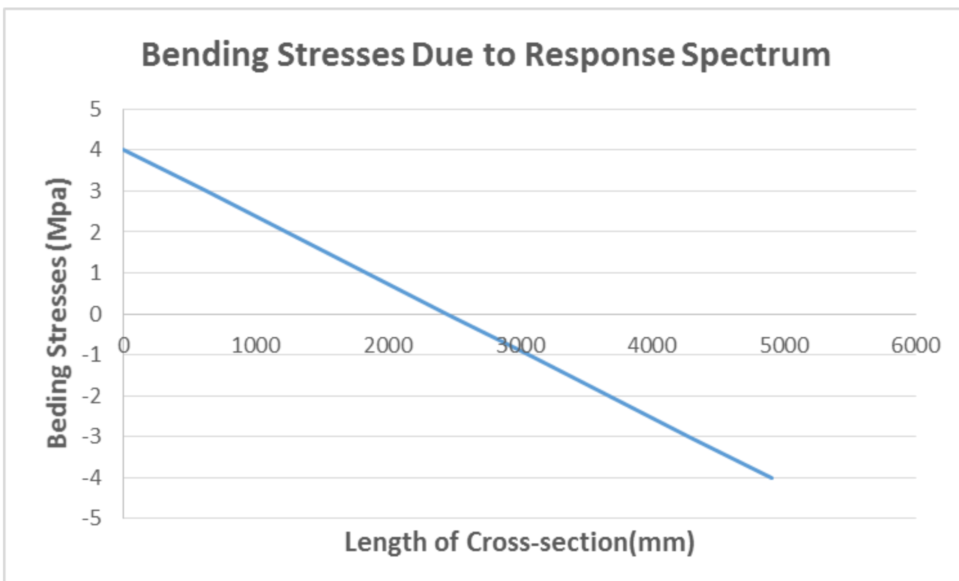


Figure 4.6. Bending Stresses Profile due to Resonse Spectrum at Cross-section of Junction

It can be seen from the graphs that maximum value of the bending stresses occur at the cross-section at junction. However, we will not limit our discussion to this cross-section only. Now, the total distribution of stresses is very important to know because this distribution will give information about the maximum tensile and compressive stresses in the cross-section.

The distribution for the combination of both self-weight and response spectrum for both cross-section is depicted;

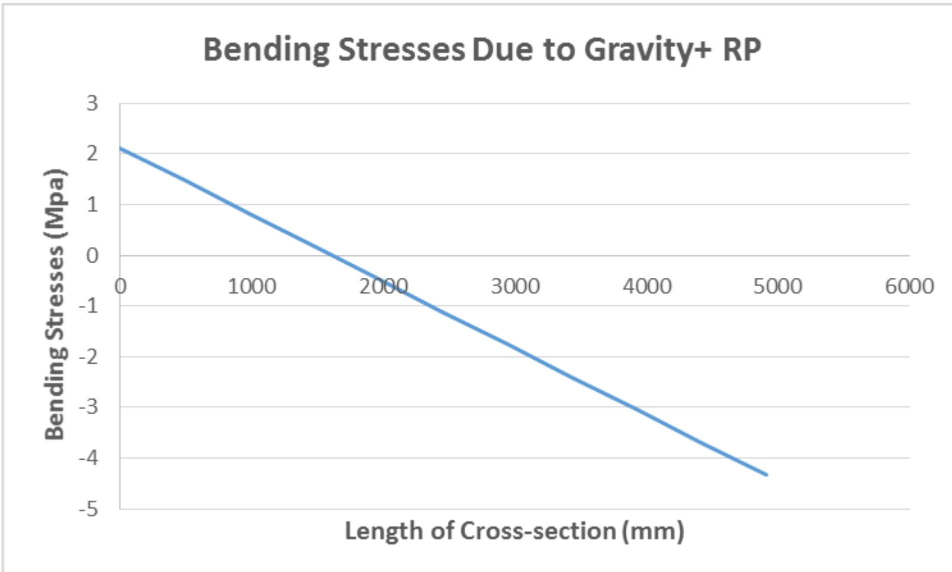


Figure 4.7.Bending Stresses due to Resonse Spectrum and Gravity at Base Cross-section

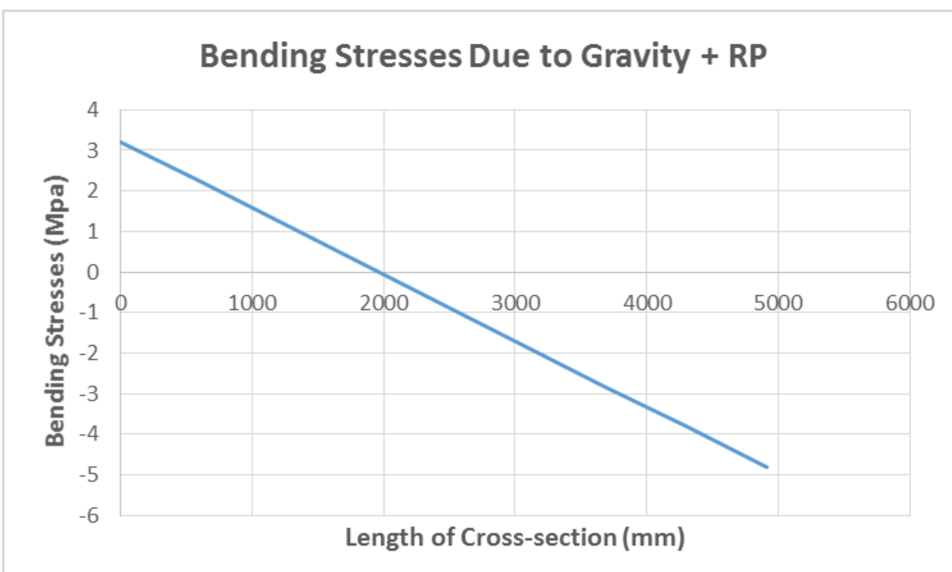
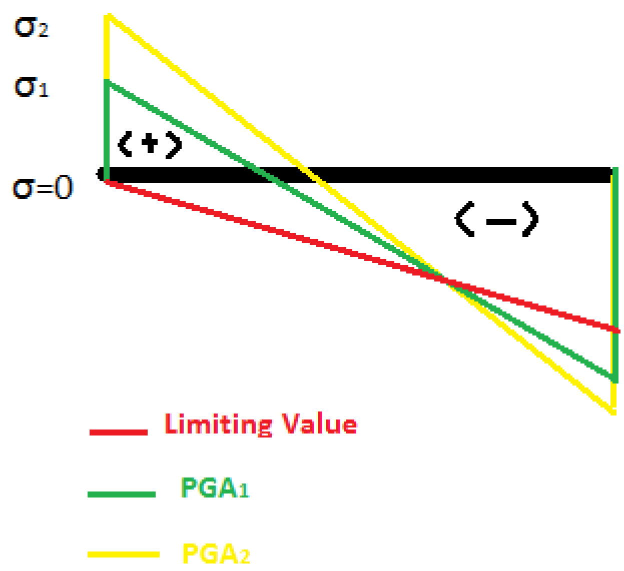


Figure 4.8.Bending Stresses due to Resonse Spectrum and Gravity at Cross-section of Junction

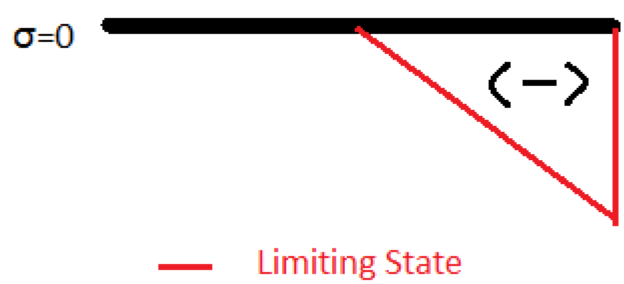
As it can be evident from the graphs that maximum stresses produced at the cross-section of junction. The maximum value experienced by cross-section at base is 3.032 Mpa while the maximum value of the stress in case of cross-section at junction is 4.01 Mpa. Therefore, the cross-section at junction is critical as compared to the other. Similar kind of results are obtained for model with stairs and central pillar contribution, but the values of stresses are comparatively higher. The maximum value produced at the cross-section at junction is 5.32 Mpa while the maximum value at the same point in the simple model is 4.01 Mpa. The distribution of stresses from the later model will be discussed in the next section.

4.3. Limiting State of the Model under Bending

The results of the gravity and response spectrum analysis shows that at the value of PGA equal to 0.30 g, the tensile stresses remain in the minaret at one end of the cross-sections. The main concern is to find the value of PGA which produces zero tensile stresses in cross-section and the whole cross-section should be under compression. This value of PGA is termed as 'Limiting value' and so this state is named as 'Limiting State' as if there are no tensile stresses in the cross-section, the minaret will fail under the effect of bending due to earthquake excitation. Moreover, the compressive stresses in the cross-section should not increase the compressive strength of the material which is 2.40 Mpa. So the idea is to decrease the value of the PGA until there are zero tensile stresses in the cross-section. Hence the analysis will start from the value of PGA equal to 0.30 g and it will be reduced until there are zero tensile stresses in the cross-section. This concept is depicted below; the main concern is to get the Red profile.



As already mentioned, the analysis in this project is performed in linear elastic regime. However, if the analysis is to be performed in non-linear regime, then different type of limiting state can be achieved, which is shown below;



Under this state, the half part of the cross-section is lost due to non-linear material behavior.

The limiting state is achieved for the both types of models.

4.3.1. Limiting State for Model B

- **For Cross-section at Junction:**

The analysis starts at value of PGA equal to 30% and gradually decreasing value of PGA until the tensile stresses at one end reduces to zero. The liming state is achieved at the value of PGA equal to 6.3%. The graphical representation is depicted;

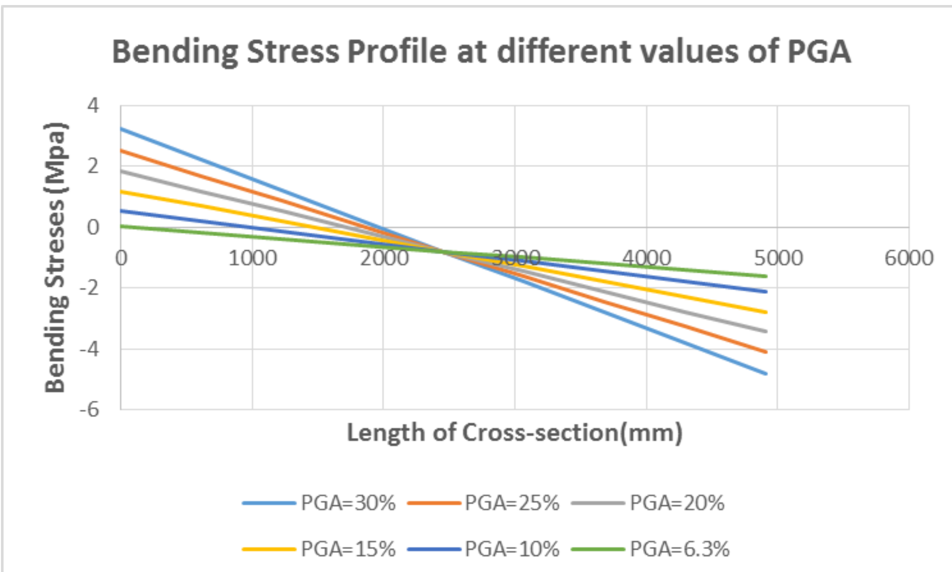


Figure 4.9. Bending Stresses Profile at different Values of PGA

The green colored profile in the graph is the limiting profile of stresses which have almost zero tensile stresses (0.02 Mpa) at one end and compressive stress value of 1.62 Mpa at the other end. It means at limiting stage, the cross-section can withstand maximum stress of 1.62 Mpa. The type of masonry considered for this project have compressive strength of 1.40 Mpa which shows that at limiting stage the value of maximum compressive strength is very close to this value.

- **For Cross-section at Base:**

For cross-section at base, the same procedure is followed and the resulting profiles are;

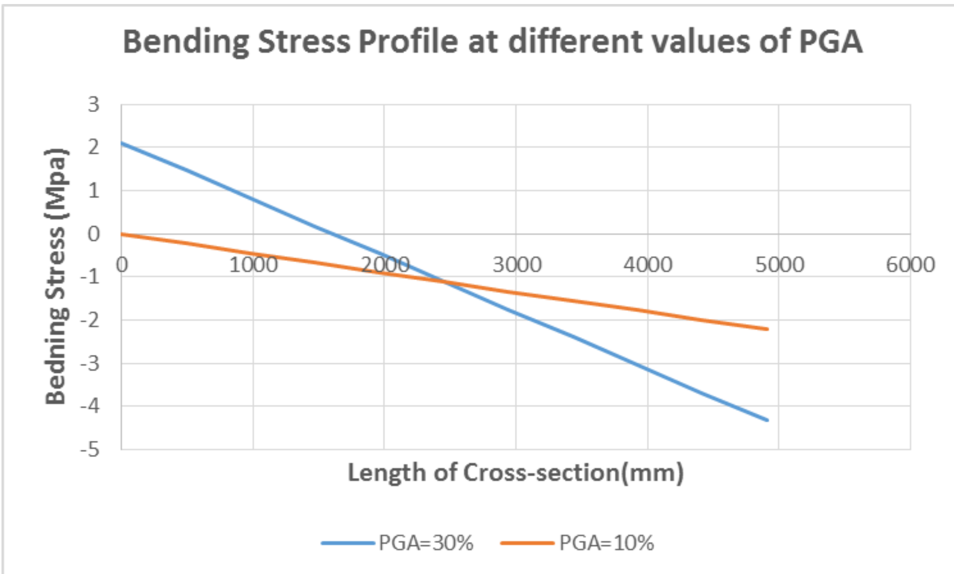


Figure 4.10. Bending Stresses Profile at different Values of PGA

In this case, the limiting value of PGA increased from 6.3% to 10% which shows that cross-section at junction is more critical due to lower value of limiting PGA. The values of maximum compressive stress taken up by this cross-section is greater (2.12 Mpa) as compared to the other cross-section (1.62 Mpa).

4.3.2. Limiting State for Model A

- **For Cross-section at Junction:**

The scattering of bending stresses profile is displayed;

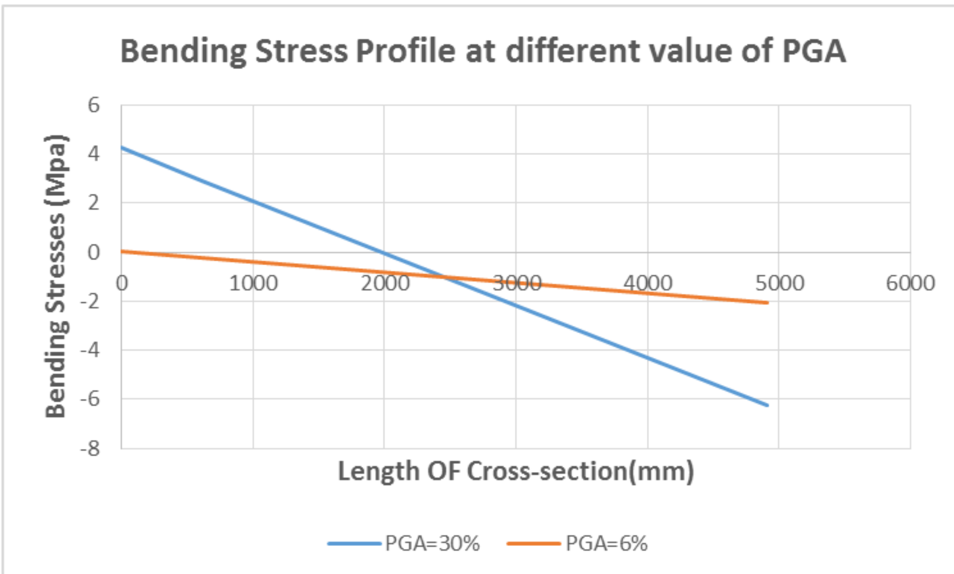


Figure 4.11. Bending Stresses Profile at different Values of PGA

In this case, the value of the limiting PGA reduce to 6% which shows that the cross-section at junction in the case of complete model is under more critical state. But the difference of PGA is not as high as the limiting value in case of simple model is 6.3%. So there is difference of 3% which can be neglected. However, the value of the maximum compressive stress taken up by the cross-section increases from 1.62 Mpa to 2.12 Mpa.

- **For Cross-Section at Base:**

The results in this case are of same nature as found out for cross-section at base of simple model. The value of PGA has very small reduction from 10% to 9.8%. In the same way, the maximum compressive stress at limiting PGA is 3.12 Mpa. The graphical representation is depicted in the following figure;

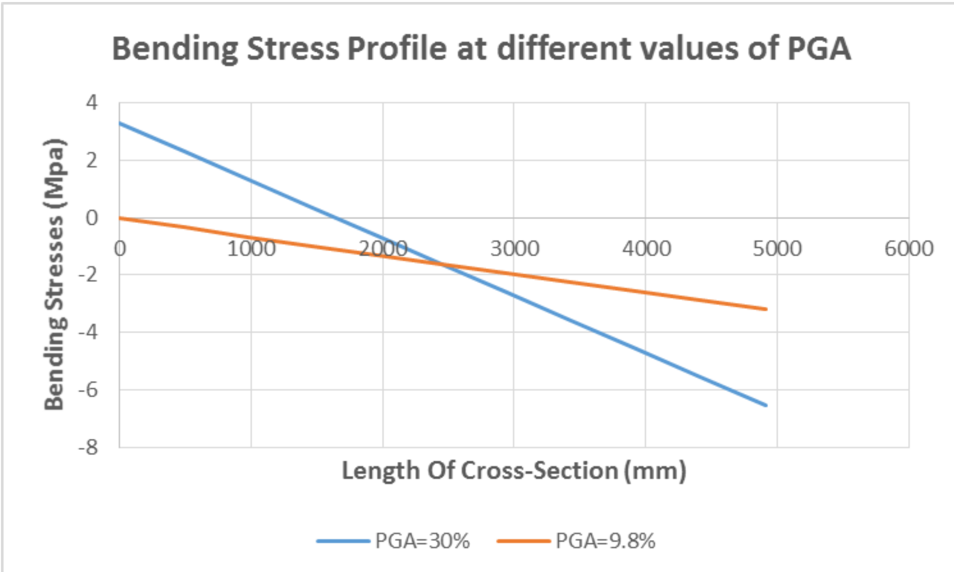
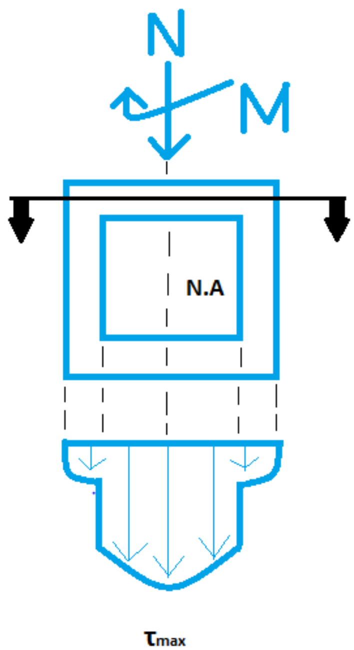


Figure 4.12. Bending Stresses Profile at different Values of PGA

4.4. Limiting State under Shear

The limiting state under shear referred to the state at which the maximum shear stresses produced in the minaret due to response spectrum analysis overcome the value of the maximum shear stress of the material used. The maximum value of the shear which the material can resist is 0.028 Mpa. The general distribution of shear across the hollow cross-section is shown in the following figure;



4.4.1. Limiting Condition for Model B

The response spectrum analysis of the model B at PGA equal to 30% is performed at the start to find out the distribution of stresses. The distribution of shear stresses along the x-direction in the minaret is presented in the following figure. The figure shows that the value of the shear stresses also decreases as we move from bottom to top of the minaret. The maximum shear stresses occur just above the cross-section of junction.

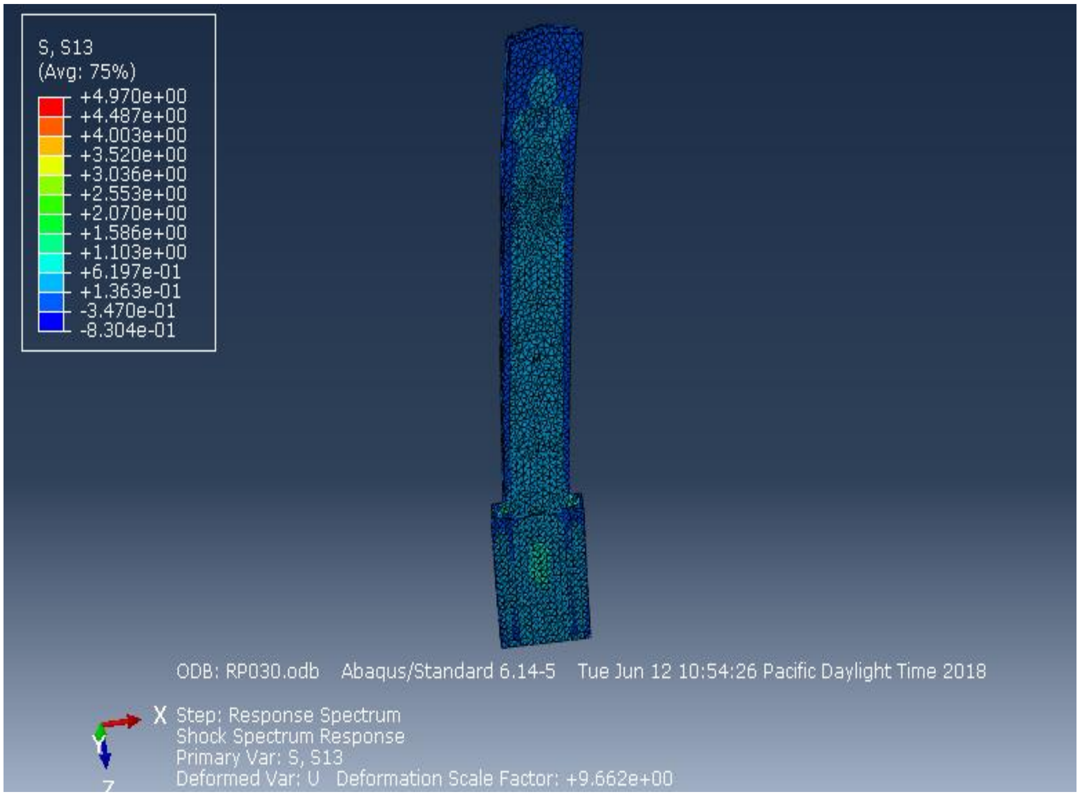


Figure 4.13. Shear Distribution under Response Spectrum

The distribution of shear stresses across the cross-section having critical condition is;

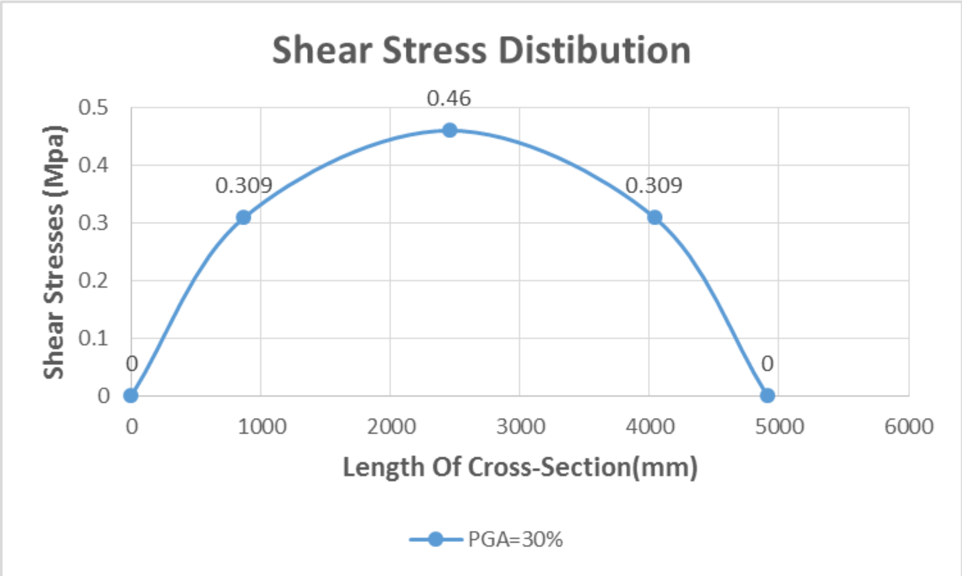


Figure 4.14. Shear Distribution under Response Spectrum and Gravity

The maximum value of the stress is 0.46 Mpa. In order to get the limiting state, the concern is to find the value of PGA at which this maximum value falls below the maximum shear capacity of the material i.e. 0.028 Mpa. For this purpose, the value of the PGA is gradually reduced and its effect is studies;

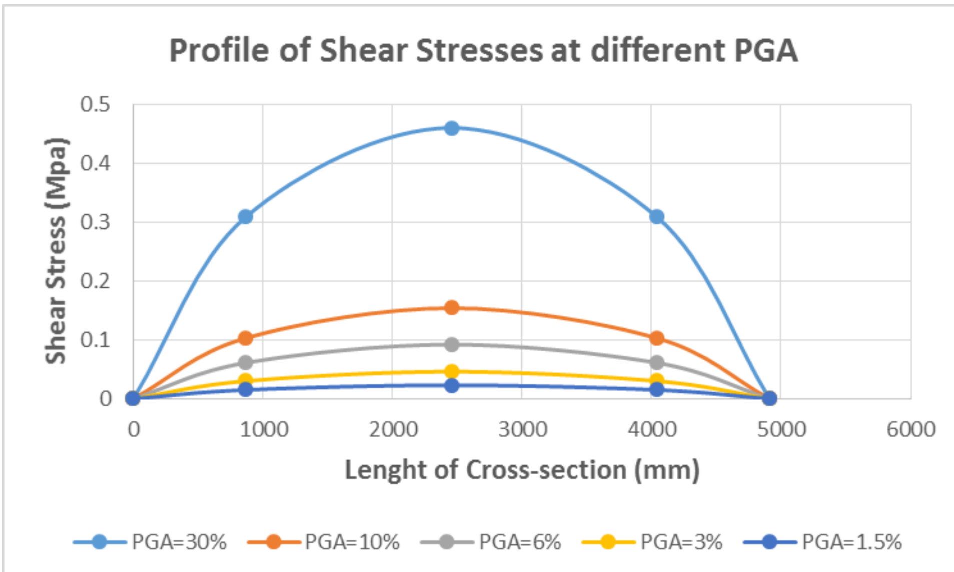


Figure 4.15. Profile of shear stresses at different values of PGA

It can be seen from the graph, that the value of PGA which produces maximum shear stresses lower than 0.028 Mpa is 1.5%. The limiting value of PGA under shear is much lower as compared to the limiting value of PGA for bending which is 6.3%.

4.4.2. Limiting State of Model A

The profile of shear stresses across the cross-section at value of the PGA equal to 30% is shown in the subsequent profile;

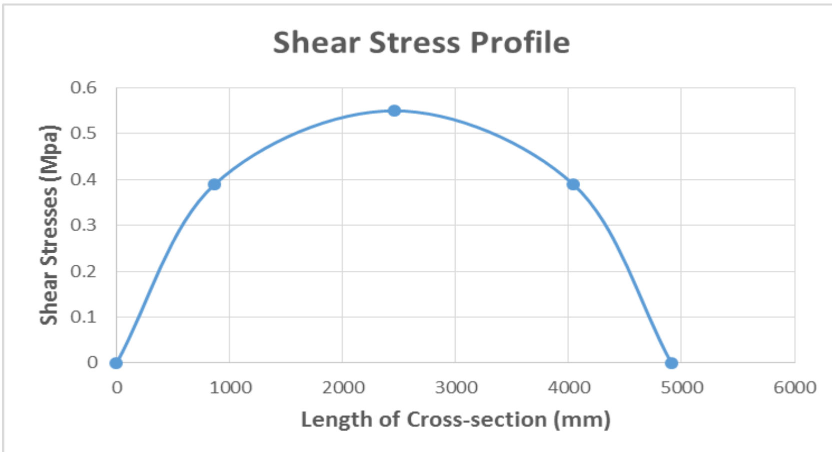


Figure 4.16. Shear Stress Profile under Response Spectrum and Gravity

The maximum value of the shear stress increased from 0.46 Mpa to 0.55 Mpa due to the presence of stairs and central pillar. Now, in order to get the limiting state again, the analysis is performed at decreasing values of PGA to get shear stresses lesser than 0.028 Mpa. The consequences of this is displayed in the resulting diagram;

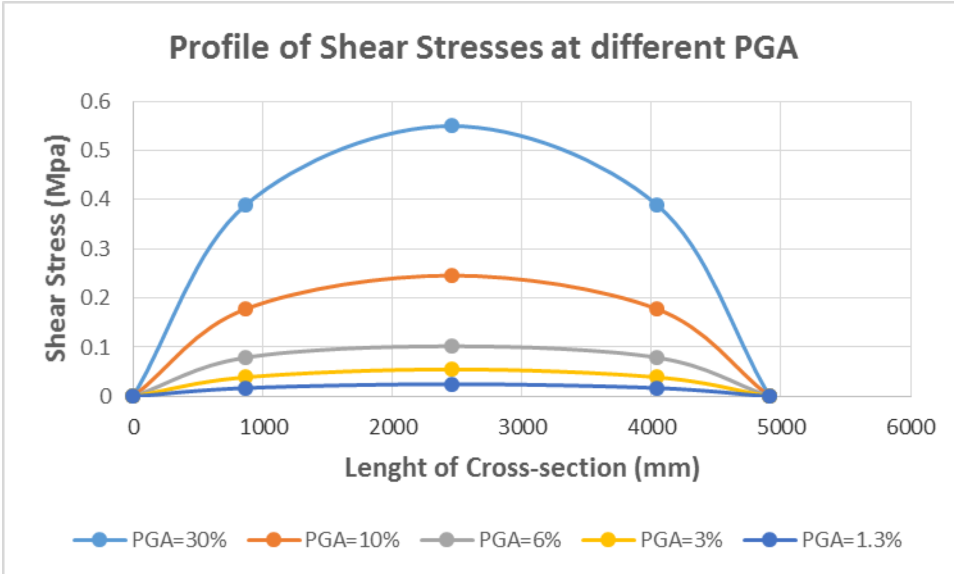


Figure 4.17. Profile of Shear Stresses at different values of PGA

Hence in this case, the limiting value of the PGA is achieved at 1.3% which is very close the value achieved for the model B.

5. Strengthening Interventions for the Minaret

In this chapter, the interventions for the strengthening of the minaret are explained. The purpose to provide strengthening interventions is to increase the seismic resistance of the minaret and decrease its vulnerability to earthquake excitations. The increase in seismic resistance is studied on the basis of value of peak ground acceleration (PGA). The main concern is to increase the value of PGA which is obtained for bending and shear cases for the minaret in the above section. Moreover, there are many different kind of strengthening interventions that can be proposed, however, in this project following interventions are proposed;

1) Increasing Mechanical properties of the masonry material
2) Providing Vertical Reinforcement
3) Providing Shear Reinforcement

All these strengthening interventions have been studied in detail in the following sections.

5.1. Increasing Mechanical Properties of the Material

The strengthening intervention covers in this section has a lot of constraints. The major problem in increasing the mechanical property or changing the material type to get higher strength is that the authorities want reconstruction of the minaret to be done with the same material type and using the damage material as much as possible to ensure originality of the minaret. The purpose is to conserve the history, culture and heritage of the site. However, there are many ways to increase the mechanical property of the material by using different techniques. Some of them are injections grouting, increasing strength of mortar placed between masonry etc.

To study the effect of increase of mechanical property on the value of PGA, the response spectrum analysis along with is performed under different values of elastic modulus of the material (E) at limiting value of PGA i.e. 10% for base cross-section and 6.3% for cross-section at junction. The elastic modulus values are increased gradually from 900 Mpa until 3200 Mpa. Following results are obtained after carrying out analysis at different values of the elastic modulus;

- **For Cross-section at base:**

The distribution of stresses across the cross-section at different value of the modulus of elasticity is shown in the following table and chart;

Length (mm)	E=900 Mpa	E=1000 Mpa	E=1100 Mpa	E=1260 Mpa	E=1600 Mpa	E=2400 Mpa	E=3200 Mpa
0	0.00099	0.01099	0.02872	0.09444	0.16523	0.33853	0.48778
491	-0.22101	-0.21301	-0.198826	-0.142468	-0.089618	0.049022	0.168422
982	-0.44301	-0.43701	-0.426372	-0.379376	-0.344466	-0.240486	-0.150936
1473	-0.66501	-0.66101	-0.653918	-0.616284	-0.599314	-0.529994	-0.470294
1964	-0.88701	-0.88501	-0.881464	-0.853192	-0.854162	-0.819502	-0.789652
2455	-1.10901	-1.10901	-1.10901	-1.0901	-1.10901	-1.10901	-1.10901
2946	-1.33101	-1.33301	-1.336556	-1.327008	-1.363858	-1.398518	-1.428368
3437	-1.55301	-1.55701	-1.564102	-1.563916	-1.618706	-1.688026	-1.747726
3928	-1.77501	-1.78101	-1.791648	-1.800824	-1.873554	-1.977534	-2.067084
4419	-1.99701	-2.00501	-2.019194	-2.037732	-2.128402	-2.267042	-2.386442
4910	-2.21901	-2.22901	-2.24674	-2.27464	-2.38325	-2.55655	-2.7058

Table 4.3. Bending Stresses values at different values of Elastic Modulus

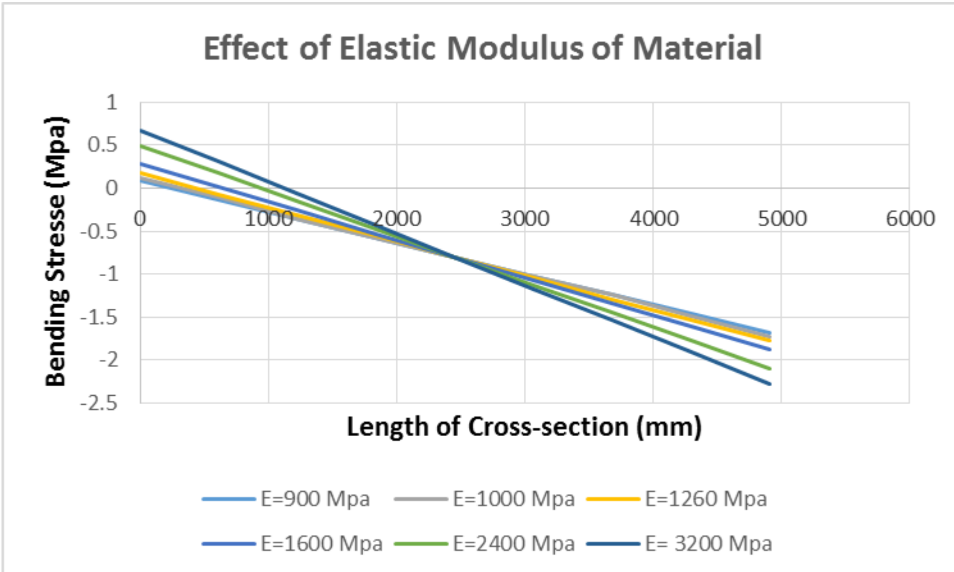


Figure 4.18. Bending Stresses Distribution at different values of Elastic Modulus

• For Cross-section at Junction:

The distribution of stresses across the cross-section at different value of the modulus of elasticity is shown in the following table and chart;

Length (mm)	E=900 Mpa	E=1000 Mpa	E=1100 Mpa	E=1260 Mpa	E=1600 Mpa	E=2400 Mpa	E=3200 Mpa
0	0.02	0.082663	0.11949	0.175173	0.2819	0.49502	0.67582
613.75	-0.185	-0.1380028	-0.1103825	-0.06862025	0.011425	0.171265	0.306865
1227.5	-0.39	-0.3586685	-0.3586685	-0.3124135	-0.25905	-0.15249	-0.06209
1841.25	-0.595	-0.5793343	-0.5793343	-0.55620675	-0.529525	-0.476245	-0.431045
2455	-0.8	-0.8	-0.8	-0.8	-0.8	-0.8	-0.8
3068.75	-1.005	-1.0206658	-1.0206658	-1.04379325	-1.070475	-1.123755	-1.168955
3682.5	-1.21	-1.2413315	-1.2413315	-1.2875865	-1.34095	-1.44751	-1.53791
4296.25	-1.415	-1.4619973	-1.4896175	-1.53137975	-1.611425	-1.771265	-1.906865
4910	-1.62	-1.682663	-1.71949	-1.775173	-1.8819	-2.09502	-2.27582

Table 4.4. Bending Stresses values at different values of Elastic Modulus

And the graphical representation is;

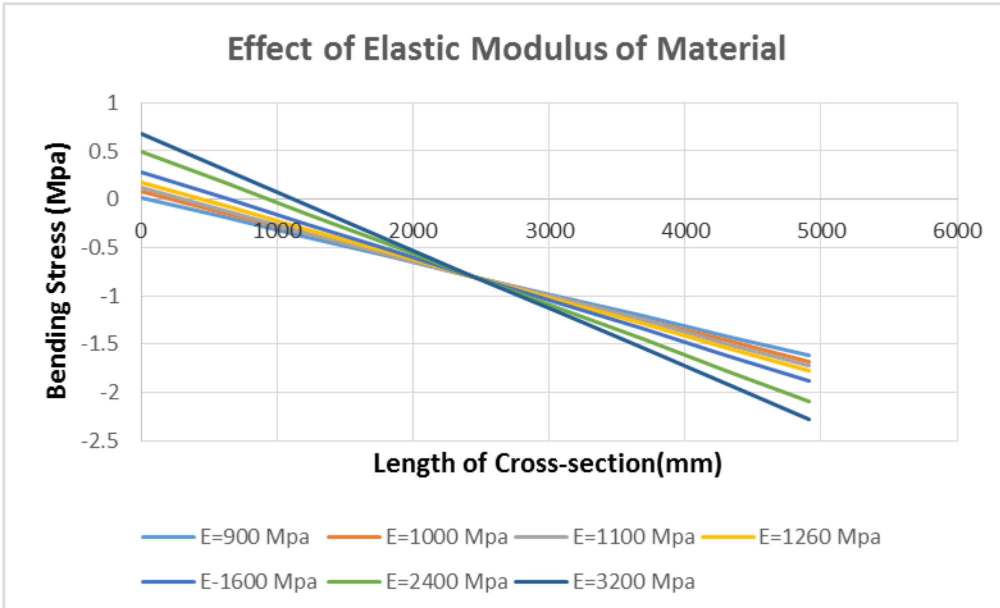


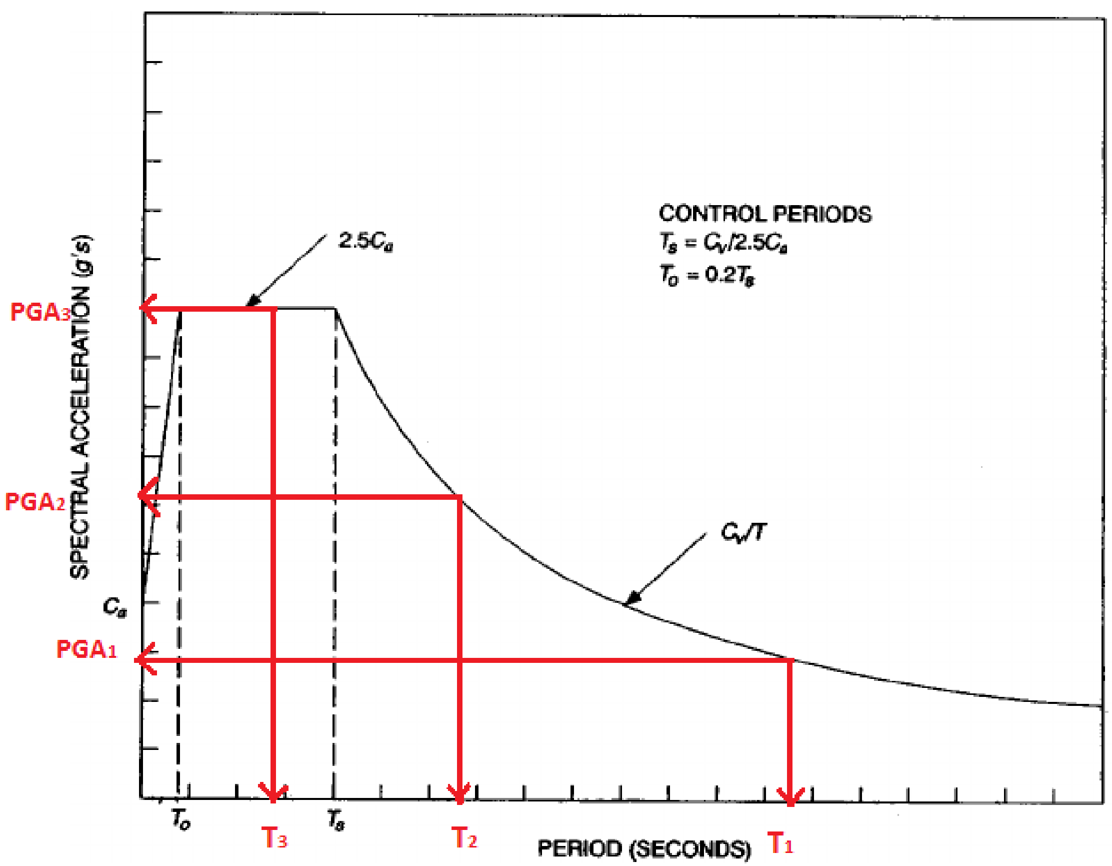
Figure 4.19. Bending Stresses Distribution at different values of Elastic Modulus

The graphical representation shows that the increase of elastic modulus increases the stresses in the minaret but it does not provide any benefit in terms of PGA. This is because of the reason that if we want to achieve the limiting condition again, then we have to decrease the value of PGA which is opposite to scope of our target. So it is not recommended to increase the value of the elastic modulus.

Another reason is that when elastic modulus of the material increases, it changes the modal response of the minaret. Due to relation between stiffness and time, when elastic modulus is increased the stiffness of the minaret increase which in turn decreases the time period of the minaret. It is explained in the following relation;

$$f_n = \frac{1}{2\pi}\sqrt{\frac{k}{m}}$$

The reduction of the time period with increasing values of elastic modulus, moves the minaret in to the region of higher PGA. It can be explained with the help of subsequent figure;



It can be clearly evident from the figure, if the stiffness of the model is increased by increasing elastic modulus of the material, time period decreases due to which model moves to state which have higher value of the PGA in the design response spectrum which makes minaret more vulnerable to earthquake excitations.

5.2. Providing Vertical Reinforcement

The vertical reinforcement is provided in the minaret to increase the seismic resistance in flexure. The main objective of providing flexural reinforcement is also to increase the value of the limiting value of PGA. The flexural reinforcement is provided at different values of steel ratios and the effect of increasing values of steel ratio is studied on the value of the peak ground acceleration. The properties of the material of the steel used for the vertical reinforcement is shown in the table;

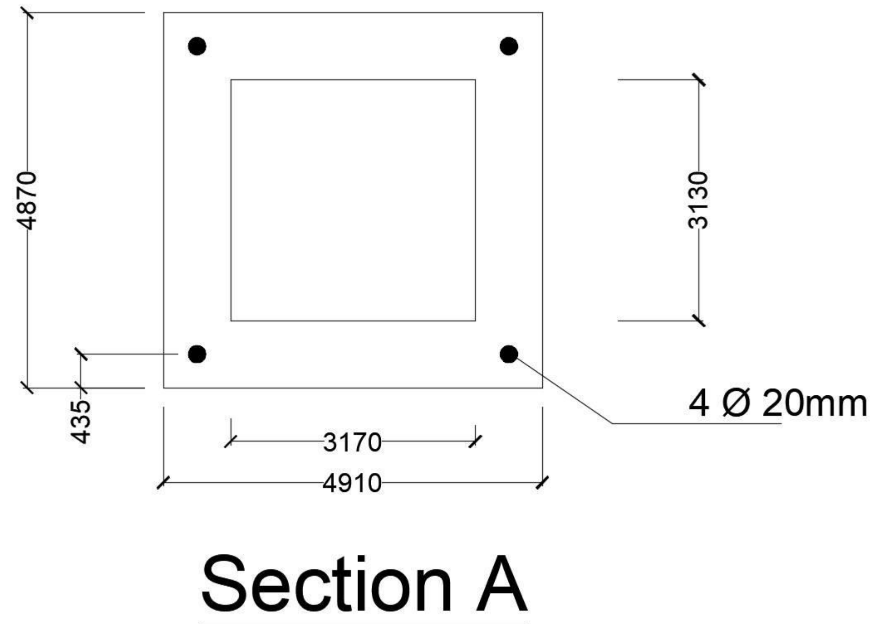
Properties of Steel	
Elastic Modulus	**210000 Mpa**
Density	**0.00000785 Mpa**
Poisson Ratio	**0.3**

The response spectrum analysis along with gravity analysis is performed at different values of steel ratio and results are reported and discussed for both types of cross-section that we have considered. As it be seen from the above table, the number of bars in the cross-section of the minaret are twelve (12) but the steel ratio is increases by increasing the diameter of the bars.

- **Cross-Section At Base:**

The result of the response spectrum analysis along with gravity analysis at different value of steel ratios has subsequent graphical representation;

➢ At Steel Ration ($\rho_{=}$ 0.0001%):



At this value of steel ratio, the following result is obtained for values of PGA equal to limiting value which is 10%;

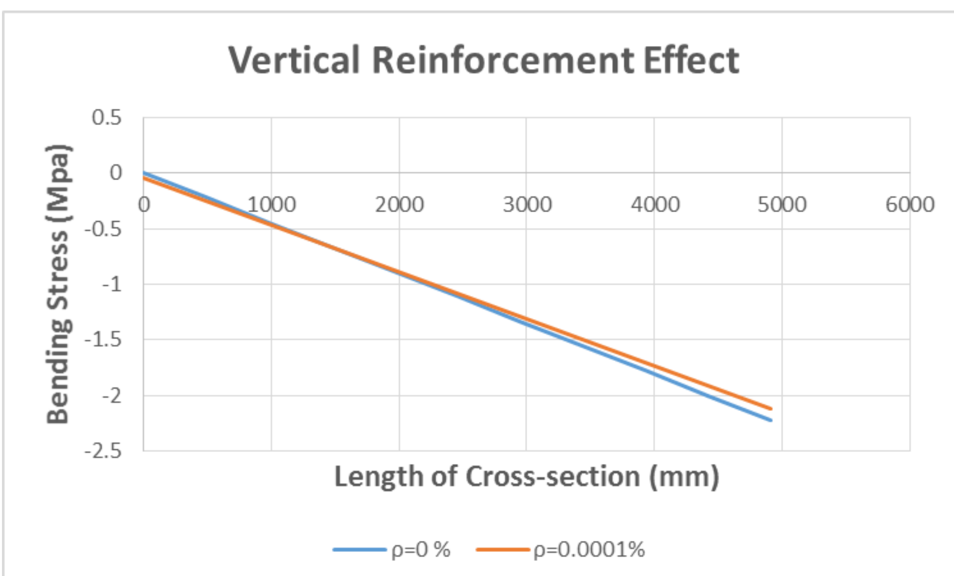
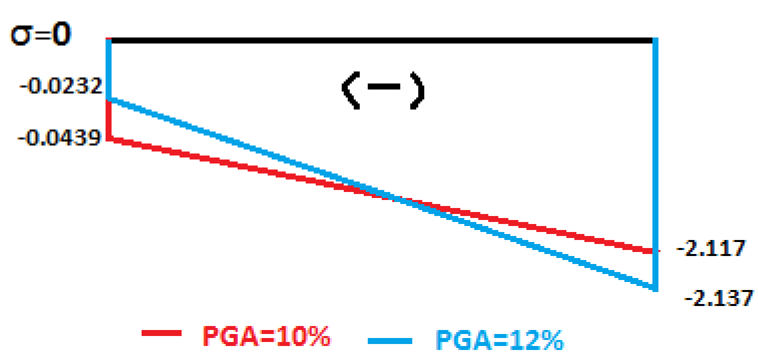


Figure 4.20. Comparison of Bending Stresses at ρ=0% & ρ=0.0001%

The graph shows that the values of stresses in the cross-section reduces when vertical reinforcement is introduced into the minaret. The effect of this reduction is that there is distribution of compressive bending stresses all over the cross-section which means that if the cross-section is to attain the limiting state again, then we have to increase the value of PGA so that stresses in the cross-section increases and the limiting state is achieved. In the graph, the blue profile corresponds to the value of stresses at 0% of steel ratio while the orange one shows the distribution of stresses due to 0.0001 % of steel. It can be clearly seen that the stresses have decreased in the cross-section and there is compression state all over the cross-section.

In order to study the increase in value of PGA, the analysis is again performed at an increase value of PGA, more than the limiting value i.e. 12%. Due to the problem of showing difference between the results in graphical representation, the general comparison of results is shown;

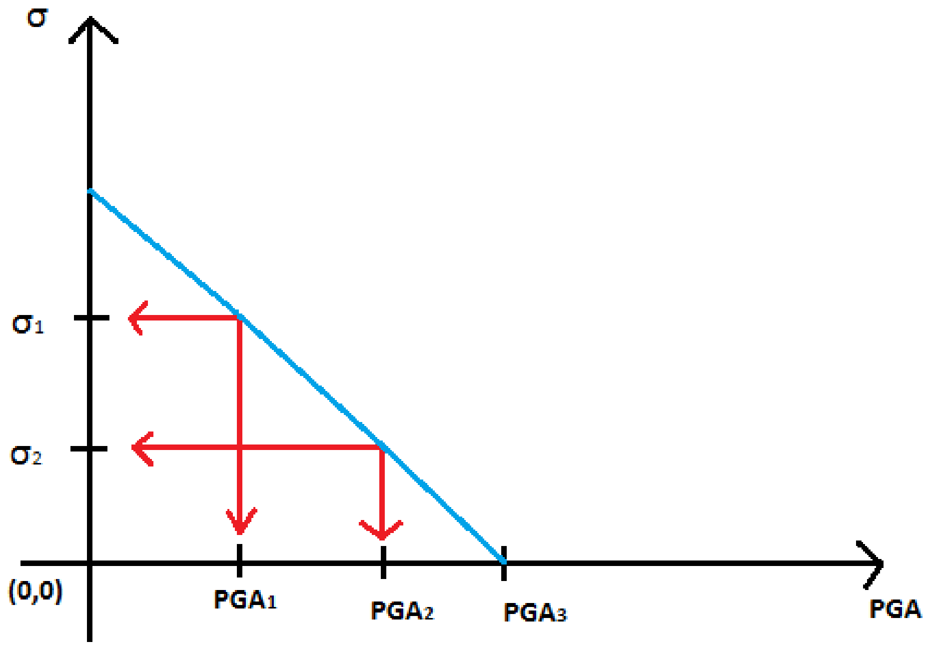


The distribution of stresses is shown in the table for all the cases;

Length(mm)	Bending Stresses (Mpa) at PGA=10%	Bending Stresses (Mpa) at PGA=12%
0	-0.04391	-0.02318
491	-0.25122	-0.234636
982	-0.45853	-0.446092
1473	-0.66584	-0.657548
1964	-0.87315	-0.869004
2455	-1.08046	-1.08046
2946	-1.28777	-1.291916
3437	-1.49508	-1.503372
3928	-1.70239	-1.714828
4419	-1.9097	-1.926284
4910	-2.11701	-2.13774

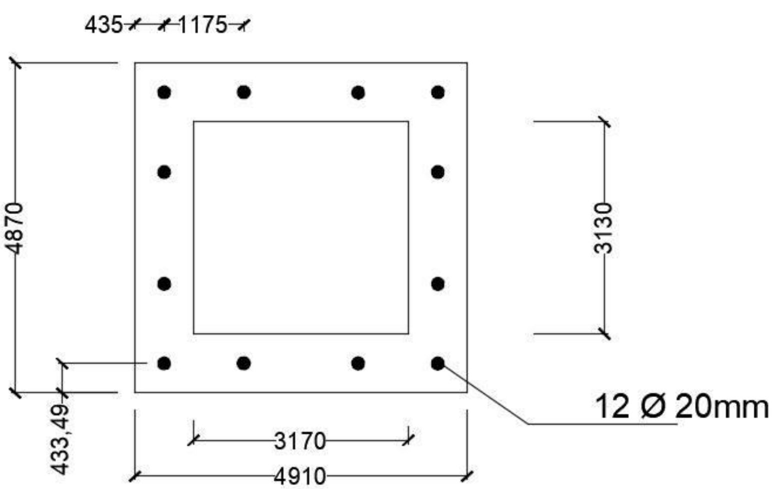
Table 4.4. Bending Stresses at different at values of PGA=10% & 12%

It is clear from the above results that if the value of the PGA is increased the cross-section is moving towards the limiting condition. For this purpose, the linear interpolation technique is followed. As we are working in linear elastic regime, linear interpolation can be applied to study the response.



In the end by following above mentioned method, the new limiting value of PGA after putting 0.0001% of steel ratio is 14.23%.

- **At Steel Ratio (ρ= 0.000269%):**



Section B

At this input, the following results are acquired;

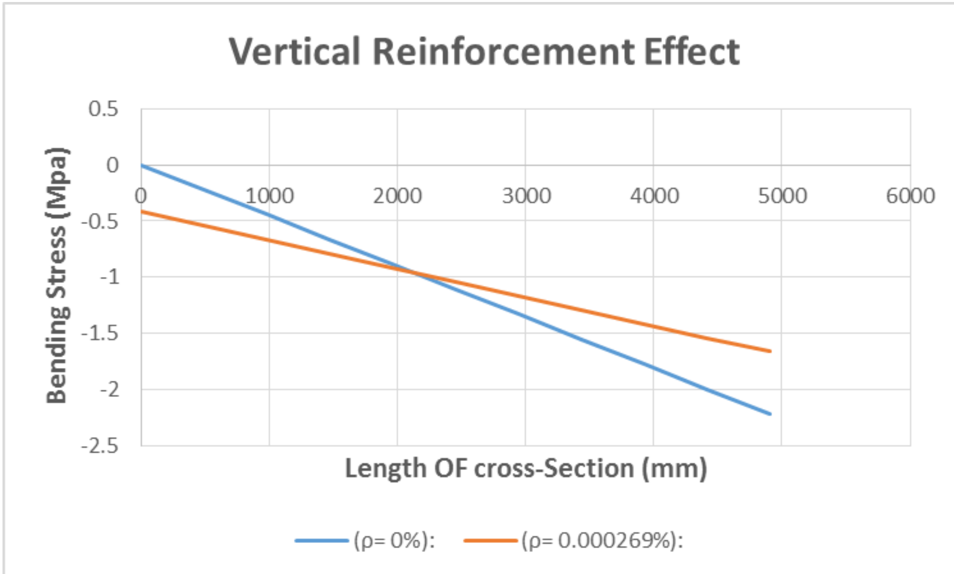


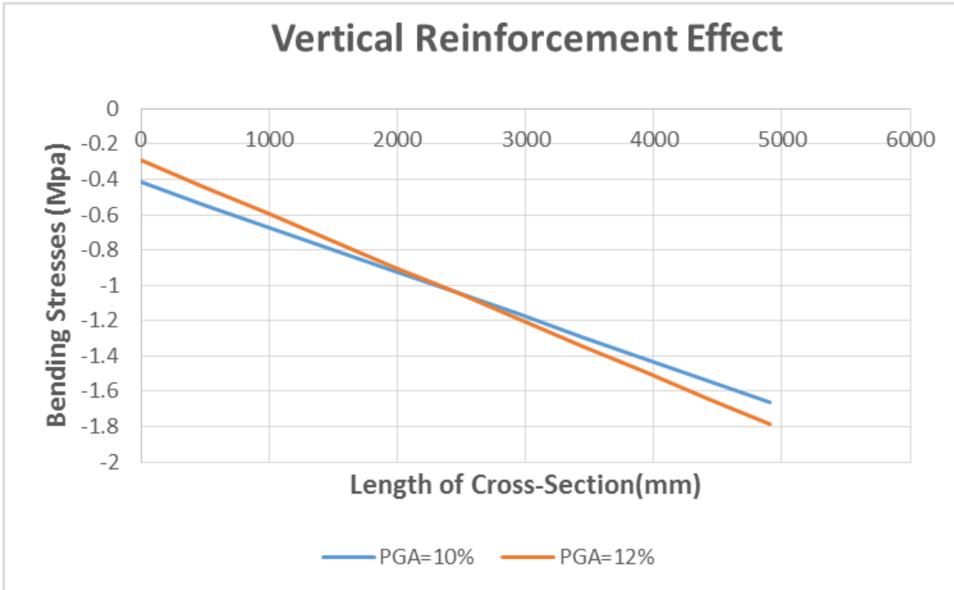
Figure 4.21. Comparison of Bending Stresses at ρ=0% & ρ=0.000269%

It can be seen that the stresses reduces again and hence the value of the PGA will increase. If the analysis is performed at an increased value, the stresses profile in the cross-section will be such that it will try to move towards the limiting sate i.e. towards zero value of stress on one end of the cross-section. The table shows the comparison of stresses at different values of PGA for this steel ratio;

Length(mm)	Bending Stresses at PGA=10%	Bending Stresses at PGA=12%
0	-0.416967	-0.292273
491	-0.5416596	-0.4419044
982	-0.6663522	-0.5915358
1473	-0.7910448	-0.7411672
1964	-0.9157374	-0.8907986
2455	-1.04043	-1.04043
2946	-1.1651226	-1.1900614
3437	-1.2898152	-1.3396928
3928	-1.4145078	-1.4893242
4419	-1.5392004	-1.6389556
4910	-1.663893	-1.788587

Table 4.5. Bending Stresses at different at values of PGA=10% & 12%

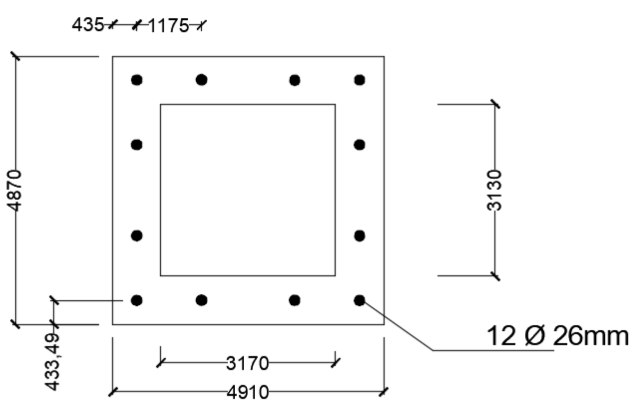
The subsequent graphical representation shows this aspect;



Hence following the same procedure as described above, the value of the PGA in this case increases to 16.68%.

- **At Steel Ratio ($\rho_=$ 0.000454%):**

The value of steel ratio is further increased in this section to study its effect on the value of PGA.



Section B-1

The graphical representation is shown in comparison to results at steel ratio of 0%, which shows the reduction of stresses;

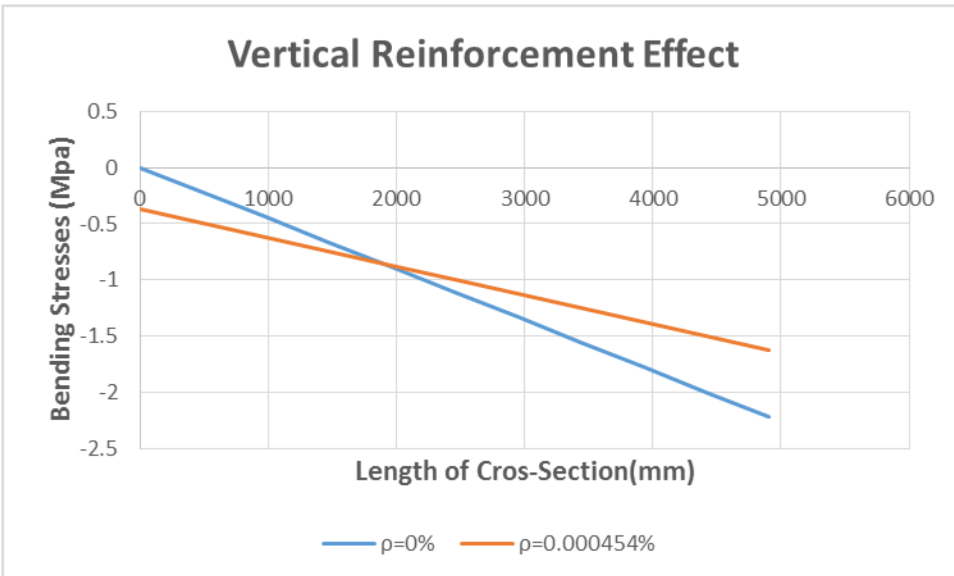


Figure 4.22. Comparison of Bending Stresses at ρ=0% & ρ=0.000454%

After running the analysis at increased value of PGA=12% and following the interpolation technique as mentioned above, the value of PGA in this case achieved is 15.99%. This value of PGA is lesser as compared to the previous case. In the previous case, at value of steel ratio 0.000269% the value of PGA was 16.68%. This is because of the reason that at value of steel ratio equal to 0.000454% the reduction of stresses in the cross-section is lesser as compared to previous case of steel ratio equal to 0.000269% when the analysis is performed at PGA equal to 10%. The table shows the comparison of stresses at different values of steel ratios;

Length(mm)	Bending Stresses at (ρ=0.000269%)	Bending Stresses at (ρ=0.000454%)
0	-0.416967	-0.374193
491	-0.5416596	-0.4990084
982	-0.6663522	-0.6238238
1473	-0.7910448	-0.7486392
1964	-0.9157374	-0.8734546
2455	-1.04043	-0.99827
2946	-1.1651226	-1.1230854
3437	-1.2898152	-1.2479008
3928	-1.4145078	-1.3727162
4419	-1.5392004	-1.4975316
4910	-1.663893	-1.622347

The graphical representation of this result is shown in the following chart;

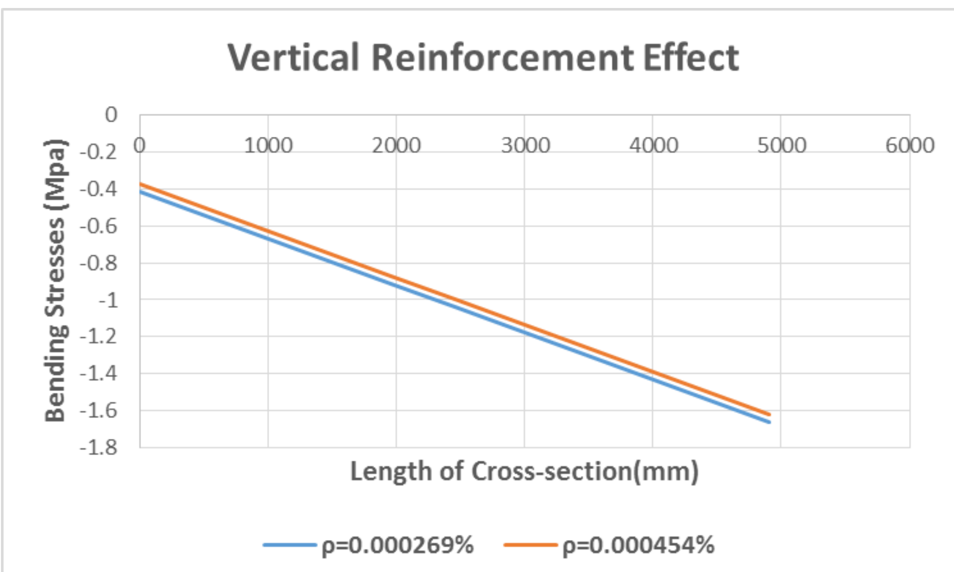
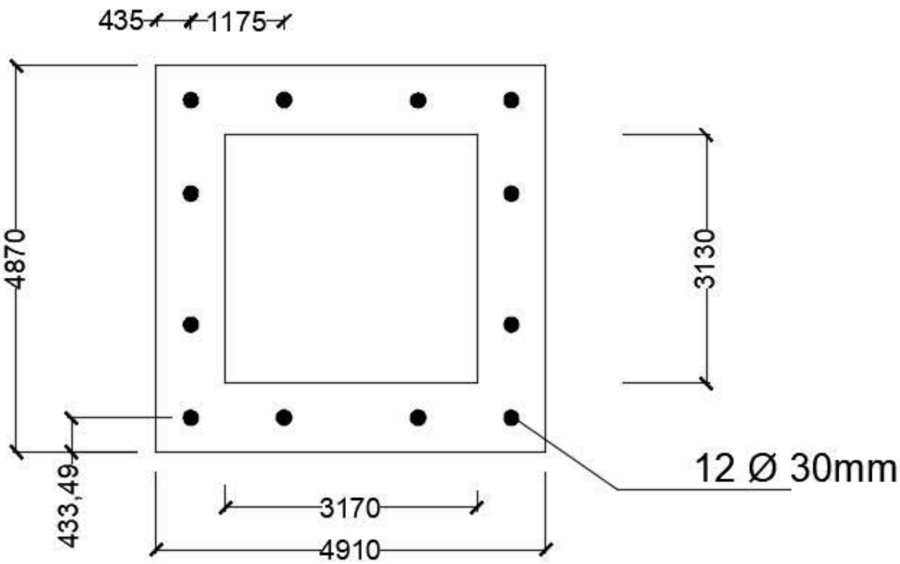


Figure 4.23. Comparison of Bending Stresses at ρ=0.000269% & ρ=0.000454%

➢ **At Steel Ratio (ρ= 0.0006%):**



Section B-2

The following results are obtained at this input of vertical reinforcement. The subsequent graph shows the comparison of stress distribution between value of steel ratios 0% and 0.0006% at PGA equal to 10%.

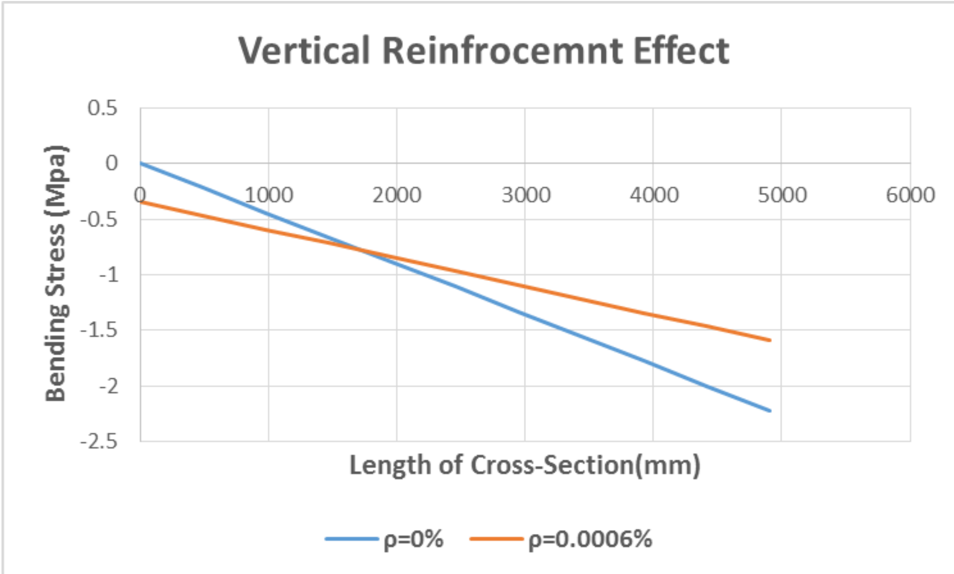


Figure 4.24. Comparison of Bending Stresses at ρ=0% & ρ=0.0006%

The value of PGA in this case further reduces to 15.47%. The reason for this reduction is same as for the previous case i.e. the reduction of stresses at this steel ratio is lesser. The amount of stresses produced in this case is lower as compared to previous cases which can be seen in the table

Length(mm)	Bending Stresses at (ρ=0.000454%)	Bending Stresses at (ρ=0.0006%)
0	-0.374193	-0.341956
491	-0.4990084	-0.4668884
982	-0.6238238	-0.5918208
1473	-0.7486392	-0.7167532
1964	-0.8734546	-0.8416856
2455	-0.99827	-0.966618
2946	-1.1230854	-1.0915504
3437	-1.2479008	-1.2164828
3928	-1.3727162	-1.3414152
4419	-1.4975316	-1.4663476
4910	-1.622347	-1.59128

It is shown in the chart as follows;

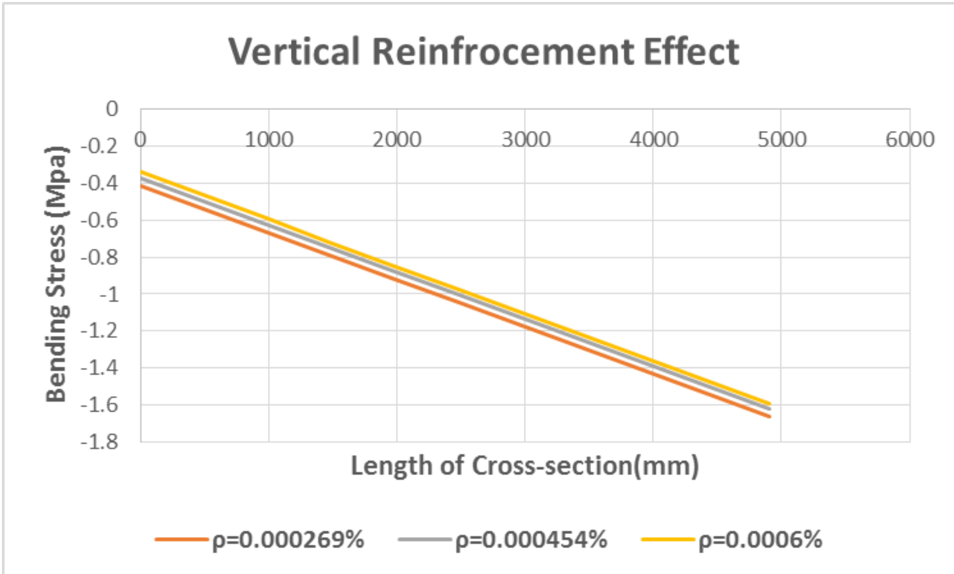
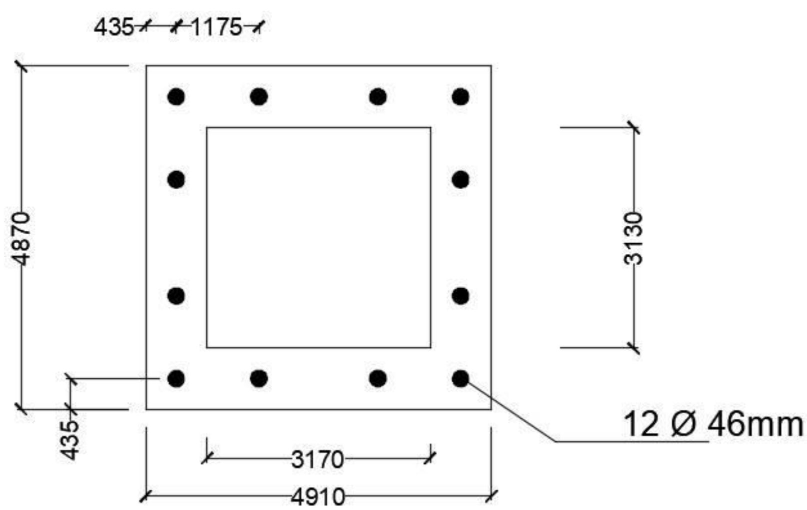
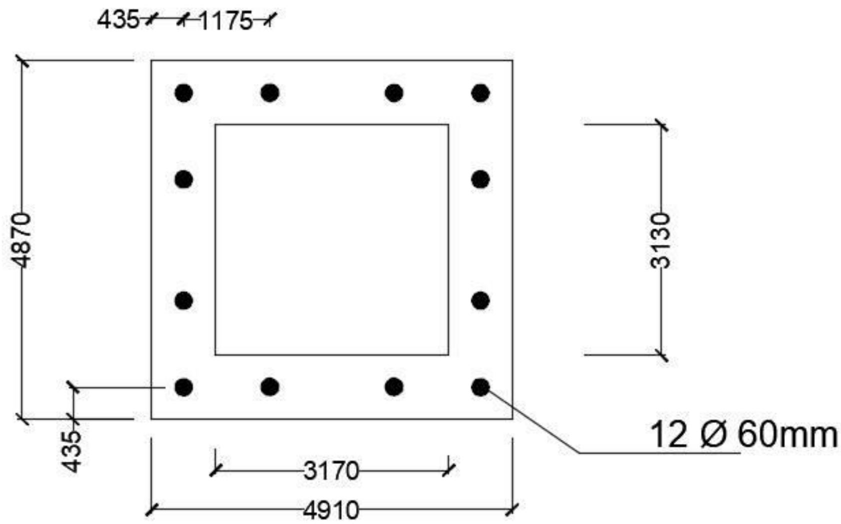


Figure 4.25. Comparison of Bending Stresses at ρ=0.000269%, ρ=0.000454% & ρ=0.0006%

➢ At Steel Ratio ($\rho_=$ 0.00142% & $\rho_=$ 0.00242%):



Section B-3



Section B-5

The same nature of results are obtained at these values of steel ratios. As expected, the value of the PGA is further reduced in both of these cases because of the same reason mentioned in the above discussion. The limiting values of PGA obtained at steel ratio of 0.00142% and 0.00242% are 13.13% and 11.35% respectively.

Finally on the basis of the above results, the relation between value of the steel ratio and peak ground acceleration (PGA) can be achieved.

Steel Ratio (%)	PGA (%)
0	10
0.0001	14.23637
0.000269	16.68
0.000454	15.99596
0.0006	15.47427
0.0009	14.5
0.00142	13.13586
0.00242	11.35

Table 4.6. Values of PGA (%) at different values of Steel ratios

And the graphical representation is;

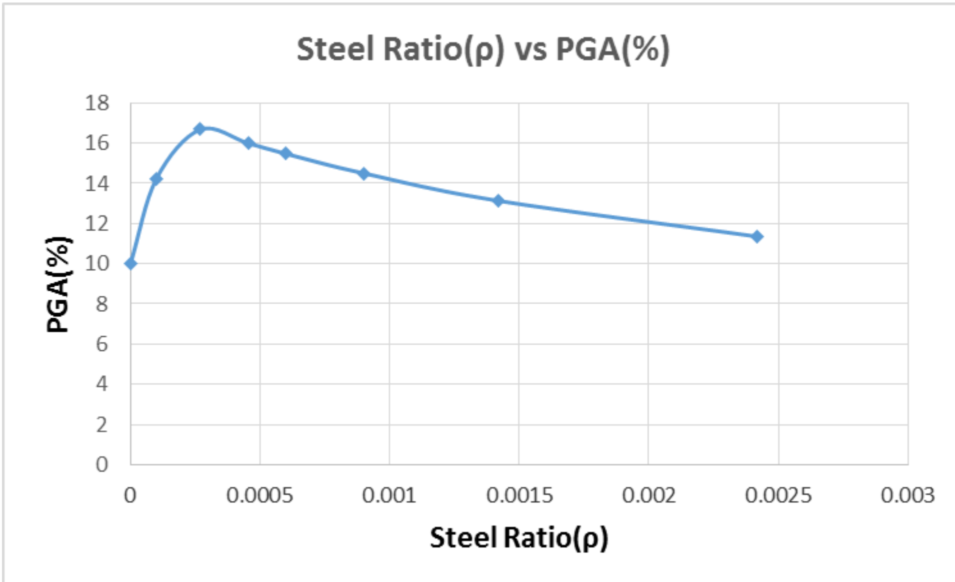


Figure 4.26. PGA (%) VS Steel Ratio (ρ) For Base Cross-section

Hence, it can be said that the maximum value of the steel ratio which provides benefit in terms of achieving maximum value of the steel ratio is 0.000269%. However, the steel ratio more than this input also provides benefit in terms of achieving PGA higher than 10%.

- **Cross-section At Junction:**

The same procedure is followed for this cross-section. The limiting value produced for this cross-section is 6.3% at value of steel ratio equal to 0%. Therefore, putting vertical reinforcement at different dosages and its effect on the value of the PGA is studied. The same nature of results are obtained for this cross-section as depicted in the following table and figure;

Steel Ratio(ρ)	PGA (%)
0	6.3
0.0001	10.4545
0.000269	16.6697
0.000454	15.979
0.0006	15.3684

And the graphical representation is;

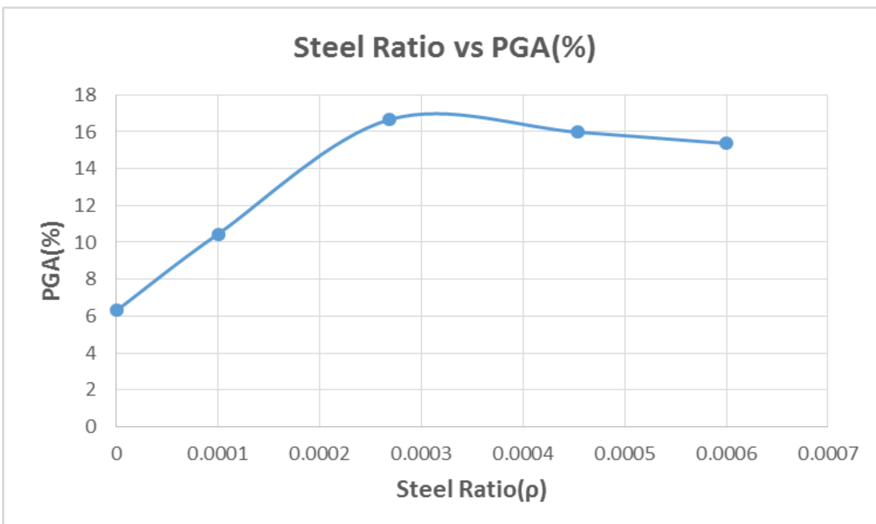


Figure 4.27. PGA (%) VS Steel Ratio (ρ) For Cross-section at Junction

It can be seen from the above results, the maximum values of the PGA is achieved for both cross-sections at same value of the steel ratio. The difference of the maximum value of PGA for two cross-sections is very negligible i.e. (0.02%).

5.3. Providing Hoop Reinforcement

In this section, the results and discussions based on the strengthening intervention against shear behavior of minaret are explained in detail. In this project, the strengthening intervention used to increase the behavior of the minaret in shear against seismic excitation is the Hoop reinforcement. The hoop reinforcement increases the shear strength of the minaret by the hooping action. It is known that flexural reinforcement affects the shear behavior of the structure in shear but the hoop or shear reinforcement does not affect the bending stresses in the structure. Therefore, to study the effect of the shear reinforcement on the minaret, the flexural reinforcement also has to take into account. As the effect of shear reinforcement on seismic response of the minaret is also studied on the basis of the value of peak ground acceleration. The optimum amount of the flexural reinforcement which produces maximum value of the PGA is retained for this purpose i.e. ($\rho_{=}$ 0.000269%). The effect of shear reinforcement is studied effect of spacing ‘s’ of shear reinforcement; which is discussed as follows;

- **Effect of Spacing (S):**

The Response spectrum analysis is performed on the minaret at increasing values of spacing of shear reinforcement. The value of the diameter for these analysis is same which is kept at 20 mm. The values of spacing used and corresponding number of shear bars are shown in the following table;

Spacing (mm)	Number of Bars
1000	39
2000	19
2500	16
3000	13
3500	11
4500	9

The amount of the shear reinforcement increases as the value of the spacing decreases. Following is the response of the minaret in shear at different values of the shear reinforcement spacing;

- **At S=4500 mm:**

The analysis starts from the highest value of the spacing which constitutes nine (9) bars and have minimum reinforcement. The analysis is first performed at limiting value of the PGA which is 1.5% as obtained in the section 4.5., which shows that the values of the stresses in the cross-section reduces due to the presence of hoop reinforcement. The comparison between the shear with and without the presence of hoop reinforcement is shown in the following table and chart;

	Without Reinforcement	**With Reinforcement**
Length(mm)	**Shear Stress (Mpa)**	**Shear Stress (Mpa)**
0	0	0
870	0.015	0.00875855
2455	0.023	0.0121263
4040	0.015	0.00875855
4910	0	0

Table 4.7. Comparison of shear stresses with and without reinforcement

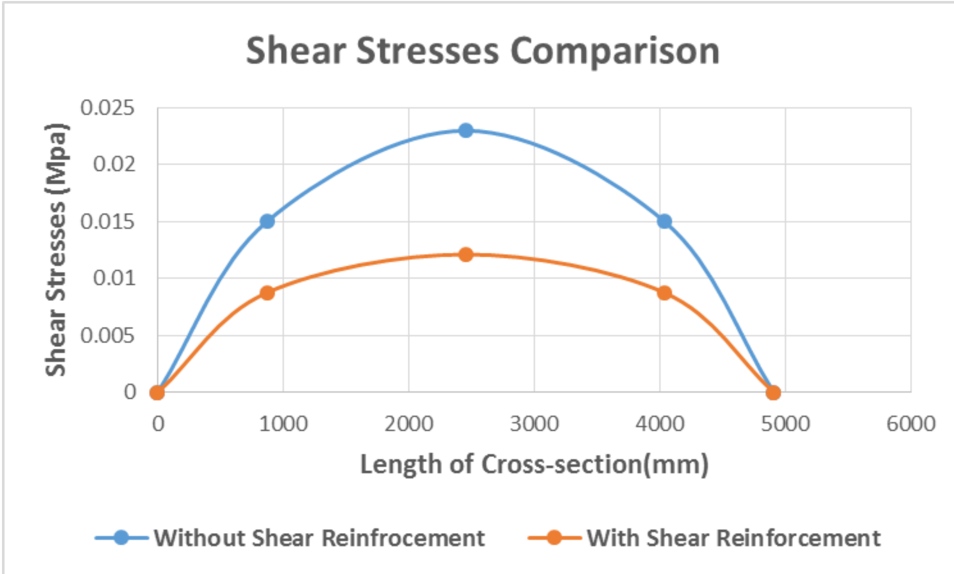
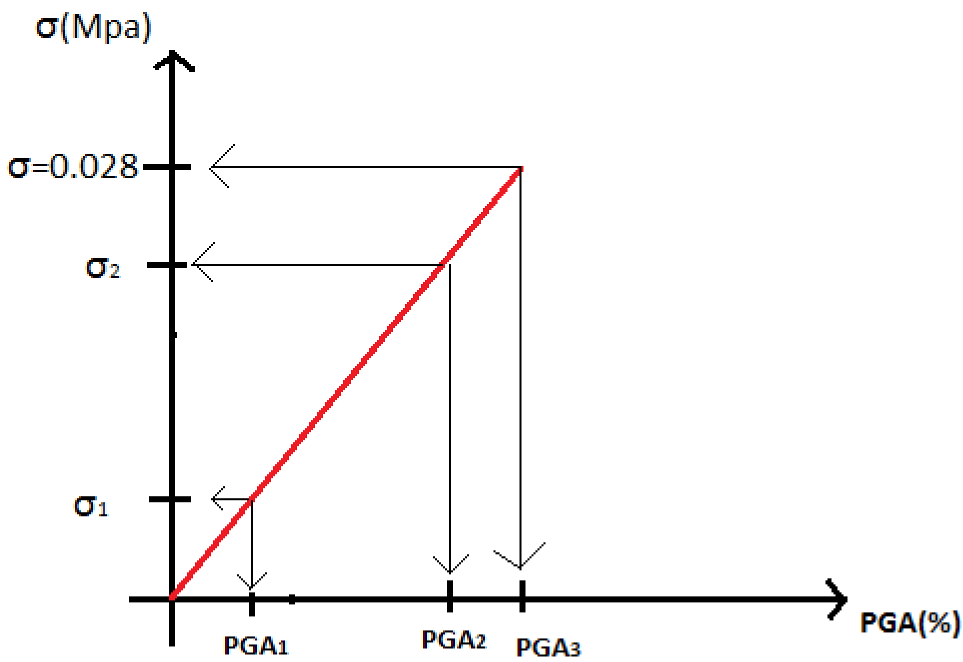


Figure 4.27. Shear Stresses Comparison

Considering the thing that shear stresses in the cross-section reduces, which means that the shear strength of minaret increases after providing shear reinforcement. Therefore, it means that we may have to increase the value of the PGA to get to limit situation i.e. the maximum shear stresses in the cross-section reaches to 0.028 Mpa. As we are working in linear elastic regime, the linear interpolation technique can be followed and the increase in the value of the PGA can be studies because of shear reinforcement. In this case the following linear interpolation is tracked;



Hence the analysis is performed at increased value of PGA=1.8%, and by following the above mentioned technique, the new limiting value of PA achieved is 7.95%.

- **At S=3500 mm:**

The number of bars used for this spacing are 11 bars of diameter 20 mm. The same nature of behavior is obtained with this configuration. The distribution and reduction of shear stresses in the cross-section due to the insertion of shear reinforcement is shown the following table;

	Without Reinforcement	With Reinforcement
Length(mm)	Shear Stress (Mpa)	Shear Stress (Mpa)
0	0	0
870	0.015	0.00878307
2455	0.023	0.0121263
4040	0.015	0.00878307
4910	0	0

The same method is followed as before and the value of PGA obtained in this case is 7.39%.

- **At S= (3000, 2500, 2000) mm:**

The same procedure as followed before is used and the corresponding value of the PGA is determined. The values of the PGA found for 3000 mm, 2500 mm and 2000 mm are 6.8%, 6.35% and 5.95% respectively.

The graphical relation between value of PGA (%) and shear reinforcement spacing (s) can be presented and is shown below;

Spacing (mm)	PGA (%)
1000	8.10
2000	7.95
2500	7.39
3000	6.8
3500	6.35
4500	5.95

Table 4.8. Comparison between Spacing (s) and PGA (%) for shear

And the chart is;

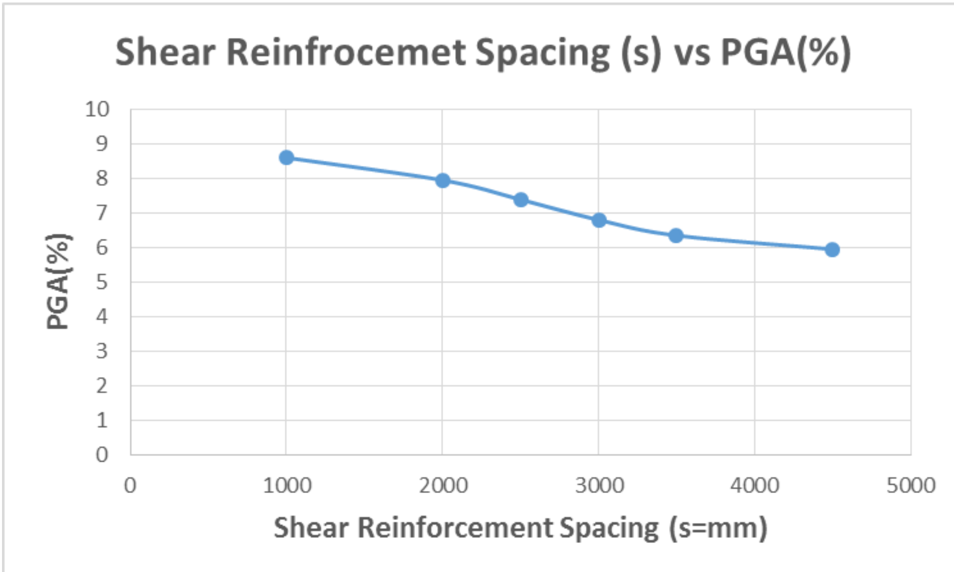
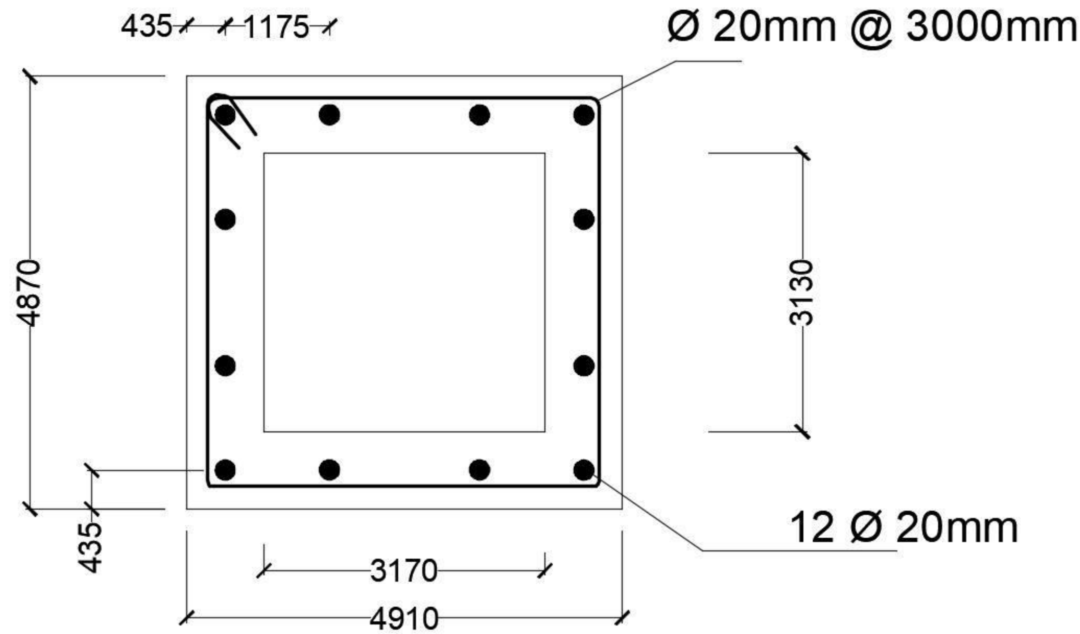


Figure 4.28. Graphical Representation of comparison between Spacing and PGA

It can be commented that the value of the PGA decreases as the shear reinforcement spacing increases (quantity of reinforcement decreases). But the there is no significant difference of results at different values of shear reinforcement spacing (s). Therefore, the value of the spacing s=3000 mm which corresponds to 13 bars of diameter 20 mm can be provided to ensure significant performance of the minaret in shear.

In the end, we have following configuration of vertical reinforcement and shear reinforcement which provides benefit in terms of seismic performance of the minaret.

The following section C shows the cross-section which gives optimum value of the PGA;



Section C

The cross-section shows the amount of the vertical and hoop reinforcement which should be used in the minaret to ensure increase of the seismic response of the minaret.

6. Conclusions

6.1. Summary

Minarets are one of the prominent form of masonry structures. They are present in abundant around the world. The minaret of ‘Al-Umayyad Mosque’ in the city of Aleppo, Syria have huge cultural and heritage importance. The minarets are vulnerable to seismic excitations and they are prone to failure. Therefore, these structures should be given utmost importance and attention, and provisions must be made to ensure safety of such structures against seismic excitations. Therefore, in this thesis the seismic performance of the minaret of Al-Umayyad mosque has been discussed and expressed in detail. Moreover, the strengthening interventions to increase the seismic safety have also discussed. The following conclusions are made based on above results;

The seismic analysis was performed on two different models in this project. One with only outer boundary and the other with stairs and central pillar. It can be said based on above results that the contribution of central pillar and stairs in the seismic response of the minaret is not significant. It does not provide substantial stiffness to the minaret.

The limiting value of the peak ground acceleration of the minaret was figured out using response spectrum analysis. The limiting value of PGA for a model without internal contribution under bending was 6.3% and 1.5% under shear. While, these values reduced to 6% and 1.3% for the complete model under bending and shear respectively. It can be seen that the difference of values of PGA in not significant.

The value of modulus of elasticity of the material used in the project is 900 Mpa. It was desirable to increase the seismic response i.e. PGA of the minaret by increasing the modulus of elasticity. It was seen that the increase in the value of elastic modulus does not provide any benefit in terms of increase in the seismic capacity of the minaret. However, the increase in elastic modulus increases the strength of the material. As our main task was to increase the seismic response, therefore it is not recommended to increase elastic modulus for this purpose.

Another strengthening intervention which was used in the project is the provision of vertical reinforcement. The reinforcement is provided at different ratios. The optimum value which favors the seismic safety of the minaret is 0.000269% and this input increases the value of the peak ground acceleration from 6.3% to 16.66%. After this much value of steel ratio, the value of PGA starts decreasing but it stays more than 10% (initial limit value) even at 0.00242%.

The strengthening intervention to increase the behavior of the minaret in shear is the hooping reinforcement. The effect of the value of the hoop reinforcement spacing (s) was studied on the value of peak ground acceleration of the minaret. It was concluded that shear reinforcement spacing of 3000 mm having 13 bars bars of 20 mm diameter provides best results and increases the value of the PGA from 1.5% to 7.4%. There was not considerable change in the value of the PGA with respect to the value of the shear reinforcement spacing.

6.2. Research Prospects

The aforementioned results can be more precise and reliable if following considerations are taken into account;

- This project is done considering linear elastic behavior of the material. The further research can be carried out considering non-linear material behavior. Therefore, results can be highly affected because of non-linearity of the material. The consideration of the non-linearity changes the limit situation of the minaret under bending and shear and hence it changes the results.

- In Abaqus, only the homogenous material can be defined. Since there can be heterogeneity in the minaret structure because of the non-homogeneity of the stone blocks material. Moreover, the presence of the mortar between the stone blocks cannot be modelled in the Abaqus. Hence, due to limitation of software, it can be said that more precise results can be achieved if above mentioned features are taken into account by using a software which can model those features.
- During the modeling of complete model, the stairs are defined as shells. The further research can be done considering stairs as solid elements. The results can be different and accurate because kinematic compatibility conditions will change between the solid-solid elements.

Acknowledge

I gratefully acknowledge the help of my supervisor, Prof. Claudio Chesi, who has offered me valuable suggestions in the academic studies. During the writing of thesis, he gave a lot of insightful suggestions about assessing the seismic performance in different aspects. He provided authentic and concrete data which really helped me to achieve results and understand the problem. Without his patient instruction, insightful criticism and expert guidance, the completion of this thesis would not have been possible.

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