

**INVESTIGATION OF COLD-FORMED STEEL CONCRETE FRAME
BUILDING WITH STRAWBALE INFILL PANEL UNDER SEISMIC
LOADING**



A Ph.D.THESIS

BY

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DECLARATION PAGE

I hereby declare that this Thesis is a record of my research work. It has neither been presented nor accepted in any previous application for higher degree.

Oyeleke, Joseph Kehinde

Date

DEDICATION

I dedicate this work to the Almighty God the giver of life.

ACKNOWLEDGMENTS

To God be the glory, FOREVER AND EVER. Amen.

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ABSTRACT`

Earthquake with its damaging occurrence along fault line causes failure of buildings. This could be reduced by introducing materials that can withstand seismicity as against large sections used in conventional frame building (CFB). Conventional method of anti-seismic technique increases the stiffness of structures by enlarging structural elements, which accommodates seismic load because of the added mass to structures, this increase the weight of building while the safety level is little improved. The aim of this study was to investigate Cold-formed steel Concrete (CSC) frame building with Strawbale infill panels (SIP) under seismic loadings. The objectives were to: (i) assess a 4 storey CSC frame symmetric building with strawbale infill panel; (ii) perform numerical modeling and analysis of the CSC frame building with SIP; (iii) determine the building response to seismic loadings; and (iv) compare the behaviour of CSC frame building model with SIP to the behaviour of CFB model with sandcrete block masonry infill panels.

The 4 storey CSC frame building model with SIP was assessed by identifying capabilities of its material properties to resist seismic loading activities. The building geometry, location of soil conditions, location of building with respect to seismicity and ductility parameters and applied loads were considered in the modeling of the building. Numerical modeling of CSC frame building with SIP was done and subjected to seismic load using Response Spectrum Analysis (RSA) and 5 Time History (TH) data. Data obtained from Haiti earthquake of 2010 was used for the RSA in accordance with Eurocode8, while the 5 TH data were obtained from Elcentro, Petrolia, Northridge, Lucerne and Slymarff earthquakes. ETABS 2016 software was employed for the modeling and analysis of the building and the results of both analyses were subjected to safety verification in accordance with Eurocode 8. A similar CFB was then modelled and compared to the CSC building with SIP.

The findings of the study were that:

- (i) the CSC frame building with SIP offered 25.3% more of resistance to external forces;
- (ii) building model showed a total weight of 16067.11KN and modal analysis of first 6 modes ranged between 0.644 to 0.117 periods;

- (iii) response of building model under RSA revealed that maximum displacement of 127.85mm occurred at the top of the building while the maximum drift of 0.016 occurred on the first storey.
- (iv) five THanalysis showed that the maximum displacement occurred at the top level for all five earthquake data used, with Petrolia earthquake data having the highest displacement value of 50.35 mm, and a maximum storey drift value of 0.0068; and
- (v) CSC frame building with SIP under seismic load has a storey drift of 0.0068 compared to CFB with drift value of 0.00912.

The study concluded that the maximum displacement and drift of CSC frame building with strawbale infill panel under seismic load occurred at the top of the building. The study recommended that CSC building with SIP be incorporated in building standard, as this composition is more stable under earthquake than CFB.

CHAPTER ONE

1.0 INTRODUCTION

Almost every year, earthquake occurs somewhere in the world and captures public attention. Meanwhile, thousands of minute tremors often go unnoticed every day. It is only noticed when its effect is felt as a result of disturbances on the ecosystem as well as recorded. Although, usually the ground is often considered to be solid and stable, the earth is constantly shifting under our feet.

Earthquake being among the most damaging events caused by the earth itself, to reduce the risk, ensuring the resilience of lifeline infrastructures to resist high seismic occurrences in populated residential areas is of high priority in any seismic design and mitigation plan. An example of a lifeline infrastructure is a building structure. Nowadays, different methods are used in strengthening buildings against earthquake impacts. According to Azizian (2015), the use of light steel framing (LSF) is one of the new methods which can be used in improving the seismic behaviour of building structures. The LSF (cold-formed steel structure is an example) is a structural system which is employed in the construction of buildings that are mostly low rise and medium rise storey buildings which are five stories at most.

Earthquake which is a natural disaster discovered to be caused by the movement of earth's tectonic plates has also been studied and believed by researchers to also be caused by man activities. Activities that disturb the earth-crust like drilling of oil, mining of coal, boring of piles, mine blasts, landslides, nuclear experiments, and many more can trigger the occurrence of earthquake. The Nigerian Association of Water-Well Drilling Rig Owners and Practitioners (AWDROP) proposed that the indiscriminate drilling of boreholes that is not in-line with code of practice has been a major fact that is responsible for the earth tremors (minor earthquakes) that have been occurring in Nigeria (Ogunyemi, 2016).

Various cases of minor earthquakes known as tremors have been recorded in Nigeria, and most recently in Mpape, Abuja on September 7, 2018 (Essel, 2018). Abolarin and Adedeji (2016) recorded in the year 2016, about four different cases of earthquake scare in the form of earth-tremors which resulted in the continuous vibration of the earth (ground) at intervals and is accompanied by loud noises from the earth-crust. Tsalha et al. (2015) researched the various occurrences in Nigeria, and the first case of recorded tremor was found in Warri in the year 1933, and since then numerous cases of tremors have also been recorded. Akpan and Yakubu (2010) have associated these earth tremor cases with the Ifewara – Zungeru fault.

Insuring the adaptability of building structure against high seismic activities in huge populated areas is the goal of Engineers and Scientist in seismic design and mitigation plan. Scientists can forecast where earthquakes are likely to happen, however, as a result of the earth's plate movement and the fault zones location, they can also reach unqualified conclusions or guess work about when earthquakes could occur in certain locations, by looking at historical records of past earthquakes in the area and detecting where pressure is building along fault lines. But these predictions as result of guess work may not be reliable because, the release of strain along one part of a fault system, actually causes an increased strain on another part (Raju et al., 2015). As urbanization advances rapidly worldwide, there is serious threat to lives and properties by earthquakes (Narasimha et al., 2014). The mitigation of the seismic risk is a complex task, which requires the cooperation of scientists, engineers and decision makers, and that has to be approached at different time scales.

The movement toward alternative building systems has come about as the result of a lot of factors, some of which include the depletion of forest resources, and concerns about energy efficiency and construction costs. Strawbale buildings are of special value in areas where earthquakes are common since they have a good aspect ratio (i.e. width to height ratio) and can be reinforced easily (Ashour, 2003). The strawbale walls absorb an appreciable percentage of the shock of the earthquake and the plastering on the wall adds to the strength of the strawbale building. The material properties of strawbales - their strength and flexibility - make them a fit and also ideal for seismic-resistant structural design. Cold-formed steel have also been used in construction for a long time in building construction. Today, as a result of its high construction quality, high speed, and high resistance against highest extent or degree of earthquake movements, it is used in building constructions in developed countries of the world (Azizian, 2015).

In this study, cold-formed steel concrete with strawbale infill panels would be tested for its strength against ground seismic excitations. Since strawbales have been used in earthquake resistance structures, it is therefore used as infill panels to cold-formed concrete composite frames.

1.1 Strawbale

There is a present need in the construction industry to focus on inventing building structures that call for fewer and non-renewable resources, for their construction and maintenance. One of the construction practices which has promised and shown great assurance in achieving this

new aim is construction using straw bale. Straw has been used for thousands of years as a building material. Traditionally straw was mixed with clays in earth construction techniques such as adobe, in order to reinforce the earth, to form a wall, barrier or fence (King, 2006). In the late 1800s, baling machines were discovered in the USA which allowed farmers to store their straw more easily. During this same period, communities started increasing to populate the farm areas of Nebraska. Conventional building materials were in very short supply within these new areas so the farmers started to use the straws (in bales) which were produced from their crops to build houses. These buildings had load-bearing strawbale walls which were then rendered both inside and out. This type of strawbale construction style has been known as the Nebraskan style and a few of these buildings are still in existence as they survive to this day (Jones, 2002). Strawbales can function in its use as both modest load bearing and infill non-load bearing wall applications. Non-loadbearing strawbale walls are mainly used for external infill above the damp-proof course level in moderate or sheltered environments. They can be constructed either on-site or as prefabricated panels delivered to the construction site already enclosed in a protective outer finish, such as lime render. Detailing prerequisite for both methods are the very similar, although their construction procedures are different (Suton et al., 2011). The material properties of straw bales; their flexibility and strength, make them ideal for seismic-resistant structural design.

Strawbale is appropriate for creating healthy living environments and minimizing the ecological impact of the building process – from construction to disposal (Guarneri, 2014). Strawbale has been found to be a highly efficient thermal insulator and high density as building material (Adedeji et al, 2018). Strawbales can be employed in the construction of both load bearing and infill non-loadbearing wall applications. Strawbales can also be used for structural walls that are lightly loaded up to two storeys high (Suton et al., 2011). Figure 1.1 to 1.3 shows stacked strawbale, strawbale in its natural habitat and strawbale infill unit respectively.



Figure 1.1: Staked Strawbale(Suton et al., 2011)



Figure 1.2: Strawbale in its natural Habitat (Suton et al., 2011)



Figure 1.3: Strawbale infill unit (Suton et al., 2011)

1.2 Cold Form Steel-Concrete Composite

Thin sheet made from steel products are extensively used in building industry and they range from floor decking to purlins and roof sheeting. Generally, they are usable as building elements for assembly at the site or as prefabricated frames or panels (Kummar and Kumar,

2002). Their sections are cold-formed, i.e. their manufacturing process includes the forming of steel sections in a cold state without the heat application, from steel sheets of uniform thickness. These are broadly called Cold Formed Steel (CFS) Sections, or Cold Rolled Steel Sections or Light Gauge Steel (LGS) Sections. The steel sheet gauge used in cold-formed construction is usually 1mm to 3mm thickness, and up to 8mm can be formed if the pre-galvanised material is not required for the particular application. As stated earlier, the method of manufacturing is important as it differentiates CFS from Hot Rolled Steel (HRS) sections. Normally, steel sheets used in CFS sections has a yield strength of at least 280N/mm^2 , (Kumar and Kumar, 2002) although, the use of steels of higher strengths is the trend, and sometimes as low as 230N/mm^2 . Figure 1.4 shows the shape of the typical cold-formed steel members.

The use of lightweight cold-formed steels concrete composite led to resistance of building against earthquake due to the reduction of lateral weight on building structure by 20% and material consumption by 50% (Parastesh and Hosseini, 2012).

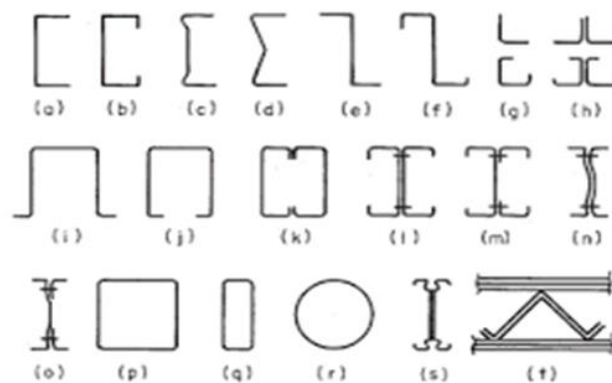


Figure 1.4: Typical Cold-Formed Steel Members (Merritt, 1999)

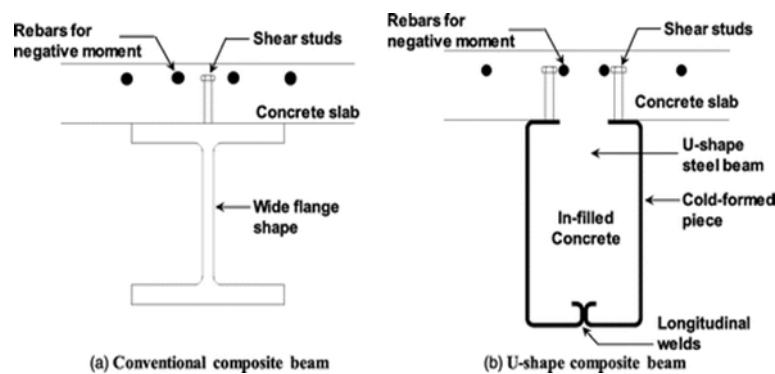


Figure 1.5: Conventional and Cold formed Reinforced Concrete Beam (Lee et al., 2013)

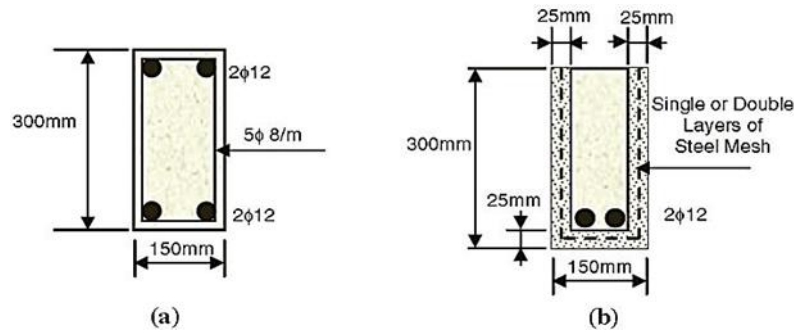


Figure 1.6: Conventional Reinforced Concrete Beam (Lee et al., 2013)

1.3 In-filled Frame and Non-In-filled Structural Frame

Reinforced concrete frame structures are usually the most common type of building structure construction, due to its ease of construction and its rapid work progression (Kaple et al, 2016). Reinforced concrete frame structures are most times masonry in-filled walls and are most commonly employed in the world. The term "in-filled frame" designates a composite structure formed by one or more infill panels surrounded by a frame (Crisafulli et al, 2000). Different terminology is normally used depending on the constructive techniques. In some cases, the term in-filled frame refers to the situation in which the frame is built first and then in-filled with one or more masonry panels or wall units.

In-filled frame may be defined as a combination of moment resisting plane frame and infill wall. In-fills are adequately separated from the RC frame such that it does not interfere with the frame under lateral deflections or deformations. The entire lateral force on the building is carried by only the reinforced concrete frame alone. In-fills are built integral with the reinforced concrete frame but considered as non-structural elements in structural analysis (Azizian, 2015), only the contribution of its mass is considered and are treated so and not considered in the design of frame structures. In-filled frame is considered as bare frame during structural analysis. This is the most common design practice in the developing countries and especially earthquake-prone areas such as India.

Non-in-filled frames (bare frames) are reinforced frame structure without in-filled wall panels or wall units. Frames behaviour under lateral loads: bare frame, predominant frame action; in-filled frame, predominant truss action (Kaushik et al., 2006); the in-filled RC framed building behaves differently as compared to a bare framed building (with no infill wall units) under lateral load. A bare frame is a lot stiffer than a fully in-filled frame; it resists the lateral

load that is applied through frame action when this frame is merged with in-filled walls, truss action is introduced (Kaple et al., 2016). A fully in-filled frame depicts less deformation or deflection, although it attracts higher base shear (due to increased stiffness).

Various materials are employed to create infill walls that span between floors in steel or reinforced concrete frame building structure. Traditionally, infill walls are concrete or timber, masonry, but the most recent form of construction employs the use of light steel C sections which span between the floors and around openings. Masonry walls constructed from sandcrete blocks are the conventional form of infill wall construction. Concrete infill walls are commonly large precast concrete panels or walls used in lift shafts and are storey high and often of a width dependent on column spacing. These large panels may be bottom supported or top hung. Timber infill walls are specifically standard timber sections of 90mm and 140mm depth to span 2.4m to 3.6m between floors, these are the same in form to light steel infill walls. These Timber sections are sawn to length and are staked at 400mm or 600mm spacing. The timber C sections are fixed at an equal spacing depending on the external front elevation materials, and at a spacing which is compatible with sheathing board sizes and standard plasterboard.

1.4 Aim and Objectives

The aim of this study was to investigate Cold-formed steel Concrete (CSC) frame building with Strawbale infill panels (SIP) under seismic loadings.

In cognizance with the aim of the research, the specific objectives are to:

- i. assess a 4 storey CSC frame symmetric building with strawbale infill panel;
- ii. perform numerical modeling and analysis of the CSC frame building with Strawbale infill panel (SIP);
- iii. determine the building response to seismic loadings utilizing Response Spectrum Analysis (RSA) and Time History (TH) data;
- iv. compare the behaviour of CSC frame building model with SIP to the behaviour of Conventional Frame Building CFB model with sandcrete block masonry infill panels.

1.5 Summary of Research Methodology

The assessment of seismic loading activities in common seismic prone areas is carried out and records of severe cases obtained, as in the study carried out by Wu (2014), in which he

studied 2 buildings damaged in the September 2010 and February 2011 earthquakes in Christchurch, New Zealand.

Numerical Modeling of a 4 storey cold-formed steel frame building with strawbale infill panel is carried out as in the study by Rajashin and Pajgade (2015). In their study a 5 storey reinforced concrete building was modeled and analyzed for dead load, live load and earthquake load by the use of standard computer package ETABS v 9.6.0 and load combinations made according to CL6.3 of IS 1892-2002. For this study, ETABS 2016 software is also employed for the modeling and analysis of the structure using response spectrum in accordance to Eurocode 8. Our ground motion records are obtained from Elcentro, Petrolia, Northridge, Lucerne and Slymarff earthquakes while the analytical model for the ground motion recorded as employed by Adedeji and Ige (2011) is also obtained from the Elcentro earthquake record alone.

Load response comparison caused by the seismic activity on the infill frame structure is done using response spectrum analysis and time history methods as done by Baghari et al. (2011). In this study, load response comparism was carried out using time history method of seismic analysis.

1.6 Problem Statement

The conventional and traditional method of the anti-seismic technique is to increase the stiffness of building structures by increasing the sectional elements such as columns, beams and shear walls, or other structural elements in a building, to accommodate the seismic load as a result of the added mass to the structure. Emmanuelle (2013) observed that this traditional anti-seismic method causes an increase in the cost of the structure while the safety level of such building structure is not really improved. Another drawback with the conventional and traditional anti - seismic technique is that it focus its attention is on the protection of the building structures, neglecting the facilities inside the structure. Thus, it cannot be rely upon in some building structures.

In many countries, large quantities of straws produced as agricultural wastes are burned, causing severe air quality problems. Morrison (2011) identifies the qualities of strawbale material as seismic resistance when used in building construction.

Various researchers have used the response spectrum method, which is a linear dynamic method of seismic analysis and employs the use of Peak Ground Acceleration (PGA) of a

ground motion which are recorded during an earthquake to analyse building structures. However, it does not depict the live representation of an earthquake effect on the building. Hence, there is the need to check the model by subjecting it to historical earthquake data by time history method of seismic analysis. These data were recorded during the occurrence of an earthquake.

1.7 Justification of Study

Elouali (2004) observed that when subjected to seismic excitations, the behavior of the building is influenced by infill panel structural elements in earthquake's horizontal loads, i.e. the failure observed during earthquakes were linked to the action of infill panels placed in the reinforced concrete frames. This study would reveal the behavior of straw bale as an infill panel with cold-formed steel frame building subjected to a seismic load. The outcome of this study would add to the body of engineering knowledge and equally justify the need to incorporate straw bale as an infill panel in frame structures in the design of frame structural elements and should be incorporated in the international standards, so that stakeholders in the construction field, as well as students and professionals, can benefit from it.

1.8 Scope of Work

The scope of this research covers the seismic analysis of a cold-formed steel concrete composite with plastered straw-bale walled building which is subjected to response spectrum method of earthquake loading in order to obtain the response of the structure. This includes the following:

- Response Spectrum Analysis for the structure
- Time History Analysis of the structure by five (5) different earthquake acceleration records.

CHAPTER TWO

2.0 LITERATURE REVIEW

2.1 General Overview

The earth's crust is a thin but hard shell which floats on denser hotter rock of the mantle. The earth's crust is divided into fractions which are known as tectonic plates that are constantly in motion and grinding past each other at boundaries known as faults. The tectonic plates rub along rough patches of rock as they slide past one another and lock together like a Velcro. However even though the fault boundaries lock together the plates are still moving pulling at the entangled sections. The pulling can further create cracks in the earth's crust and thereby creating more faults near the boundaries' plate. An earthquake can be said to takes place when the built up pressure along a fault lines becomes stronger than the pressure holding the rocks together. Then the rocks on either side of the fault suddenly rip apart sometimes at great speeds. The two sides of the fault lines move in continuous contact past one another causing friction and discharging the pent-up pressure. Energy from the separation of the faults radiates outward in all directions including towards the surface that it is felt as an earthquake.

Simply put earthquakes occur as a result of a sudden slip on a fault. The tectonic plates are moving slowly regularly but as a result of friction they get stuck at their edges. When the stresses on the edge overcomes the friction the energy released in waves form travel through the earth's crust and cause the shaking earthquake that we feel.

Earthquakes display themselves at the earth's surface by shaking and also displacing the ground. When a large epicentre of an earthquake is located at an offshore the seabed undergoes lots of displacement sometimes which leads to tsunami. The shakings due to earthquake leads to landslides.

While most earthquakes are caused by the movement of the earth's tectonic plates human activities can also lead to the occurrence of earthquakes. It has been researched by many researchers that human activities like drilling and mining of oil and coal respectively boring of piles mine blasts landslides nuclear experiments and other human activities that can disturb the earth-crust can trigger earthquakes.

In order to understand the effect of earthquakes on structures it is important to review its fatalities. Wai-Feh and Charles 2003 deduced that about 17000 persons were killed per year

in the twentieth century. Table 2.1 shows earthquake occurrences since 1900 with fatalities greater than 1000.

Table 2.1: Earthquake Occurrences since 1900 with Fatalities Greater than 1000

Year	Day & Month	Location	Death	Magnitude
1902	19-Apr	Guatemala	2000	7.5
	16-Dec	Turkestan	4500	6.4
1903	19-Apr	Turkey	1700	
	28-Apr	Turkey	2200	6.3
1905	04-Apr	India Kangra	19000	8.6
	08-Sep	Italy Calabria	2500	7.9
1906	31-Jan	Colombia	1000	8.9
	16-Mar	Taiwan Kagi	1300	7.1
	18-Apr	San Francisco CA	2000	8.3
	17-Aug	Chile Santiago	20000	8.6
1907	14-Jan	Jamaica Kingston	1600	6.5
	21-Oct	Central Asia	12000	8.1
1908	28-Dec	Italy Messina	70000	7.5
1909	23-Jan	Iran	5500	7.3
1912	09-Aug	Turkey Marmara Sea	1950	7.8
1915	13-Jan	Italy Avezzano	29980	7.5
1917	21-Jan	Indonesia Bali	15000	
	30-Jul	China	1800	6.5
1918	13-Feb	China Canton	10000	7.3
1920	16-Dec	China Gansu	200000	8.6
1923	24-Mar	China	5000	7.3
	25-May	Iran	2200	5.7
	01-Sep	Japan Kanto	143000	8.3
1925	16-Mar	China Yunnan	5000	7.1

Table 2.1 Contd.

Year	Day & Month	Location	Death	Magnitude
1927	07-Mar	Japan Tango	3020	7.9
	22-May	China nr Xining	200000	8.3
1929	01-May	Iran	3300	7.4
1930	06-May	Iran	2500	7.2
	23-Jul	Italy	1430	6.5
1931	31-Mar	Nicaragua	2400	5.6
1932	25-Dec	China Gansu	70000	7.6
1933	02-Mar	Japan Sanriku	2990	8.9
	25-Aug	China	10000	7.4
1934	15-Jan	India Bihar- Nepal	10700	8.4
1935	20-Apr	Formosa	3280	7.1
	30-May	Pakistan Quetta	30000	7.5
	16-Jul	Taiwan	2700	6.5
1939	25-Jan	Chile Chillan	28000	8.3
	26-Dec	Turkey Erzincan	30000	8
1940	10-Nov	Romania	1000	7.3
1942	26-Nov	Turkey	4000	7.6
	20-Dec	Turkey Erbaa	3000	7.3
1943	10-Sep	Japan Tottori	1190	7.4
	26-Nov	Turkey	4000	7.6
1944	15-Jan	Argentina San Juan	5000	7.8
	01-Feb	Turkey	2800	7.4
	07-Dec	Japan Tonankai	1000	8.3
1945	12-Jan	Japan Mikawa	1900	7.1
	27-Nov	Iran	4000	8.2

Table 2.1 Contd.

Year	Day & Month	Location	Death	Magnitude
1946	31-May	Turkey	1300	6
	10-Nov	Peru Ancash	1400	7.3
	20-Dec	Japan Tonankai	1330	8.4
1948	28-Jun	Japan Fukui	5390	7.3
	05-Oct	Turkmenistan	110000	7.3
1949	05-Aug	Ecuador Ambato	6000	6.8
1950	15-Aug	India Assam; Tibet	1530	8.7
1954	09-Sep	Algeria Orleansvl	1250	6.8
1957	27-Jun	USSR Russia	1200	
	02-Jul	Iran	1200	7.4
	13-Dec	Iran	1130	7.3
1960	29-Feb	Morocco Agadir	10000	5.9
	22-May	Chile	4000	9.5
1962	01-Sep	Iran Qazvin	12230	7.3
1963	26-Jul	Yugoslavia Skopje	1100	6
1966	19-Aug	Turkey Varto	2520	7.1
1968	31-Aug	Iran	12000	7.3
1969	25-Jul	Eastern China	3000	5.9
1970	04-Jan	Yunnan China	10000	7.5
	28-Mar	Turkey Gediz	1100	7.3
	31-May	Peru	66000	7.8
1972	10-Apr	Iran southern	5054	7.1
	23-Dec	Nicaragua	5000	6.2
1974	10-May	China	20000	6.8
	28-Dec	Pakistan	5300	6.2

Table 2.1 Contd.

Year	Day & Month	Location	Death	Magnitude
1975	04-Feb	China	10000	7.4
	06-Sep	Turkey	2300	6.7
1976	04-Feb	Guatemala	23000	7.5
	06-May	Italy northeastern	1000	6.5
	25-Jun	New Guinea	422	7.1
	27-Jul	China Tangshan	255000	8
	16-Aug	Philippines	8000	7.9
	24-Nov	Iran-USSR border	5000	7.3
1977	04-Mar	Romania	1500	7.2
1978	16-Sep	Iran Tabas	15000	7.8
1980	10-Oct	Algeria El Asnam	3500	7.7
	23-Nov	Italy southern	3000	7.2
1981	11-Jun	Iran southern	3000	6.9
	28-Jul	Iran southern	1500	7.3
1982	13-Dec	W. Arabian Peninsula	2800	6
1983	30-Oct	Turkey	1342	6.9
1985	19-Sep	Mexico Michoacan	9500	8.1
1986	10-Oct	El Salvador	1000	5.5
1987	06-Mar	Colombia- Ecuador	1000	7
1988	20-Aug	Nepal-India border	1450	6.6
	07-Dec	Armenia Spitak	25000	7
1990	20-Jun	Iran western	40000	7.7
	16-Jul	Philippines Luzon	1621	7.8

Table 2.1 Contd.

Year	Day & Month	Location	Death	Magnitude
1991	19-Oct	India northern	2000	7
1992	12-Dec	Indonesia Flores	2500	7.5
1993	29-Sep	India southern	9748	6.3
1995	16-Jan	Japan Kobe	6000	6.9
	27-May	Sakhalin Island	1989	7.5
1997	10-May	Iran northern	1560	7.5
1998	04-Feb	Afghanistan	2323	6.1
	30-May	Afghanistan	4000	6.9
	17-Jul	Papua New Guinea	2183	7.1
1999	25-Jan	Colombia	1185	6.3
	17-Aug	Turkey	17118	7.4
	20-Sep	Taiwan	2297	7.6
2001	26-Jan	India Bhuj	19988	7.7
2003	21-May	Northern Algeria	2266	6.8
	26-Dec	Southeastern Iran	29000	6.6
2004	26-Dec	Northern Sumatra Indonesia	265000	9.1
2005	28-Mar	Northern Sumatra Indonesia	1313	8.6
2005	08-Oct	Pakistan	93500	7.6
2006	26-May	Java Indonesia	5782	6.3
2008	12-May	Sichuan China	88000	7.9
2009	30-Sep	Southern Sumatra Indonesia	1115	7.6

Table 2.1 Contd.

Year	Day & Month	Location	Death	Magnitude
-------------	------------------------	-----------------	--------------	------------------

2010	12-Jan	Haiti	222570	7.0
	14-Apr	Qinghai China	3000	6.9
2011	11-Mar	Honshu Japan	20896	9.0
2015	25-Apr	Nepal	9018	7.8

Source: Wai-Feh and Charles 2003; Abolarin and Adedeji 2016.

2.1.1 Effect of Earthquakes

The massive loss of lives and damages to properties during earthquakes are caused by diverse forms in which earthquake manifest itself. Examples of these effects are shaking of the ground landslides liquefaction sand volcanoes disrupted bedding and tsunamis.

2.1.1.1 Ground Shaking

The principal cause of damages on structures instigated by an earthquake is ground shaking. As the earth vibrates all the building structures on the surface of the ground responds to the vibration in varying degrees depending on the closeness to the source of earthquake. Rapid horizontal movements which earthquake brings may shift homes from their foundations and results in the collapse of tall buildings. These vibrations weaken the joints of the structural members and also weaken the reinforcement of the members which eventually leads to the failure and collapse of the building.

A shaking of low frequency might cause a tall building to shake rigorously while having a small effect on a nearby low-rise building. Whereas a higher frequency of vibration might have an opposite effect. The motion of upper storeys is greater than that of the lower storeys and that makes the damage to tall buildings often concentrated on the upper storeys. Pounding by collisions with adjacent or close buildings is another common cause of damage during earthquakes.

The severity of the shaking during earthquakes depends on the magnitude of the earthquake distance from hypocentre and nature of the substrate location unconsolidated rock shakes a lot more basins can amplify waves and the frequency of the seismic waves high-frequency waves do most damage but do not travel very far

The shaking caused by earthquakes causes' movement in all principal axes. These axes include up and down left and right and forward and back. Ground shaking may vary over an area depending on factors like topography type of bedrock and the orientation and location of the fault rupture. These factors affect the manner seismic waves travel through the ground. If

the shaking intensity generated by an earthquake is high building structures can be damaged completely. Figure 2.1 shows a three storey building damaged by ground shaking.



Figure 2.1: A damaged building from the Marina District in San Francisco after the Loma Prieta earthquake in 1989 Anderson 1997

2.1.1.2 Landslides

Unstable regions of mountains or hillsides may fail under stresses of seismic waves. An earthquake that occurred in 1970 off the coast of Peru produced a landslide that starts about 80 miles from the earthquake. The slide which has an estimated height of 30 metrestravelled at speed more than one hundred and sixty kilometre 160km per hour destroyed a village and ploughed through another village killing more than 18000 people Ammon 2005.

Landslides are caused by direct rupture and by the sustained shaking of unstable slopes. They can destroy buildings in their path easily or block roads and railroad lines. They can also dam rivers like the Hebgen Lake earthquake of magnitude 7.1 which occurred on 17 August 1959 in Montana Anderson 1997. Figure 2.2 is an image of a massive landslide that fell across Highway 17 in the Santa Cruz Mountains after the Loma Prieta earthquake. The concrete blocks are broken chunks of the centre divider on the highway. The road which was a major route was closed for several weeks after the earthquake



Figure 2.2: Broken chunks of concrete blocks on a Highway Anderson 1997.

2.1.1.3 Liquefaction and Sand Volcanoes

In liquefaction the soil loses its ability to resist shear and flows like quick-sand and thereby making all that relies on the substrata for support to shift rupture tilt or collapse. Soil liquefaction can occur in low-density saturated sands of relatively uniform size. The phenomenon of liquefaction is particularly important for dams bridges underground pipelines and buildings standing on such ground.

Liquefaction can occur on a large scale. The ground may separate into several coherent slump blocks if a basal layer liquefies. This happened to a housing development after a large earthquake near Anchorage Alaska in 1964.

Figure 2.3 shows the Kwangishicho Apartments in Niigata Japan. Three of the buildings due to liquefaction have tilted over. Despite the tilts the buildings were largely intact.



Figure 2.3: The Kwangishicho Apartments in Niigata Japan being tilted due to liquefaction
Ammon 2005

A sand volcano also known as sand blow is a cone of sand formed by discharging of sand onto a surface from a central point. The sand builds up as a cone with slopes at the sand's angle of repose. A crater is usually seen at the top. The cone looks like a small volcanic cone and can increase in size with varied diameter.

Sand volcanoes sand blows and sand dikes occurs as a result of liquefaction of water-saturation of soil and sandy sediment which responds to ground shaking produced by earthquakes of magnitude greater than 5. As seismic waves go through the sediment stresses are built-up in the water between the sand grains if the pore-water pressure increases to the point that it equals the weight of the overlying soil the sediment liquefies whence liquefied the pressurized water with entrained sand intrudes towards the ground surface this forcefully enters the pre-existing cracks and fissures that form as the overlying sediment finding its way into the liquefied sand slides downslope. In the instance where the pressurized slurry of water and sand erupts to the ground surface fountains may be observed and sand may be deposited on the surface around the vent to form a Sand volcano sand blow. Over a period of time soils formed in the sand volcano are buried by other deposits preserving them in the geologic record. Organic material and sediment above and below the sand volcanoes can be used to estimate their ages and earthquakes that caused the sand volcanoes Tuttle 2011.

The process is frequently related to soil liquefaction and the occurrence of sand volcanoes during an earthquake. Many such features were exhibited by New Madrid Seismic zone during the 1811 to 1812 New Madrid earthquakes. Adjacent sand volcanoes aligned in rows along a linear fracture within fine-grained surface sediments are common and are still seen in

the New Madrid area Crone and Wheeler 2000. Figure 2.4 shows an illustration of a cross-section of a modern sand dike superimposed on a photograph of actual sand blows also known as “sand boils” or “sand volcanoes.

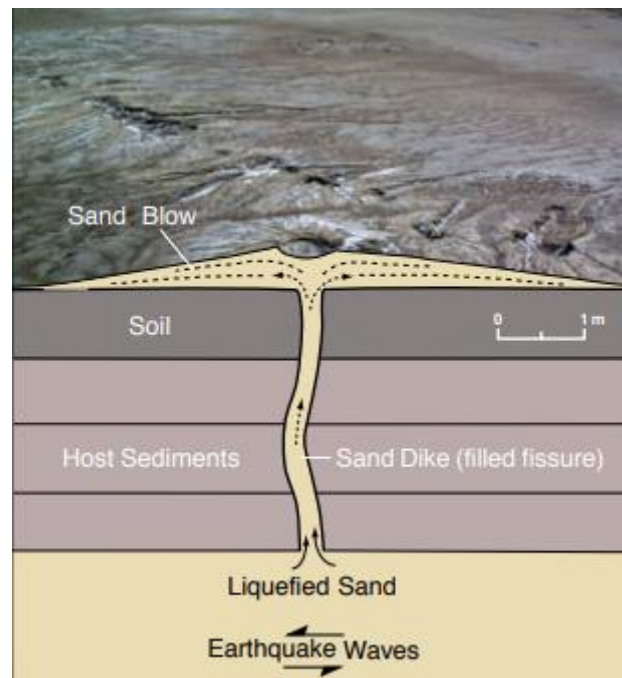


Figure 2.4: Illustration of a cross-section of a modern sand dike superimposed on a photograph of actual sand blows Tuttle 2011

Sand volcanoes were widespread in the Avon - Heathcote Estuary following individual major events of the Christchurch 2010 -2011 Earthquake sequence Reid et al. 2012. The earthquake has a moment magnitude of 6.0. Sand volcanoes were composed of fine grey sand and varied in diameter over the range 0.2 - 5 m fed by vertical to sub-vertical feeder pipes of 15 - 25 cm diameter that cross-cut mud and shell horizons. Tides and wind-driven waves reduced and at times removed the profile of surface features; however subsurface features remained unaffected. Figure 2.5 shows Sand Volcanoes of various sizes that were Recorded in 14th June 2011 following the 13th June 2011 6.0 Mw Earthquake in Christchurch Reid et al. 2012



Figure 2.5: Sand Volcanoes of various sizes Reid et al. 2012

2.1.1.4 Disrupted Bedding/Ground Rupture

Ground rupture is an effect of the earthquake that occurs when the movement of an earthquake along a fault breaks the earth's surface. Earth rupture causes problems by rupturing and disrupting things like pipelines aqueducts tunnels railway lines roads and airport runways. Figure 2.6 is a picture of a scarp formed by the Dixie Valley earthquake of magnitude 6.8 in Nevada 16 December 1954. The miner's shack next to the scarp was undamaged despite being a few metres to the offset. Figures 2.7 and 2.8 shows disrupted beddings in highways and buildings.



Figure 2.6: A scalp formed by the Dixie Valley earthquake in Nevada in 1954 Anderson 1997



Figure 2.7: Disrupted bedding effect on highways Schierle 2003

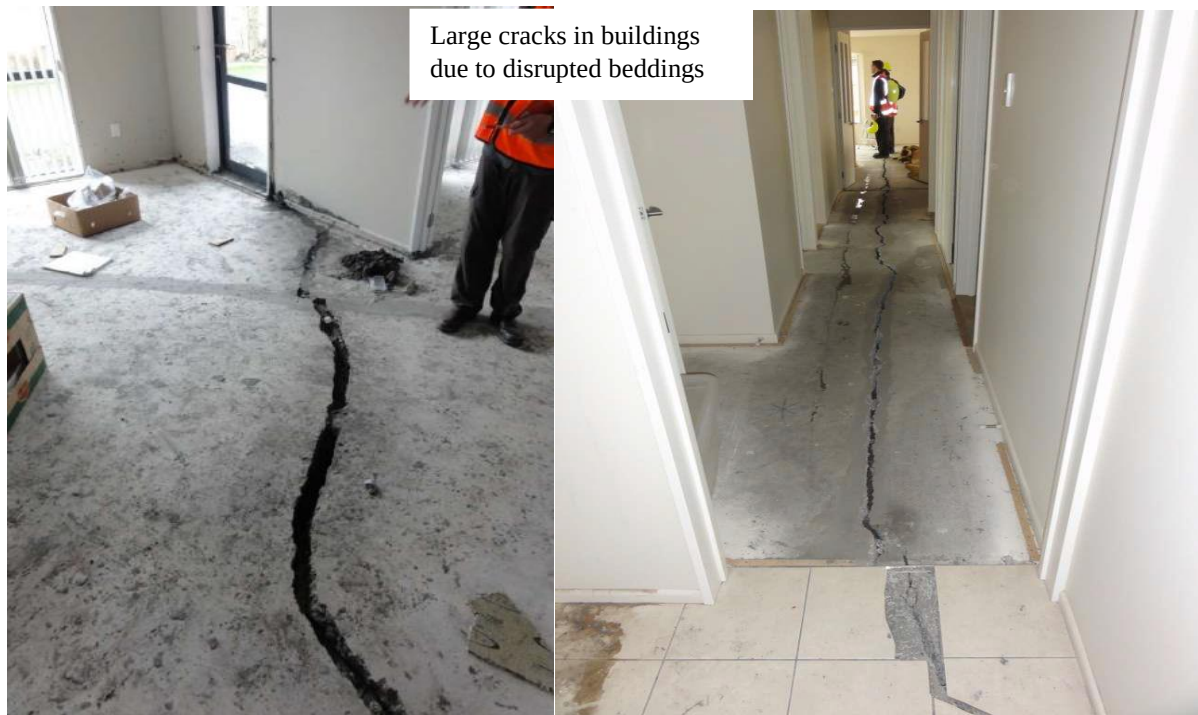


Figure 2.8: Disrupted bedding effect on buildings Schierle 2003

2.1.1.5 Tsunamis

When a large epicentre of an earthquake is located at an offshore the seabed undergoes lots of displacement sometimes which leads to tsunami. Tsunami is a Japanese term which means “harbor wave”. It has been incorrectly referred to as tidal waves. They occur as a result of a sudden vertical movement or offset in the ocean floor during earthquakes. Another process like volcanic deformation and deformation by landslides also generate tsunamis. A sudden

movement changes in the elevation of the ocean and a water wave is initiated which travels outward from the region of sea-floor disruption. Tsunamis can travel for a very long distance all the way across the ocean and large earthquakes in Chile and Alaska have generated waves which caused deaths and damages in regions as far as Hawaii Japan and California Ammon 2005. It is sometimes called seismic sea waves are generally produced by a sudden movement of the ocean floor. As the water waves approach land their velocity decreases and their height increases from 5 to 8 m or even more.

An earthquake of magnitude 9.1 – 9.3 occurred on the Sunda trench megathrust on 26 December 2004 releasing an immense amount of energy and thereby causing over 230000 deaths and over 1000000 people displaced. Entire communities were erased buildings and people during the tsunami as shown in Figure 2.9.



Figure 2.9: Damages after the Sumatra Tsunami Ammon 2005

2.1.1.6 Rockfall

The ground shaking which occurs during earthquakes usually displaces massive object from its resting positions. An example of such object is a rock. During an earthquake a rock from a

very high altitude can travel several kilometres by rolling at a high speed and killing or destroying anything on its path as shown in Figures 2.10 and 2.11.



Figure 2.10: Effect of rock fall from a high altitude Ammon 2005



Figure 2.11: A fallen rock inside a building Ammon 2005

2.1.2 Building Reaction to Earthquakes

Typically an earthquake can cause four types of damage to a building. A building collapse which can result in the total loss of the building and possibly the lives of the occupants. A building can also suffer structural damage which leaves the building standing but unsafe and either result in the subsequent demolition of the building or deploy expensive costs remediation techniques to repair the building structural damages. A building can also suffer non-structural damage to its walls water pipes windows et.c. These costs can be expensive to

repair but are preferable to losing lives. Non-structural damages of buildings usually amount to over 70% of total damage due to earthquakes Schierle 2003. Lastly a building might suffer damage to the contents inside which result from objects not being properly anchored to walls or otherwise properly secured.

The expectations of Engineers and architects are that a building in a seismically prone area would suffer minimal damage but of the four types a designer would prefer cosmetic damage to structural damage or collapse in an effort to preserve life safety. Of course architects would prefer no damage but due to the nature of seismicity earthquakes are unpredictable vary in terms of magnitude strength period and peak ground acceleration. No two earthquakes are alike and the effects of earthquake strength can still surprise engineers and seismologists CUS 1995.

Primarily engineers are concerned with the lateral forces that earthquakes generate. The rationale is that structural engineer already designs for vertical gravity dead loads and live loads. Because designers include a safety factor compensate for unexpected loads in the vertical direction it is assumed that vertical forces are not necessarily the problem in an earthquake. Therefore lateral forces tend to drive earthquake-resistant building design in building codes and in practice. Designers should note that the combination of strong accelerated vertical and lateral forces results into loss of containment in concrete columns supporting buildings as recorded directly over the epicentre in the Northridge earthquake

2.2 Measurement of Earthquakes

Earthquakes are complex multidimensional and highly extraordinary which scientific analysis of requires measurement Wai-Fah and Charles 2003. Prior to the invention of modern scientific instruments earthquakes were measured qualitatively by intensity i.e. by their effect which differed from point to point. With the invention of seismometers an instrument that quantifies the entire earthquake event that is the unique magnitude of the earthquake event became possible. Up till date these are the two most widely used methods of measures for an earthquake and a number of different scales for each have been developed. These are sometimes confused. Engineering design however requires measurement of earthquake in units such as force or displacement. Earthquake magnitude and intensity are analogous to a light bulb and the light it emits. A particular light bulb has only one energy level or wattage. For example for a 100watts bulb the nearer the light bulb the brighter the light intensity while

the farther away it decreased light intensity. That is a particular earthquake has only one magnitude value but many intensity values. Wai-Fah and Charles 2003.

2.2.1 Earthquake Magnitude

An earthquake is unique in the release of strain energy. The quantification of this energy has formed the basis for measuring earthquake events. Richter was the first to define earthquake magnitude as:

$$M_L = \text{Log } A - \text{Log } A_0 \quad 2.1$$

where:

M_L is local magnitude which Richter only defined for Southern California

A is the maximum trace amplitude in microns recorded on a standard Wood–Anderson short-period torsion seismometer at a site 100 km from the epicentre and

$\text{Log } A_0$ is a standard value as a function of distance for instruments located at distances other than 100 km and less than 600 km.

Afterwards a number of other magnitudes have been defined the most important are as follows; surface wave magnitude M_s body-wave magnitude M_b and moment magnitude M_w . Because M_L was only locally defined for California that is for events within about 600 km of the observing stations surface wave magnitude M_s was defined similarly to M_L using teleseismic observations of surface waves of 20 seconds period. Magnitude which is defined on the basis of the amplitude of ground displacements can be related to the total energy in the expanding wavefront generated by an earthquake and thus to the total energy emitted. An empirical equation by Richter is:

$$\text{Log}_{10} E_s = 11.8 + 1.5 M_s \quad 2.2$$

Where:

E_s is the total energy in ergs and

M_s is surface wave magnitude.

It should be noted that $10^{1.5} = 31.6$ so that an increase of one magnitude unit is equivalent to 31.6 times more energy release; two magnitude units increase is equivalent to 1000 times more energy. Subsequently due to the observation that deep-focus earthquakes always do not

register measurable surface waves with periods near 20 second a body wave magnitude M_b was defined which can be related to M_s Darragh et al. 1994:

$$M_b = 2.5 + 0.63sM_s \quad 2.3$$

M_b Body wave magnitudes are more frequently used in eastern North America due to the deeper earthquakes there. A number of other magnitudes scales have been created most of which tend to saturate to an upper bound due to larger earthquakes radiating large amounts of energy at periods longer than which is used for determining the magnitude for instance for M_s defined by measuring 20-seconds surface waves saturation occurs at about $M_s > 7.5$. Recently seismic moment has been used to define moment magnitude M_w Hanks and Kanamori 1979; which is finding increased and widespread use:

$$\text{Log}M_0 = 1.5M_w + 16.0 \quad 2.4$$

where seismic moment M_0 dyne-cm is defined as

$$M_0 = \mu A \bar{u} \quad 2.5$$

where:

μ is the material shear modulus

A is the area of fault plane rupture and

$\bar{u}$ is the mean relative displacement between the two sides of the fault the averaged fault slip.

Comparatively M_w and M_s are numerically almost identical up to magnitude 7.5.

Figure 2.12 indicates the relationship between moment magnitude and various magnitude scales.

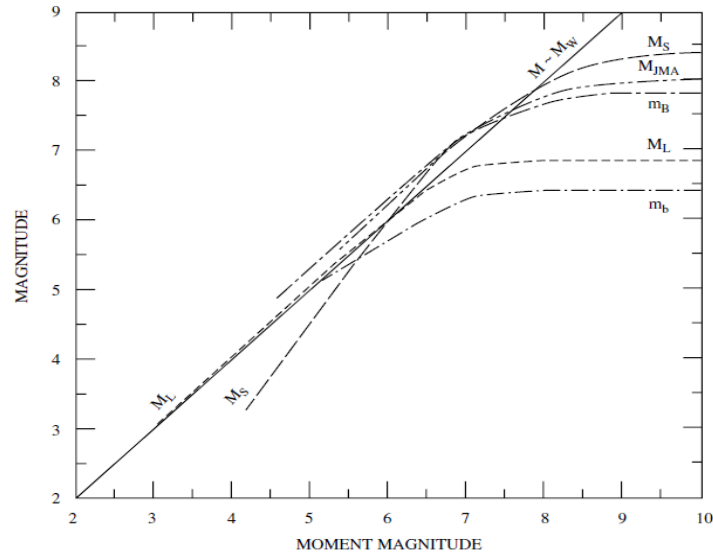


Figure 2.12: Relationship between moment magnitude and various magnitude scales Campbell 1985

Table 2.2 gives an approximate categorization for earthquake magnitude Wai-Fah and Lui 2006.

Table 2.2: Approximate categorization for the magnitude of earthquakes

Earthquake:	Micro	Small	Moderate	Large	Great
Magnitude:	Not Felt	< 5	5 – 6.5	6.5 – 8	> 8

Source: Wai-Fah and Lui 2006

From the foregoing discussion it can be seen that magnitude of an earthquake and its energy are related to fault rupture length and slip. Studies have been done to determine the statistical relations between these parameters i.e. earthquake magnitude and energy for worldwide and regional data sets aggregated and segregated by type of faulting normal reverse and strike-slip. Bonilla et al.'s 1984 worldwide results for all types of faults are:

$$M_s = 6.95 + 0.723 \log_{10} d \quad 2.6$$

$$\log_{10} d = -3.58 + 0.550 M_s \quad 2.7$$

which indicates for example that for $M_s = 7$ the average fault rupture length is about 36 km and the average displacement is about 1.86 m and s indicates standard deviation.

Conversely a fault of 100 km length is capable of about a $M_s = 7.55$ event.

2.2.2 Earthquake Intensity

The Earthquake intensity is a non-instrumental perceptive measure of damage to building structures effects of ground surface and human reactions to the earthquake shaking. It is a descriptive method which has been used traditionally to establish and quantify earthquake size especially for pre-instrumental events.

In other words earthquake intensity describes how much ground shaking occurred or how “strong” an earthquake was at a particular location. The waves of an earthquake weaken as they travel away from the source. In other words an earthquake becomes less strong the further away from the source. The intensity of an earthquake shaking at location depends on the magnitude of the earthquake i.e. its dependent on how much energy that was released and how deep and how far away it was. Local soils and their topography and geology also influence the rate of earthquake shaking. The severity of earthquake shaking is assessed using a descriptive scale – the Modified Mercalli intensity scale. In general seismic intensity is the strength of an earthquake hazard at a particular location. While the term can be generically applied to engineering measures such as peak ground acceleration PGA. It is usually reserved for qualitative measures of location-specific earthquake effects based on observed human behaviour and structural damage.

Earthquake intensities are mostly obtained from interviews of the victims and observers after an earthquake event. Intensity observation helps to provide information that aids characterizing the ground shaking distribution in a region.

Discrete scales usually used to quantify seismic intensities with the levels being represented by Roman numerals and the degree of each intensity providing a basic description of earthquake effects. There are numerous intensity scales developed in pre-instrumental times. The most common in use today according to Anbazhagan 2008 are;

1. Mercalli-Cancani-Seiberg MCS: 12-level scale used in southern Europe as shown in Table 2.3.
2. Modified Mercalli MM: 12-level scale proposed in 1931 by Wood and Neumann who adapted the MCS scale to the California data set. It is used in North America and several other countries. This is shown in Table 2.4.
3. Medvedev-Sponheuer-Karnik MSK: 12-level scale developed in Central and Eastern Europe and used in several other countries. The MSK intensity scale is shown in Table 2.5.

4. European Macroseismic Scale EMS: 12-level scale adopted since 1998 in Europe. It is a development of the MM scale. It is shown in Table 2.6.
5. Japanese Meteorological Agency JMA: 7-level scale used in Japan. It has been revised over the years and has recently been correlated to the maximum horizontal acceleration of the ground. The JMA scale is presented in Table 2.7.

Table 2.3: Mercalli-Cancani-Seiberg MCS

I.	Instrumental	Not felt by many people unless in favorable conditions.
II.	Weak	Felt only by a few people at best especially on the upper floors of buildings. Delicately suspended objects may swing.
III.	Slight	Felt quite noticeably by people indoors especially on the upper floors of buildings. Many do not recognize it as an earthquake. Standing motorcars may rock slightly. Vibration similar to the passing of a truck. Duration estimated.
IV.	Moderate	Felt indoors by many people outdoors by few people during the day. At night some awakened. Dishes windows doors disturbed; walls make cracking sound. Sensation like heavy truck striking building. Standing motorcars rock noticeably. Dishes and windows rattle alarmingly.
V.	Rather Strong	Felt outside by most may not be felt by some outside in non-favorable conditions. Dishes and windows may break and large bells will ring. Vibrations like large train passing close to house.
VI.	Strong	Felt by all; many frightened and run outdoors walk unsteadily. Windows dishes glassware broke; books fall off shelves; some heavy furniture moved or overturned; a few instances of fallen plaster. Damage slight.
VII.	Very Strong	Difficult to stand; furniture was broken; damage negligible in building of good design and construction; slight to moderate in well-built ordinary structures; considerable damage in poorly built or badly designed structures; some chimneys broken. Noticed by people driving motor cars.
VIII.	Destructive	Damage slight in specially designed structures; considerable in ordinary substantial buildings with partial collapse. Damage great in poorly built structures. Fall of chimneys factory stacks columns monuments walls. Heavy furniture moved.
IX.	Violent	General panic; damage considerable in specially designed structures well-designed frame structures thrown out of plumb. Damage great in substantial buildings with partial collapse. Buildings shifted off foundations.
X.	Intense	Some well built wooden structures destroyed; most masonry and frame structures destroyed with foundation. Rails bent.
XI	Extreme	Extreme Few if any masonry structures remain standing. Bridges destroyed. Rails bent greatly.
XII	Cataclysmic	Total damage - Everything is destroyed. Total destruction. Lines of sight and level distorted. Objects thrown into the air. The ground moves in waves or ripples. Large amounts of rock move position. Landscape altered or leveled by several meters. In some cases even the route of rivers is changed.

Table 2.4: Modified Mercalli Intensity scale of 1931

I	Not felt except by a very few under especially favourable circumstances.
II	Felt only by a few persons at rest especially on upper floors of buildings. Delicately suspended objects may swing.
III	Felt quite noticeably indoors especially on upper floors of buildings but many people do not recognize it as an earthquake. Standing motor cars may rock slightly. Vibration like passing truck. Duration estimated.
IV	During the day felt indoors by many outdoors by few. At night some awakened. Dishes windows and doors disturbed; walls make creaking sound. Sensation like heavy truck striking building. Standing motorcars rock noticeably.
V	Felt by nearly everyone; many awakened. Some dishes windows broken; a few instances of cracked plaster; unstable objects overturned. Disturbance of trees poles and other tall objects sometimes noticed. Pendulum clocks may stop.
VI	Felt by all; many frightened and run outdoors. Some heavy furniture moved; a few instances of fallen plaster or damaged chimneys. Damage slight.
VII	Everybody runs outdoors. Damage negligible in buildings of good design and construction slight to moderate in well-built ordinary structures; considerable in poorly built or badly designed structures. Some chimneys broken. Noticed by persons driving motor cars.
VIII	Damage slight in specially designed structures; considerable in ordinary substantial buildings with partial collapse; great in poorly built structures. Panel walls thrown out of frame structures. Fall of chimneys factory stacks columns monuments walls. Heavy furniture overturned. Sand and mud ejected in small amounts. Changes in well water. Persons driving motor cars disturbed.
IX	Damage considerable in specially designed structures; well-designed frame structures thrown out of plumb; great in substantial buildings with partial collapse. Buildings shifted off foundations. Ground cracked conspicuously. Underground pipes broken.
X	Some well-built wooden structures destroyed; most masonry and frame structures destroyed with foundations; ground badly cracked. Rails bent. Landslides considerable from river banks and steep slopes. Shifted sand and mud. Water splashed over banks.
XI	Few if any masonry structures remain standing. Bridges destroyed. Broad fissures in ground. Underground pipelines completely out of service. Earth slumps and land slips in soft ground. Rails bent greatly.
XII	Damage total. Waves seen on ground surfaces. Lines of sight and level distorted. Objects thrown upward into the air.

Source: Wai-Fah and Lui 2006

Table 2.5: Medvedev-Sponheuer-Karnik MSK

I.	Not perceptible	Not felt registered only by seismographs. No effect on objects. No damage to buildings.
II.	Hardly perceptible	Felt only by individuals at rest. No effect on objects. No damage to buildings
III.	Weak	Felt indoors by a few. Hanging objects swing slightly. No damage to buildings.
IV.	Largely observed	Felt indoors by many and felt outdoors only by very few. A few people are awakened. Moderate vibration. Observers feel a slight trembling or swaying of the building room bed chair etc. China glasses windows and doors rattle. Hanging objects swing. Light furniture shakes visibly in a few cases. No damage to buildings.

Table 2.5 Contd.

V.	Fairly strong	Felt indoors by most outdoors by few. A few people are frightened and run outdoors. Many sleeping people awake. Observers feel a strong shaking or rocking of the whole building room or furniture. Hanging objects swing considerably. China and glasses clatter together. Doors and windows swing open or shut. In a few cases window panes break. Liquids oscillate and may spill from fully filled containers. Animals indoors may become uneasy. Slight damage to a few poorly constructed buildings.
VI.	Strong	Felt by most indoors and by many outdoors. A few persons lose their balance. Many people are frightened and run outdoors. Small objects may fall and furniture may be shifted. Dishes and glassware may break. Farm animals may be frightened. Visible damage to masonry structures cracks in plaster. Isolated cracks on the ground.
VII.	Very strong	Most people are frightened and try to run outdoors. Furniture is shifted and may be overturned. Objects fall from shelves. Water splashes from containers. Serious damage to older buildings masonry chimneys collapse. Small landslides.
VIII.	Damaging	Many people find it difficult to stand even outdoors. Furniture may be overturned. Waves may be seen on very soft ground. Older structures partially collapse or sustain considerable damage. Large cracks and fissures opening up rockfalls.
IX.	Destructive	General panic. People may be forcibly thrown to the ground. Waves are seen on soft ground. Substandard structures collapse. Substantial damage to well-constructed structures. Underground pipelines ruptured. Ground fracturing widespread landslides.
X.	Devastating	Masonry buildings destroyed infrastructure crippled. Massive landslides. Water bodies may be overtopped causing flooding of the surrounding areas and formation of new water bodies.
XI.	Catastrophic	Most buildings and structures collapse. Widespread ground disturbances tsunamis.
XII.	Very catastrophic	All surface and underground structures completely destroyed. Landscape generally changed rivers change paths tsunamis.

Table 2.6: European Macro-seismic Scale EMS

I.	Not felt	Not felt even under the most favorable circumstances.
II.	Scarcely felt	Vibration is felt only by individual people at rest in houses especially on upper floors of buildings.
III.	Weak	The vibration is weak and is felt indoors by a few people. People at rest feel a swaying or light trembling.
IV.	Largely observed	The earthquake is felt indoors by many people outdoors by very few. A few people are awakened. The level of vibration is not frightening. Windows doors and dishes rattle. Hanging objects swing.
V.	Strong	The earthquake is felt indoors by most outdoors by few. Many sleeping people awake. A few run outdoors. Buildings tremble throughout. Hanging objects swing considerably. China and glasses clatter together. The vibration is strong. Top heavy objects topple over. Doors and windows swing open or shut.
VI.	Slightly damaging	Felt by most indoors and by many outdoors. Many people in buildings are frightened and run outdoors. Small objects fall. Slight damage to many ordinary buildings; for example fine cracks in plaster and small pieces of plaster fall.

Table 2.6 Contd.

VII.	Damaging	Most people are frightened and run outdoors. Furniture is shifted and objects fall from shelves in large numbers. Many ordinary buildings suffer moderate damage: small cracks in walls; partial collapse of chimneys.
VIII.	Heavily damaging	Furniture may be overturned. Many ordinary buildings suffer damage: chimneys fall; large cracks appear in walls and a few buildings may partially collapse.
IX.	Destructive	Monuments and columns fall or are twisted. Many ordinary buildings partially collapse and a few collapse completely.
X.	Very destructive	Many ordinary buildings collapse.
XI.	Devastating	Most ordinary buildings collapse.
XII.	Completely devastating	Practically all structures above and below ground are heavily damaged or destroyed.

Table 2.7: Japanese Meteorological Agency JMA

0	Imperceptible to people. Less than 0.008 m/s ²
1	Felt by only some people in the building. 0.008–0.025 m/s ²
2	Felt by many people in the building. Some sleeping people awake. 0.025–0.08 m/s ²
3	Felt by most people in the building. Some people are frightened. 0.08–0.25 m/s ²
4	Many people are frightened. Some people try to escape from danger. Most sleeping people awake. 0.25–0.80 m/s ²
5–	Most people try to escape from danger some finding it difficult to move. 0.80–1.40 m/s ²
5+	In many cases unreinforced concrete-block walls collapse and tombstones overturn. Many automobiles stop due to difficulty in driving. Occasionally poorly installed vending machines fall. 1.40–2.50 m/s ²
6–	In some buildings wall tiles and windowpanes are damaged and fall. 2.50–3.15 m/s ²
6+	In many buildings wall tiles and windowpanes are damaged and fall. Most unreinforced concrete-block walls collapse. 3.15–4.00 m/s ²
7	In most buildings wall tiles and windowpanes are damaged and fall. In some cases reinforced concrete-block walls collapse. Peak ground acceleration greater than 4 m/s ²

Table 2.8 shows the relationship of the different intensity scales. The Rossi-Forel Modified Mercalli JMA and MSK scales were compared.

Table 2.8: Comparison of different intensity scales

Damage %	Rossi-Forel	Modified Mercalli	JMA	MSK
0-	I	I	0	I
		II		II
	II	III	I	III
	III			IV
	IV	IV	II	V
	V		III	
	VI	V	IV	VI
		VI		VII
	VII	VII	V	VIII
	VIII			
	IX	VIII	VI	IX
		IX		X
40- 50- 60- 70- 90-	X	X	VII	XI
		XI		XII
		XII		

The basis for which the EMS was established was the MSK scale which preceded it one of a family of intensity scales which originated with the widely used simple ten-degree scale by Rossi and Forel this was revised by Mercalli and expanded by Cancani subsequently to twelve degrees and then defined in a very elaborate way by Sieberg as the Mercalli-Cancani-Sieberg MCS scale.

The main difference between the EMS-98 and other intensity scales is in the detail with which different terms are used and defined at the outset in some building types damage grades.

2.2.3 Elastic Response Spectra

Since the 1930s sensitive strong motion seismometers which record the actual ground motions that are specific to their location have been available. An example of this is shown in Figure 2.13.

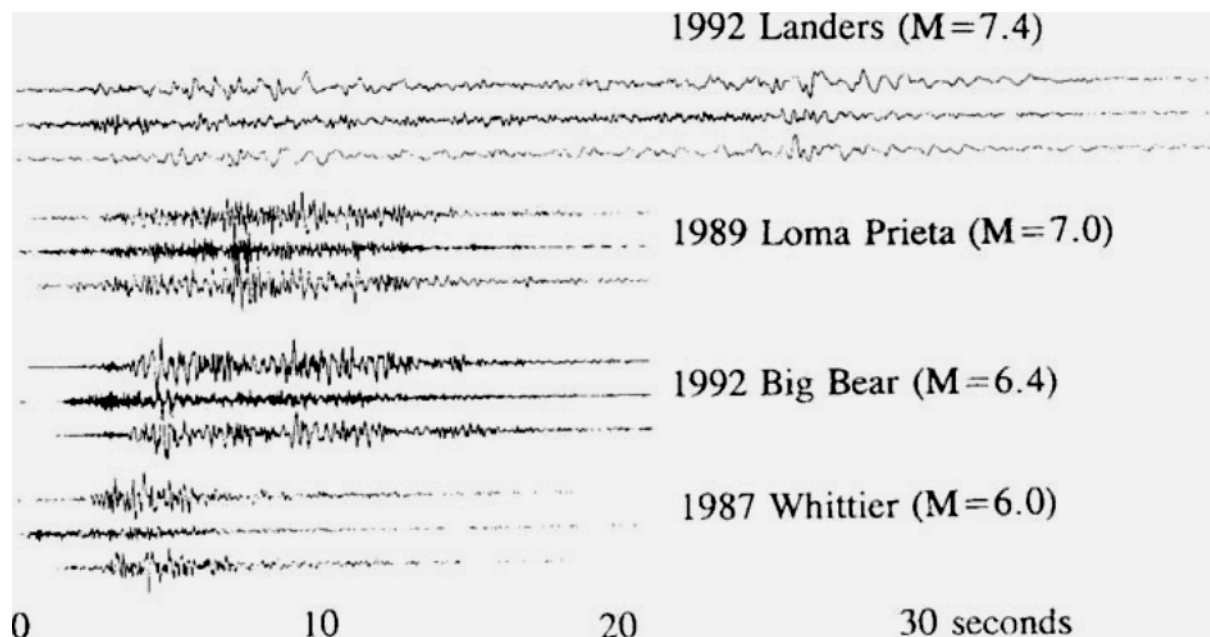


Figure 2.13: Typical earthquake accelerograms Darragh et al. 1994

When a single-degree-of-freedom SDOF mass is subjected to a time history of ground motion that is similar to that shown in Figure 2.13 the mass or elastic structural response can be readily computed as a function of time which generates a structural response time history as shown in Figure 2.14 for several oscillators with differing natural periods. The response time history can be computed by direct integration or by solution of the Duhamel integral Wai-Fah and Charles 2003.

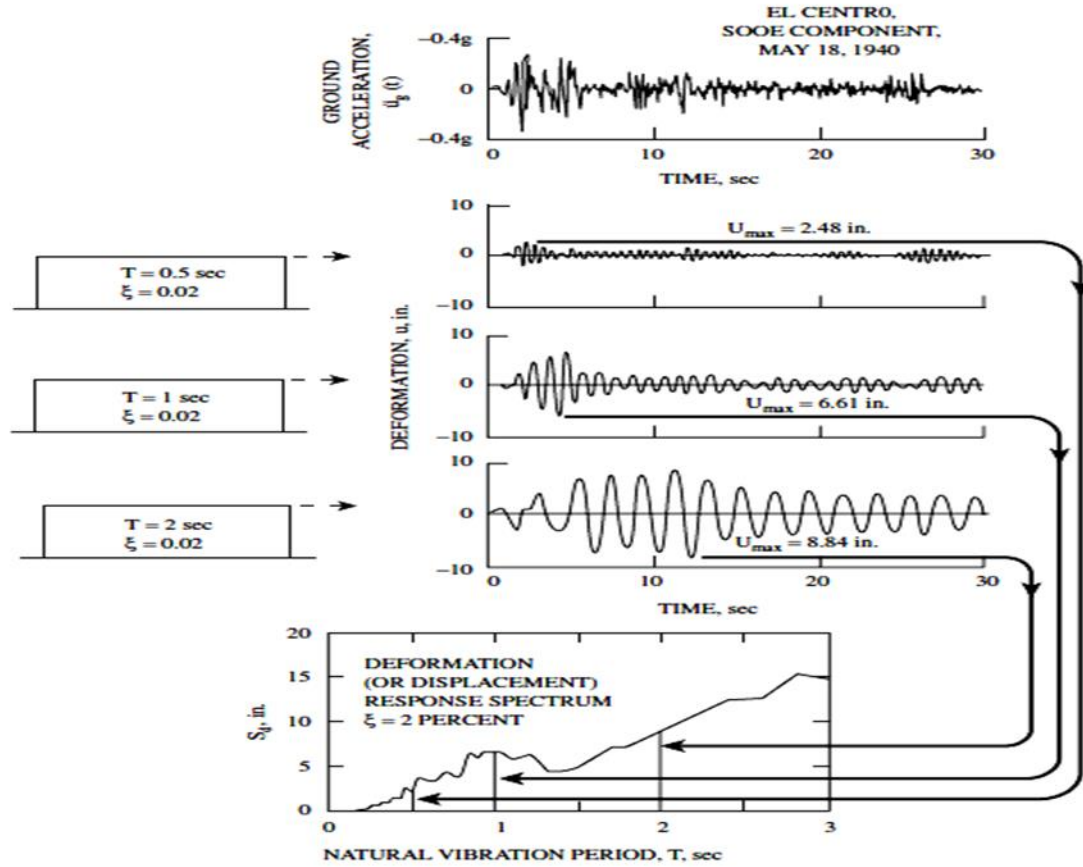


Figure 2.14: Computation of deformation or displacement response spectrum Wai-Fah and Charles 2003

However the elastic response is more typically calculated in the frequency domain. Wai-Fah and Charles 2003

$$v(t) = \frac{1}{2\pi} \int_{-\infty}^{\infty} H(\omega) c(\omega) \exp(i\omega t) d\omega \quad 2.8$$

Where:

$v(t)$ is the elastic structural displacement response time history

ω is frequency

$H(\omega) = \frac{1}{-\omega^2 m + ic + k}$ is the frequency response function

$c(\omega) = \frac{1}{2\pi} \int_{-\infty}^{\infty} p(t) \exp(-i\omega t) dt$ is the Fourier transform of the input motion i.e.

the Fourier transform of the ground motion time history which takes advantage of calculation efficiency using the fast Fourier transform.

For design purposes it is enough to know only the maximum amplitude of the response time history. If the natural period of the Single Degree of Freedom SDOF is varied across a spectrum of engineering interest. Usually for natural periods from .03 to 3 or more seconds then the plot of these maximum amplitudes is termed a response spectrum. Figure 2.15 shows a S_d the displacement response spectrum b S_v the velocity response spectrum and c S_a the acceleration response spectrum.

Note that:

$$S_v = \frac{2\pi}{T} S_d = \omega S_d \quad 2.9$$

and

$$S_a = \frac{2\pi}{T} S_v = \omega S_v = \left(\frac{2\pi}{T}\right)^2 S_d = \omega^2 S_d \quad 2.10$$

Response spectra is the basis for contemporary earthquake engineering structural analysis and design. They are easily computed if the ground motion is known. For the purposes of design however response spectra must be estimated.

Response spectra may be plotted as shown in Figure 2.15 with arithmetic axes and in Figure 2.16 where the velocity response spectrum is plotted on tripartite logarithmic axes which equally enables reading of displacement and acceleration response. Response spectra are most normally presented for 5% of critical damping. Damping is any effect that tends to reduce the amplitude of vibrations.

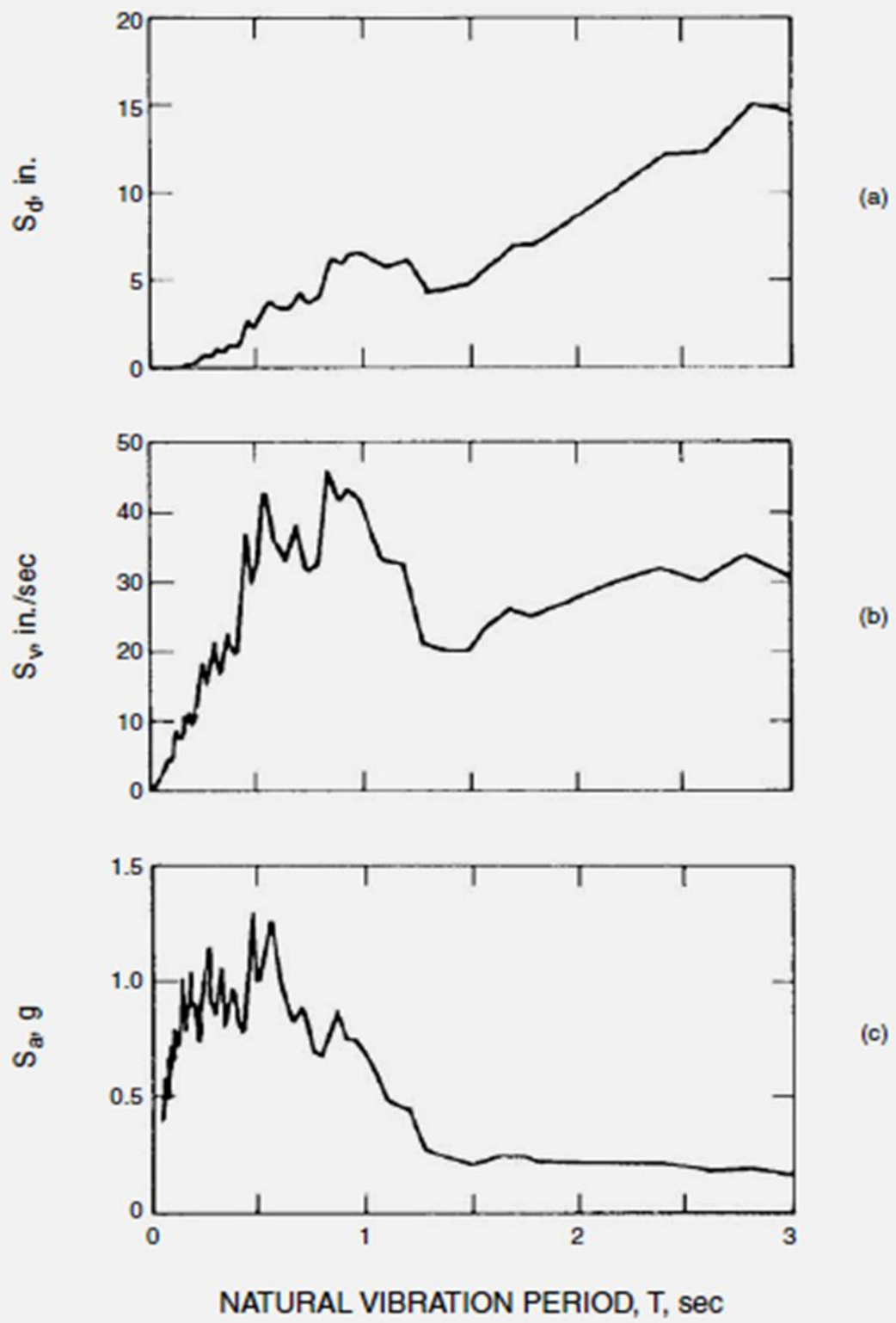


Figure 2.15: Response spectra Chopra 1981

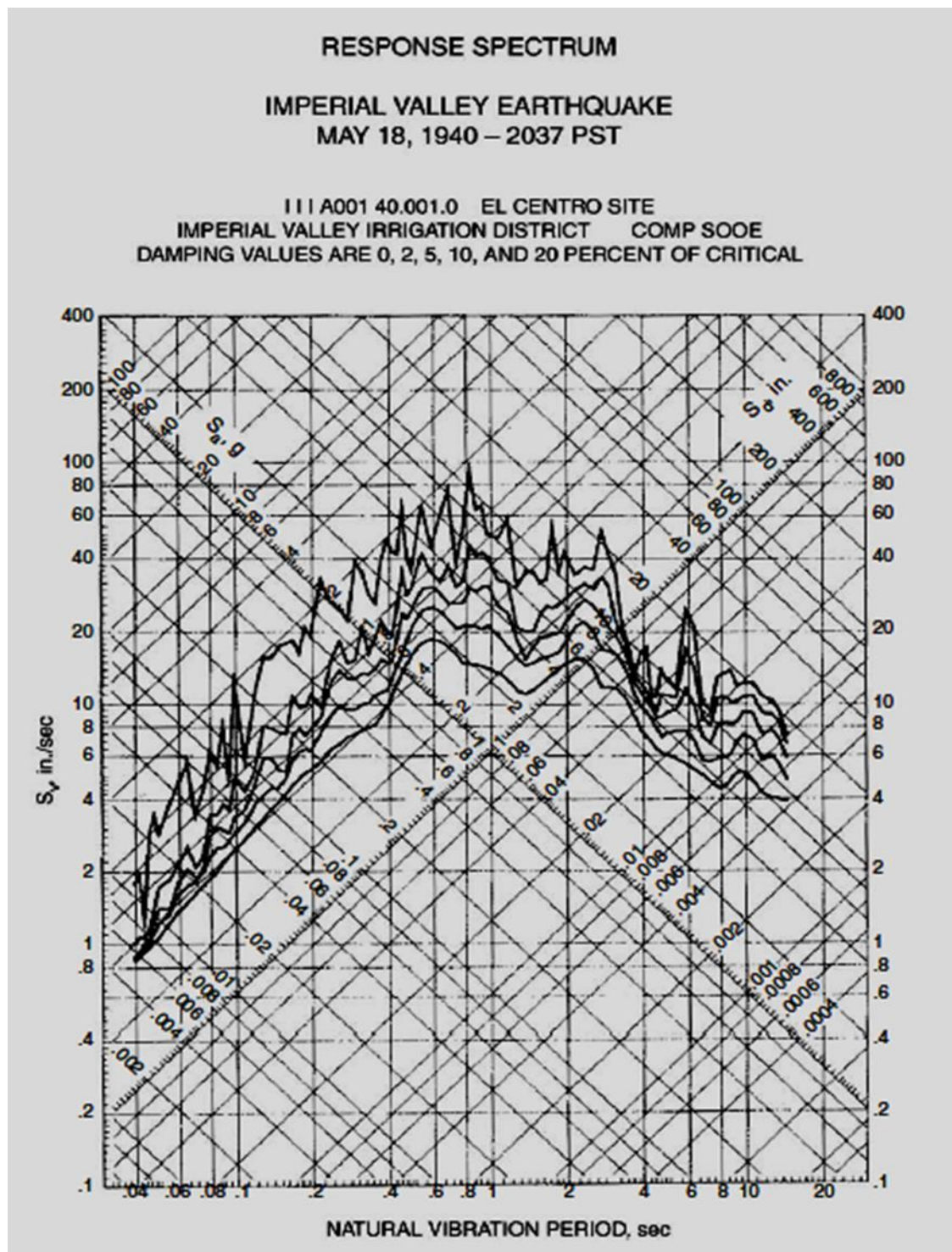


Figure 2.16: Response spectra tri-partite plot Chopra 1981

The shape of Response spectra are irregular and generally have a concave-down arch or trapezoidal shape when plotted on tripartite log paper. It has been observed that they tend to be characterized by three regions: 1 a region of constant acceleration in the high-frequency portion of the spectra; 2 constant displacement at low frequencies; and 3 constant velocity at intermediate frequencies. If a spectrum amplification factor is defined as the ratio of the

spectral parameter to the ground motion parameter. The parameter here indicates acceleration velocity or displacement whereby response spectra can then be estimated from the data in Table 2.9 provided estimates of the ground motion parameters is available. An example spectrum using these data is given in Figure 2.17.

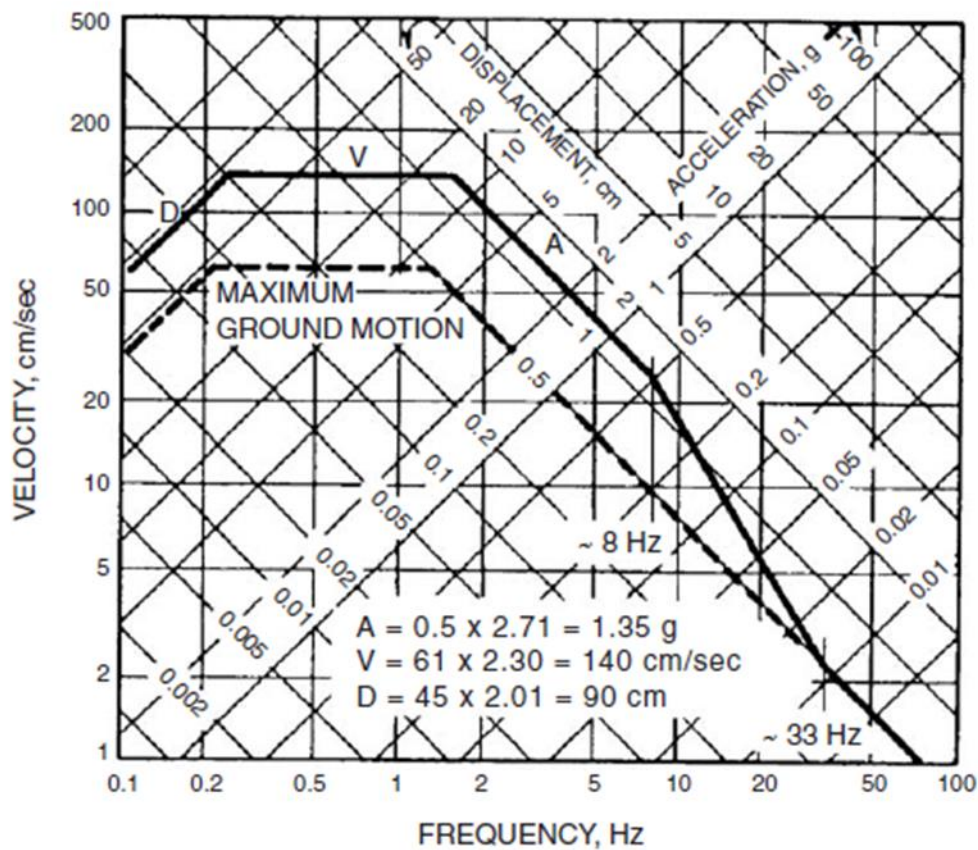


Figure 2.17: Idealized elastic design spectrum horizontal motion $ZPA = 0.5g$ 5% damping one sigma cumulative probability Newmark and Hall 1982.

Table 2.9: Spectrum Amplification Factors for Horizontal Elastic Response

Damping % Critical	One Sigma 84.1%			Median 50%		
	A	V	D	A	V	D
0.5	5.10	3.84	3.04	3.68	2.59	2.01
1	4.38	3.38	2.73	3.21	2.31	1.82
2	3.66	2.92	2.42	2.74	2.03	1.63
3	3.24	2.64	2.24	2.46	1.86	1.52

Table 2.9Contd.

5	2.71	2.30	2.10	2.12	1.65	1.39
7	2.36	2.08	1.85	1.89	1.51	1.29
10	1.99	1.84	1.69	1.64	1.37	1.20
20	1.26	1.37	1.38	1.17	1.08	1.01

Source: Newmark and Hall 1982

Where A = Acceleration V = Velocity and D = Displacement

A standardized response spectrum is provided in the Uniform Building Code UBC 1997. The spectrum is a smoothed average of normalized 5% damped spectra obtained from actual ground motion records grouped by subsurface soil conditions at the location of the recording instrument and is applicable for earthquakes characteristic of those that occur in California SEAOC 1980. This normalized shape may be employed to determine a response spectrum appropriate for the soil conditions. Note that the maximum amplification factor is 2.5 over a period range of approximately 0.15 sec to 0.4 ~ 0.9 sec depending on the soil conditions as shown in Figure 2.18.

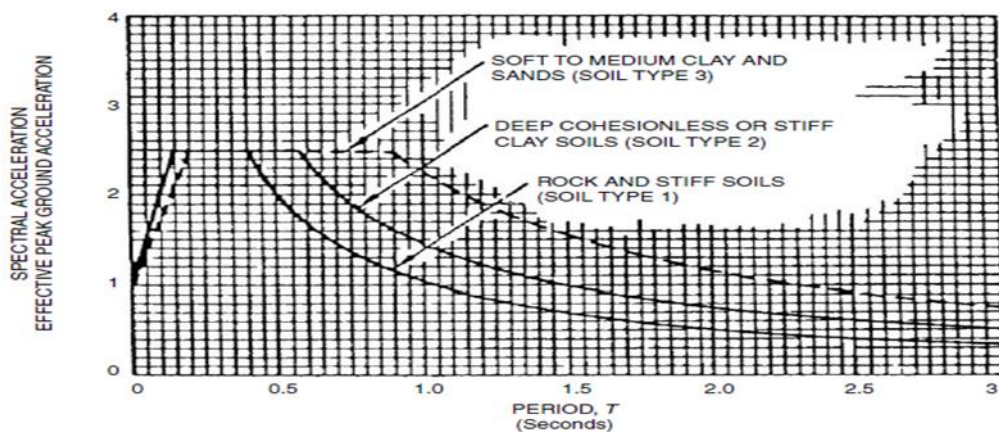


Figure 2.18: Normalized response spectra shapes Wai-Fah and Charles 2003

A Response Spectrum Analysis RSA is a systematic procedure for calculating the statistical maximum response of a structure to a base excitation or an earthquake. Each of the vibration modes considered may be assumed to respond independently as a single degree of freedom system. Design codes i.e. Euro Code 8 EN 1998-1:2005E specify response spectral which determine the base acceleration applied to each mode according to its period the number of seconds required for a cycle of vibration. The design response spectrum is obtained by

multiplying these curves by a structural performance factor a risk factor a zone factor and limit state factor. After determining the response of each vibration mode to the excitation it is necessary to obtain the response of the structure by combining the effects of each vibration mode. The maximum response of each mode does not necessarily occur at the same time instant. Therefore a statistical maximum response is taken as the square root of the sum of squares SRSS of the individual responses.

Kamara and Katale 2013 reported that the Response Spectrum Method RSA was introduced in 1932 by Italian Engineers after the Reggio Calabria earthquake of 1908 which in conjunction with the tsunami claimed nearly 100000 lives.

The RSA is an approach to finding earthquake response of structures using waves or vibrational mode shapes. The Mathematical principles of oscillations in n degree of freedom systems were largely taken into account. The concept of the response spectrum was applied in design requirements in the mid-20th Century for example in building codes in the state of California Trifunac and Todorovska 2008. It came into widespread use as the primary theoretical tool in earthquake engineering in the 1920s when strong motion accelerograph data became widely available Trifunac and Todorovska 2008.

2.2.4 Inelastic Response Spectra

Most Building structures are not expected to be designed or to remain elastic under strong ground motions. Rather structures are expected to enter the inelastic region. The extent to which they behave in elastically can be defined by the ductility factor μ :

$$\mu = \frac{u_m}{u_y} \quad 2.11$$

where:

u_m is the actual displacement of the mass under actual ground motions and

u_y is the displacement at yield i.e. that displacement which defines the extreme of elastic behavior.

Inelastic response spectra can be calculated in the time domain by direct integration similarly to elastic response spectra but with the structural stiffness as a nonlinear function of displacement $k = ku$. If elastoplastic behavior is assumed then elastic response spectra can be readily modified to reflect inelastic behavior Newmark and Hall 1982 on the basis that at low

frequencies <0.3 Hz displacements are the same at high frequencies >33 Hz accelerations are equal and at intermediate frequencies the absorbed energy is preserved. Actual construction of inelastic response spectra on this basis is shown in Figure 2.19 where $DVA A_o$ is the elastic spectrum which is reduced to D' and V' by the ratio of $1/\mu$ for frequencies less than 2 Hz and by the ratio of $1/2 \mu - 1^{1/2}$ between 2 and 8 Hz. Above 33 Hz there is no reduction. The result is the inelastic acceleration spectrum $D'V'A'A_o$ while $A''A_o'$ is the inelastic displacement spectrum. A specific example for $ZPA = 0.16 g$ damping = 5% of critical and $\mu = 3$ is shown in Figure 2.20.

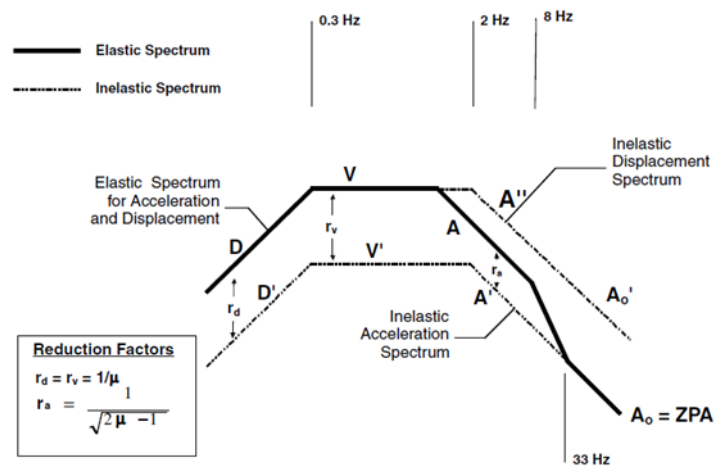


Figure 2.19: Inelastic response spectra for earthquakes Wai-Fah and Charles 2003

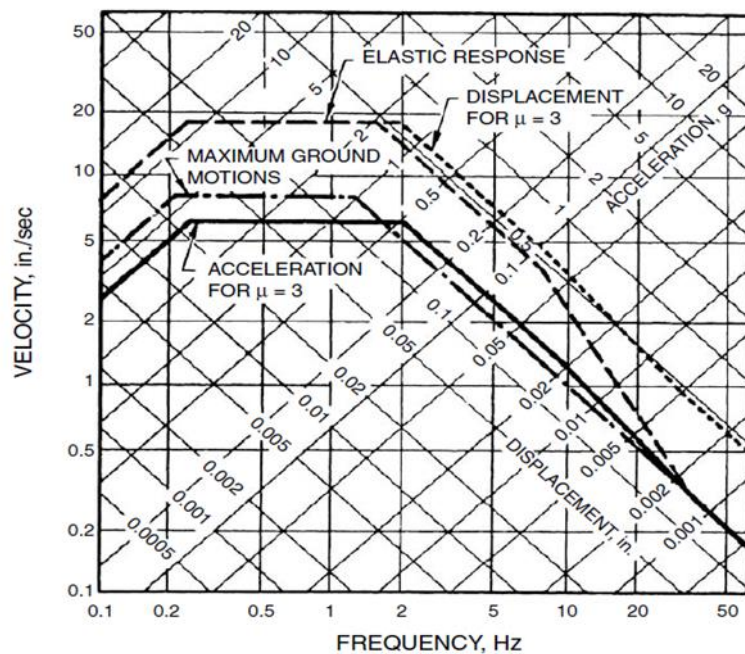


Figure 2.20: Example of inelastic response spectra Wai-Fah and Charles 2003.

2.2.5 Response Spectrum Intensity and Other Measures

Elastic response spectrum cannot directly define damage to a structure which is essentially inelastic deformation but it captures in one curve the amount of elastic deformation for a wide variety of structural periods and therefore may be a good overall measure of ground motion intensity. On this wise responsespectrum intensity is definedas the integral of the elastic response spectrum velocity over the period range 0.1 to 2.5 sec:

$$SI(h) = \int_{T=0.1}^{2.5} S_v(h,T)dT \quad 2.12$$

where:

h = damping.

A number of other measures exist including Fourier amplitude spectrum Clough and Penzien 1975 and Arias intensity Arias 1970:

$$I_A = \frac{\pi}{g} \int_0^{\tau} a^2(t)dt \quad 2.13$$

2.2.6 Time History

Since the 1930s sensitive strong motion seismometers which record the actual ground motions that are specific to their location have been available. Figure 2.21 shows the recorded time history accelerogram of the 1967 Koynanagar earthquake which occurred close to Konya dam near Koynanagar town in Maharastra India. The earthquake has a magnitude of 6.6 with a maximum Mercalli Intensity of VIII.

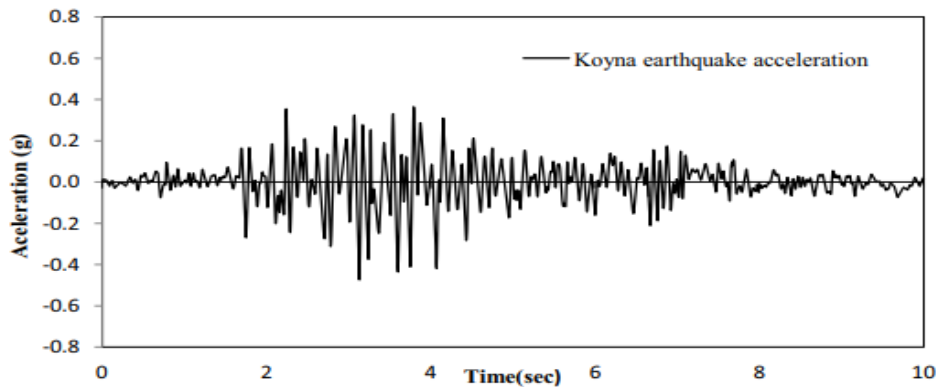


Figure 2.21: Koyna earthquake 1967 acceleration time history data Mandal and Maity 2014

Figure 2.22 shows the recorded time history accelerogram of the Loma Prieta earthquake of 1989 in California. It has a magnitude of 6.9.

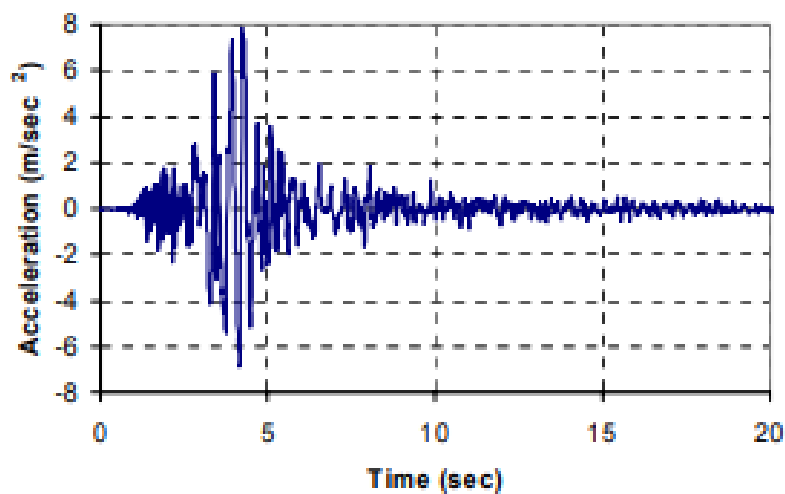


Figure 2.22: Loma Prieta acceleration time history data Nejad et al. 2010

The time history record of the 1994 Northridge earthquake is shown in Figure 2.23. The earthquake has a magnitude of 6.9

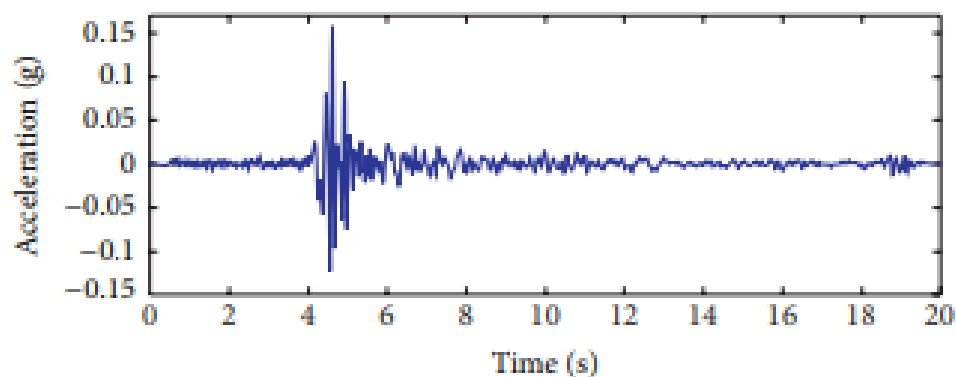


Figure 2.23: Time history of Northridge earthquake at Hollywood Wei and Yuan 2013

The ground motion records which are termed time histories or seismographs have typically recorded accelerations these records are known as accelerograms and are in analogue form on photographic film for many years and more recently digitally. Analogue records required more effort for correction due to instrumental drift before they could be used. Theoretically Time histories contain complete information about the motion at the instrument location recording three traces or orthogonal records two horizontal and one vertical. Time histories that is the earthquake motion on site can differ dramatically in duration frequency content and amplitude. The maximum amplitude of recorded acceleration is called peak ground

acceleration PGA also termed the ZPA or zero period acceleration. Peak ground velocity PGV and peak ground displacement PGD are the maximum respective amplitudes of velocity and displacement. Acceleration is normally recorded with velocity and displacement being determined by integration. However actual velocity and displacement meters are also deployed to a lesser extent. Acceleration can be expressed in units of cm/sec^2 termed gals but is often also expressed in terms of the fraction or percent of the acceleration of gravity 980.66 gals termed 1 *g*. Velocity is expressed in cm/sec.

2.2.7 Engineering Intensity Scale

Engineering Intensity Scale EIS is as a measure of earthquake intensity and has been relatively underutilized but is worth noting as it tries to combine the engineering benefits of response spectra with the simplicity of qualitative intensity scales such as MMI. The EIS is simply a 10 X 9 matrix which characterizes a 5% damped elastic response spectrum Figure 2.15. Nine period bands 0.01–0.1 –0.2 –0.4 –0.6 –1.0 –2.0 –4.0 –7.0 –10.0 sec and ten S_v levels 0.01–0.1 –1.0 –4.0 –10.0 –30.0 –60.0 –100 –300 –1000 kine are defined. As can be seen since the response spectrum for the example ground motion in period band II 0.1–0.2 sec is predominantly in S_v level 5 10–30 kine it is assigned EIS 5 X is assigned where the response spectrum does not cross a period band. In this manner a nine-digit EIS can be assigned to a ground motion in the example it is X5677776X which can be reduced to three digits 576 by averaging or even to one digit 6 for this example. Numerically single-digit EIS values tend to be a unit or so lower than the equivalent MMI intensity value.

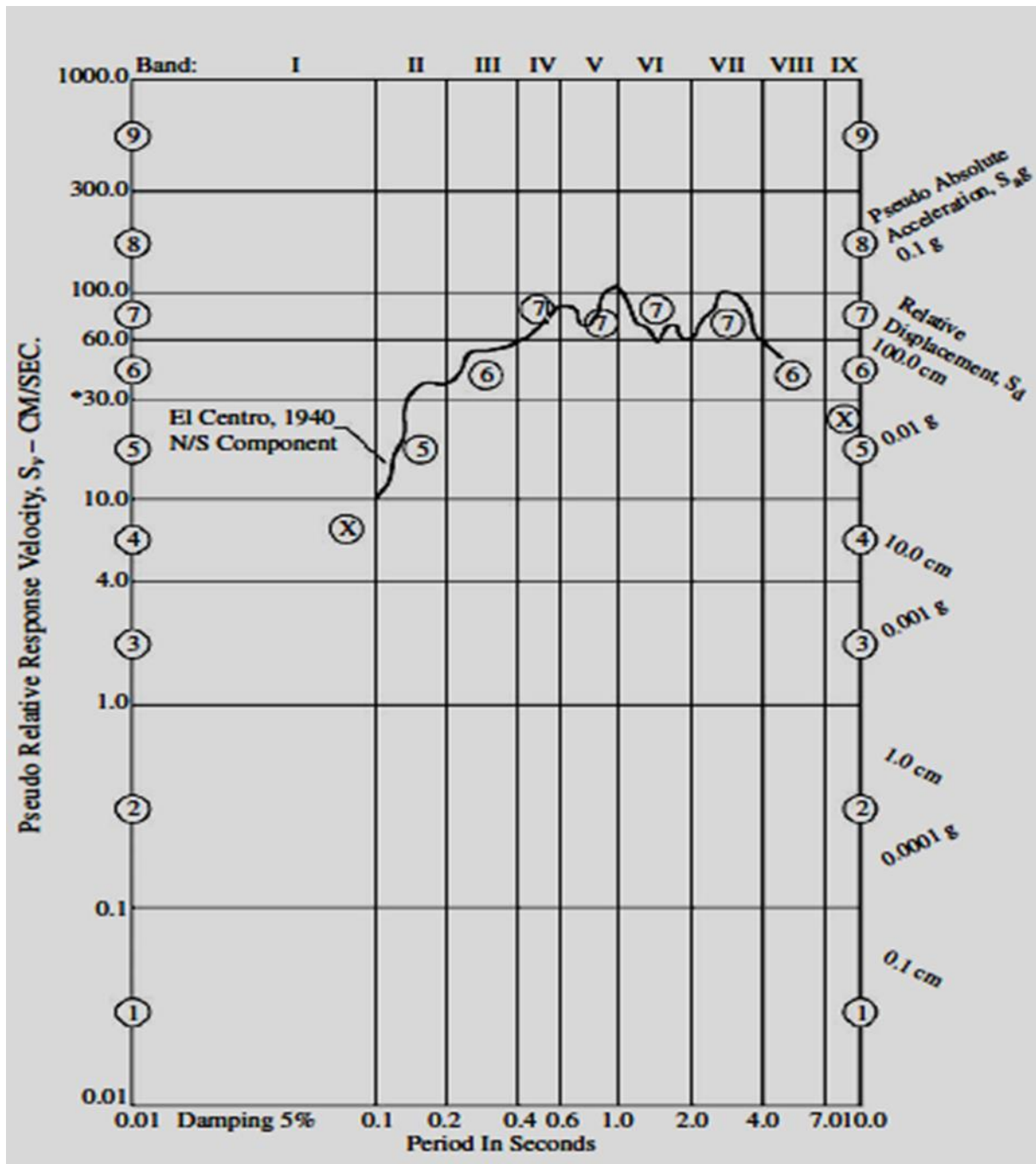


Figure 2.24: Engineering intensity scale matrix with an example Wai-Fah and Charles 2003

2.3 Earthquakes in Africa

Some parts of Africa have been experiencing earthquakes and also tremors.

2.3.1 Earthquakes in Ghana

Ghana is located on the south-eastern margin of the West Africa craton and is far away from the major earthquake zones that mark the present day lithospheric plate boundaries. However a number of major and minor earthquakes have struck the country in past and present as

presented in Table 2.10. According to Amponsah 2003 Ghana recorded damaging earthquakes dating as far back as 1636 of which the major three major occurred in 1862 1906 and 1939 being the last. The most destructive earthquake that struck the then Gold Coast caused a lot of damages and loss of life and property was the 22nd June 1939 earthquake with a magnitude of 6.5 on the Richter scale. Seventeen people were killed and one hundred and thirty three injured Amponsah 2003. Earth tremors of magnitude 1.0 to 4.8 on the Richter scale have been recorded in recent times Paulina 2004.

Among the first recorded earthquake in Ghana happened at about 2:00 p.m. on the 18th of December 1636 in the Axim district in southwestern Ghana near the border between Ghana and Cote d'Ivoire Ivory Coast. Its surface magnitude M_s was 5.7 and the maximum intensity was IX on the MSK scale. Buildings and undergrounds such as the Gold mine at Aboasi northeast of Axim collapsed burying many of the miners Paulina 2004.

A very powerful earthquake hit the capital city of Accra in the year 1862 and caused considerable damage to many important building. The maximum intensity and magnitude M_L of the earthquakes were estimated to be IX and 6.5 respectively. A surface wave magnitude greater than or equal to 6.5 and a maximum intensity of IX uncertain on the MSK scale for this event was obtained. The castle the seat of government and some forts were rendered unserviceable together with all stone buildings. Three people were killed in Accra. The effect of the earthquake's shock was felt along the coast East of Togo and in Benin Republic which are neighbouring countries to Ghana Paulina 2004.

An earthquake which was felt in Accra in 1863 appears to have been an aftershock of the 1862 earthquake but there is no detailed information on the event to confirm this.

Two severe shocks rocked Accra in 1871 and 1872. The magnitudes of these events were 4.6 and 4.9 respectively and the maximum intensities were VI and VII. The 1872 event caused damages to buildings in Accra.

In 1883 there was another minor event which was reported to have been felt in Accra.

According to Paulina 2004 two severe shocks were recorded and felt in Eastern Ghana and Togo at about 9:00 p.m. and 9:20 p.m. on 20 November 1906. The maximum intensity of the shocks was estimated to be VIII at Ho in the Eastern region of Ghana. The magnitude M_L estimation was 6.2; however the estimation of the surface magnitude M_s was 5.0 and the maximum intensity VIII on the MSK scale which they considered uncertain. Many buildings

were greatly affected by this event some cracked and others partly destroyed. No casualties were reported for this event also felt by people in Togo and Benin. During the same month several minor shocks were felt lasting for three weeks.

On 11th February 1907 a mild shock was felt in Accra and Lome. An earthquake whose epicentre was out to sea and accompanied by tidal waves destroyed the wharf at Lome in Togo on 11th May 1911. During the same day at 3:21 p.m. an earthquake was felt in Accra but did not cause any damage.

The most damaging earthquake in Ghana that caused a lot of destruction to buildings and infrastructure and loss of life occurred on 22nd June 1939. The earthquake occurred at about 7:20 p.m. and was felt for about 20 to 30 seconds. The earthquake was assigned a magnitude of 6.5 on the Richter scale and a maximum intensity of IX. The surface wave and body wave magnitudes were estimated to be 6.4 and 6.8 respectively and the maximum intensity as IX. The magnitude *ML* was also estimated to be 6.4. The intensity of the shock was greatest in James town a suburb of Accra. Seventeen people were killed 133 injured and an estimated one million pounds' worth of damage done. The earthquake was extensively felt. People who are at an area of about 750 000 square kilometres and at places more than 800 kilometres from Accra felt the earthquake. The epicentre was out at sea otherwise the felt area would have been more than the estimated figure. The earthquake was recorded tele-seismically at various observatories around the world. The epicentre was located at 5.18° N 0.13° W with a focal depth of 13 kilometres Odeyemi 2013.

The famous geological effect of the 1939 earthquake was a line of fissure in the alluvium bordering the Akwapim scarp between Weiija and Fete. In April 1939 two minor shocks believed to be foreshocks of the 1939 earthquake occurred in Accra. As many as five aftershocks were felt in a day. This trend continued until July 1939.

On 11 March 1964 and 9 February 1969 earth tremors of magnitudes *ML* 4.5 and 4.7 were recorded respectively. Both events were felt in Accra. The 1964 event was located not far from the multi-million-dollar hydroelectric dam in Akosombo.

Tremors which were felt in all the regional capitals occurred on 8 January 1997 14 February 1997 and 6 March 1997. These events caused panic among the people in the affected areas. Many minor tremors were recorded from 1998 to 2002 with magnitudes ranging from 1.0 to 3.0 on the Richter scale. The intensities range from I to IV Paulina 2004.

Another earthquake was recorded at 8am on Wednesday September 26 2007 at 385 km from Takoradi Western Ghana with a magnitude of 5.9 Earthquake Track 2018. The earthquake results in no loss of lives and properties.

The Ghana Geological Survey Authority has successfully installed eight seismic stations across the country to transmit signal timely to the Authority for processing in order to warn the nation ahead of impending disaster Essel 2018.

Table 2.10: Earthquakes and tremors recorded in Ghana

Date	Time	Region	Magnitude	Intensity	Comments
1636-12-18	2.00pm	Axim district Southwestern Ghana	5.7	IX	Buildings and underground gold mines collapsed burying many of the miners
1862	-	Accra	6.5	IX	3 people killed and damages to infrastructures
1863	-	Accra	-	-	Aftershock
1871	-	Accra	4.6	VI	Aftershock
1872	-	Accra	4.9	VII	Damages to buildings and infrastructures
1883	-	Accra	-	-	Aftershock
1906-11-20	9.00pm	Eastern Ghana	6.2	VIII	Damages to buildings
1907-02-11	-	Accra and Lome	-	-	
1911-05-11	-	Accra	-	-	The sea accompanied by tidal waves destroyed the wharf at Lome in Togo
1939-06-22	7.20pm	James town suburb of Accra	6.5	IX	17 people killed 133 injured and damages to buildings worth an estimated value of one million pounds
1964-03-11	-	Accra	4.5	4.5	-
1969-02-06	-	Accra	4.7	4.7	-
1990-02-12	1.34am	The Weija enclave Gbawe	2.7	2.7	-
1990-04-14	11.43am	Weija Gbawe	3.1	3.1	-

Table 2.10 Contd.

Date	Time	Region	Magnitude	Intensity	Comments
2018-03-24	3.09am 3.41am 11.31am	WeiJa Gbawe McCarthy Hill Nyanyano enclave	-	-	-

Source: Paulina 2004; EarthquakeTrack 2018; Essel 2018

2.3.2 Earthquakes in Algeria

Various cases of earthquakes have been recorded in Algeria which has resulted in the loss of lives and damages to properties.

2.3.2.1 Algeria El Asnam earthquake 1980

The El Asnam earthquake of 1980 occurred in the Algerian town of El Asnam now known as Chlef within the central part of the Chelif valley. The earthquake - at 7.3 on the Richter scale was the largest earthquake in Algeria and was followed 3 hours later by an aftershock measuring 6.3 on the Richter scale these two quakes caused considerable destruction of life and properties.

The initial earthquake occurred at 12.24 GMT. Reports initially put the death toll much higher at around 20000. The final death toll however ended up being around 3500. The town's main hospital a big department store a girls' school 2 housing complexes and a Central mosque were destroyed. Around 300000 people were left homeless. The earthquake was the largest in the Atlas range since 1790 Ambraseys 1981.

2.3.2.2 Boumerdès earthquake; Algeria 2003

The northern Algeria is situated at the boundary between the African Plate and the Eurasian Plate thus creating a zone of compression. This zone of compression manifests itself by several thrust and faults. Because of the location between two tectonic plates many earthquakes occurred in the region. The mechanism of the earthquake on 21st of May corresponds to a northeast-striking thrust fault line named Zemmouri fault and was identified for the first time after the earthquake.

Approximately 2266 people were killed and 10261 injured in the earthquake. More than 1243 buildings were completely or partially destroyed and nearly 150000 people became homeless. Infrastructures were damaged in Algiers Boumerdès Réghaïa and Thénia.

The earthquake generated a Tsunami which damaged boats off the coast of the Balearic Islands. East of the capital Algiers was affected most and the Boumerdès Province was the hardest-hit region. Several cities were heavily damaged in the Boumerdès Province including Thénia Zemmouri and Boumerdès which were the worst affected cities. Many buildings of the twentieth century during the colonial rule suffered heavy damage in the Belcourt Bab-El-Qued and El-Casbah areas in Algiers Province. A water treatment plant in Boudouaou also suffered light damage to its clarifiers and clear water storage tanks. This treatment plant provides more than 12% of the treated water to the Boumerdès and Algiers. The water pipeline from the Keddara Dam to the water treatment plant was broken at a concrete junction structure. The main power plant in Cap Djenet suffered minor to moderate damage. A high voltage switch yard located near Réghaïa had heavy damage.

Société Nationale des Transports Ferroviaires Algeria's state-owned railway company suffered track damage near the town of Thénia. Some rail tracks were also blocked by debris of destroyed buildings. Eighteen bridges in the affected region were affected with minor to moderate damage. Cracks developed in some roads and highways. The port of Algiers suffered light to moderate damage due to soil liquefaction and settlement caused by the earthquake. This port at that time handled approximately 40% of the national cargo traffic. Port operations were reportedly reduced by 30% immediately after the quake. The airport control tower and terminal were moderately damaged.

The earthquake had significant effect on local communication. An 8000-switch central office in the El Harrach area of Algiers completely damaged and another 20000-switch office was heavily destroyed. Central offices in Boumerdes Zemmouri and Tidjelabine were not spared. Submarine telecommunication cables also suffered damage. Two underwater fibre optic cables between Algeria and Spain received heavy damage due to undersea landslide caused by the quake Edwards 2004.

Figure 2.11: Earthquakes in Algeria

Date	Region	Coords	Mag.	I	Deaths	Injuries	Comments
2018-01-02	Blida	36.401°N 2.535°E	5.0	-	-	-	-
2016-05-28	Bouïra	36.351°N 3.517°E	5.4	V	-	28	-
2014-08-01	Algiers	36.85°N 3.16°E	5.6	-	6	420	-
2010-05-14	Bouïra	35.9°N 4.12°E	5.3	VII	2	43	Sequence
2008-06-06	Oran	35.88°N 0.66°E	5.5	VI	1	-	Rockfall
2008-01-09	Oran	35.62°N 0.57°E	4.6	-	1	-	Several buildings damaged
2006-03-20	Béjaïa	36.62°N 5.33°E	5.2	-	4	9	-
2003-05-27	Boumerdès	36.94°N 3.58°E	5.8	-	9	200	Tsunami
2003-05-21	Boumerdès	36.91°N 3.71°E	6.8	X	2266	10261	Tsunami
1999-12-22	Aïn Témouchent	35.2°N 1.35°W	5.6	VII	22–24	175	-
1994-08-18	Mascara	35.52°N 0.11°E	5.9	-	159	289	-
1989-10-29	-	36.75°N 2.44°E	5.6	-	-	-	Additional casualties and damage
1989-10-29	Tipaza	36.77°N 2.39°E	5.9	VIII	22–35	184–700	-
1987-01-26	Mohammadia	35.96°N 1.37°E	4.9	-	1	7	629 homes damaged
1985-10-27	Constantine–Skikda	36.46°N 6.76°E	5.8	-	6	-	-
1980-10-10	Chlef	36.2°N 1.37°E	7.1	X	2633–5000	8369–9000	Extreme damage / tsunami
1975-07-11	-	36.32°N 5.24°E	4.3	-	1	18	Moderate damage
1968-02-25	-	36.08°N 5.07°W	4.9	VIII	1	4	Limited damage
1967-07-13	-	35.5°N 0.1°W	5.0	-	10	15	Limited damage
1965-01-01	M'Sila	35.7°N 4.4°E	5.5	-	4	350	Moderate damage
1960-02-21		36°N 4.1°E	5.6	-	47	88	Moderate damage

Table 2.11 Contd.

1959-11-07	Boumedfaâ	36.38°N 2.55°E	4.9	VIII		2	300 million Francs
1950-09-09	Chlef	36.27°N 1.59°E	6.7	XI	1243– 1409	5000	Severe damage / tsunami
1947-08-06	Annaba	37°N 8°E	5.3	IX	3	3	Moderate damage
1946-02-12	Hodna Mountains	35.75°N 5°E	5.6	IX	264	-	Severe damage
1910-06-24		36°N 4°E	6.4	XI	12	-	Limited damage
1891-01-15	Tipaza	36.5°N 1.8°E	-	X	36	-	Moderate damage / tsunami
1887-11-29	Kalaa	35.58°N 0.33°E	-	X	20	-	Severe damage
1885-12-03	M'Sila	36.1°N 4.6°E	5.9	IX	33	-	Severe damage
1869-11-16	Biskra	34.9°N 5.9°E	-	X	40	-	Moderate damage
1867-01-02	El Affroun	36.4°N 2.7°E	-	X	70	100	Moderate damage
1856-08-22		37.1°N 5.7°E	-	IX	3	-	Tsunami
1825-03-02	Blida	36.2°N 2.8°E	-	X	7000	-	Extreme damage
1790-10-09	Oran	35.7°N 0.6°W	-	X	3000	-	Severe damage / tsunami
May 1716	Algiers	36.8°N 3°E		-	20000	-	Extreme damage

Source: Edwards 2004; Karuri 2016; NOAA 2018.

2.3.3 Rwanda Earthquake

The Rwanda earthquake took place in 2008 at the Western Rift of the East African rift system. The East African rift system is diffused about 3000-km-long zone of lithospheric extension or crustal extension passes that through Djibouti from eastern Africa and Eritrea on the north to Malawi on the south and that constitutes the boundary between the Africa plate on the west with Somalia plate on the east. At the earthquake's latitude the Africa and Somalia plates are spreading apart at a rate of about four millimetres per year. The earthquake occurred near Lake Kivu the basin of which was created by normal faulting. The largest earthquake to have occurred in the rift system since 1900 had a magnitude of about 7.6. The epicentre of the February 3 2008 earthquake is within several tens of kilometres of the epicentre of a magnitude 6.2 earthquake that killed two people in Goma in October 2002.

Earthquakes within the East African rift system happened due to both normal faulting and strike-slip faulting. The earthquake was felt in Burundi resulting to power supply cut and as far as the Kenyan capital Nairobi. When two earthquakes struck Rwanda and neighbouring Democratic Republic of Congo on Feb 3 2008 21 people were recorded killed and 200 seriously injured. The two earthquakes struck close together in Africa's great lakes region along the same western Great Rift Valley fault line. The first quake with a magnitude of 6.0 and its epicentre in Democratic Republic of Congo happened at 10.30am followed by another 5.0 quake in southern Rwanda. Earthquakes are common in the western Great Rift Valley—a seismically active fault line straddling western Uganda eastern Democratic Republic of Congo Rwanda and neighbouring Tanzania.

Another Earthquake occurred in 2005. The Lake Tanganyika earthquake occurred along the East African Great Rift Valley and struck at 12:19:55 UTC 14:19:55 local time at the epicentre on 5 December 2005 with its focus approximately 10 km 6 miles below the surface of Lake Tanganyika. Its estimated magnitude was between 6.3 and 6.8.

Though earthquakes with 6-7 magnitudes in range are not usually associated with massive destruction in East Africa such seismic occurrence can wreak havoc. The earthquake was centred roughly below Lake Tanganyika in addition to the Democratic Republic of Congo where the most widespread damage has been reported it was felt in Burundi Tanzania Rwanda and as far away as the coastal city of Mombasa in Kenya. The earth tremor was felt as far as Luanda in Angola where it was felt for around 20 seconds enough to send people running in panic in search of a shelter. No damage was reported there Cooks 2008.

2.3.4 Earthquake in Southern Africa

Southern Africa region a portion of the African plate is located between Latitude 35°S and 15°S and Longitude 10°E and 50°E. The map of the major tectonic plate indicates that the Southern Africa region is not traversed by any major tectonic plate boundary. Southern Africa is located in the interior of large Africa plate. Mozambique is the area with active seismicity in Southern Africa Zimbabwe and Northern Botswana. These activities are known to an extension of the East Africa rift system.

The Plate's borders to the south are situated in the mid-Atlantic and mid-India Ocean ridges. The Southern Africa region is situated in an intra-plate area and it is known to be a rather

complex seismic characteristics. Most serious earthquakes are from tectonics origin and the epicentre of over 90% of global natural earthquakes take place at boundary of major plates.

The mining activities have also contributed immensely to the occurrence seismic activities it has been established that 40 or more tremors are recorded monthly. The largest mine related earthquake in South Africa history is a 5.3 magnitude earthquake which occurred on 5th March 2005 in the Klerksdorp Stilfontein Gold mining district 200km west of Johannesburg below the ground. Substantial damage was observed within the mines while above ground the structural damage to property was relatively low.

In South Africa south-western Cape has one of the highest levels of seismicity which is characterized by its dual source of seismicity from mine related events and tectonic origin earthquakes.

The most severe earthquake in history of South Africa is magnitude 6.3 which occurred on 29th September 1969 in Ceres 100 km northeast of Cape Town. The event resulted in loss of lives and numerous buildings damaged in the town of Tulbagh. An earthquake also occurred on 4th September 1809 with an estimated magnitude 6.3 at the Milnerton Fault a mere 10km from Cape Town CBD and the location of the Cape Town Stadium Alabi et al. 2013.

Figure 2.25 shows a map of Southern Africa showing the seismicity of the region.

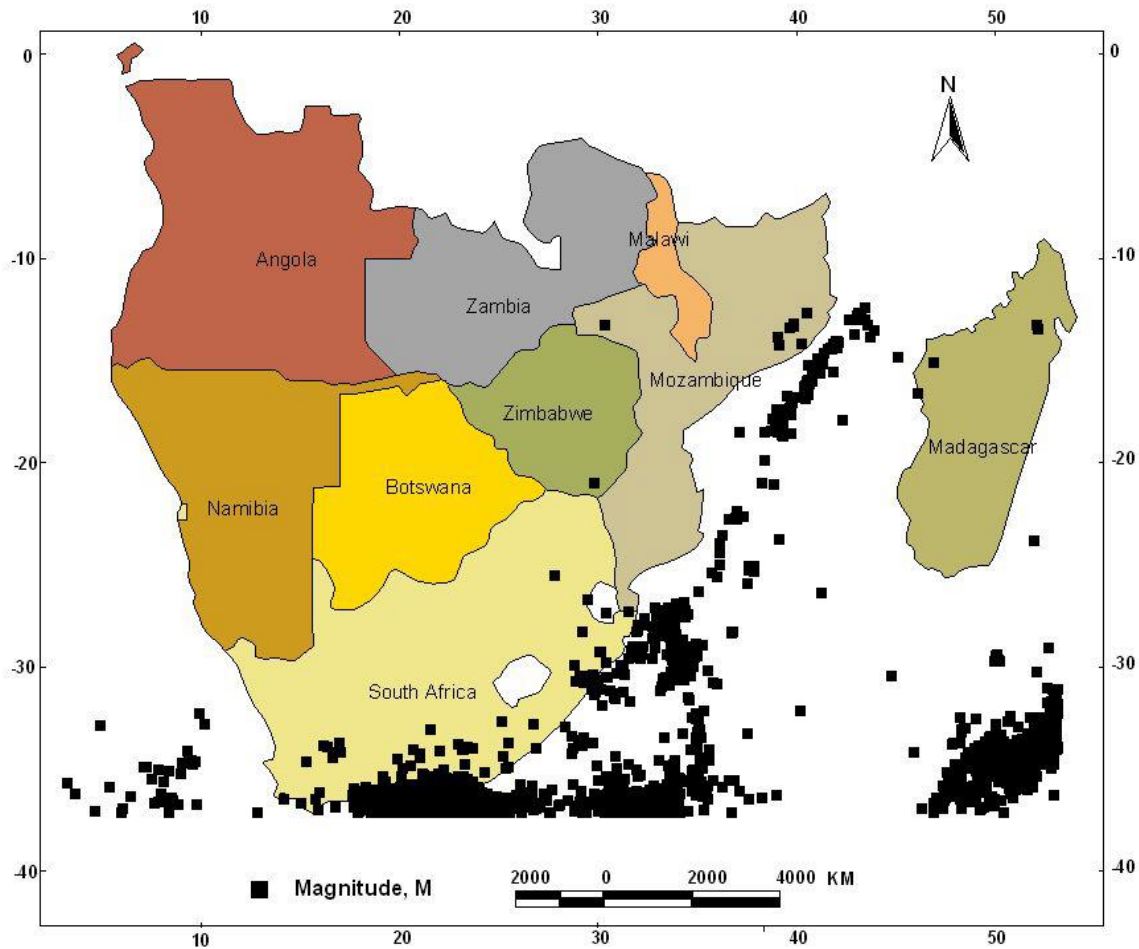


Figure 2.25: Map of the Southern Africa showing seismicity distributions $M \geq 3$ between 1986-2009 Alabi et al. 2013

2.3.5 Earthquakes and Tremors in Nigeria

Nigeria lies on the eastern flank of the Atlantic Ocean and since Jurassic times the Atlantic Ocean margins have been opening consistently. Unlike the Pacific Ocean margins which are characterized by subduction tectonics and occurrence of devastating earthquakes the Atlantic margins are generally thought to be quiet and as a result there was little consciousness and preparedness for earthquake occurrences and mitigation in Nigeria and west Africa.

Some seismic events have occurred within the last 70 years especially in the south-western part of Nigeria but no destruction and loss of lives were reported and/or documented; they were only felt as vibrations. This is an indication that micro-earthquakes might actually be occurring but the lack of seismic data occasioned by the non-availability of seismological stations had made the determination of the locations and magnitudes of these events difficult Afegbua et al. 2011. In 2002 the Federal Government of Nigeria established the Centre for

Geodesy and Geodynamics in Toro and saddled it with the responsibility of monitoring seismic activities in the country. The Nigeria National Network of Seismological Stations NNNSS established by the National Agency for Science and Engineering Infrastructure NASENIAbujahas been managed bythe Centre for Geodesy and Geodynamics Since 2006 Akpan and Yakubu 2010

The Nigerian National Network of Seismological Stations is currently made up of seven stations which are headed by the centre in Bauchi. Other stations are in Minna Niger Sate; Kujama Kaduna State; Ile-Ife Osun State; Nsukka Enugu State; Awka Enugu State; and Abakaliki Ebonyi State. These stations are equipped with 24-bit 4-channel data acquisition system and broadband seismometers form the seismicity instrumental network of Nigeria. There are efforts improve the available stations and also to increase the number of these stations Afegbua et al. 2017. Table 2.12shows the location of both the existing and proposed seismic stations in Nigeria.

Table 2.12: Locations of current and proposed seismic stations in Nigeria

N/S	Station code	Name	Geological Foundation	Instruments installed	Coordinates
1	Oyo	Oyo	Granite	SP-400 Seismometer DR4000	07° 53'131 ¹¹ N 03°57'078 ¹¹ E
2	IBN	Ibadan	Gneiss	No Instrument Installed	07°27'251 ¹¹ N 03°53'520 ¹¹ E
3	IFE	Ile – Ife	Gneiss	EP-105 Broadband Seismometers DR4000 Recorder	07°32'800 ¹¹ N 04° 32'815 ¹¹ E.
4	AWK	Awka	Shale and Silt stone	EP-105 Broadband Seismometers DR4000 Recorder	06° 14'561 ¹¹ N 07° 06' 693 ¹¹ E
5	NSU	Nsukka	Sandstone	EP-105 Broadband Seismometer DR4000 Recorder	06° 52'011 ¹¹ N 07° 25'045 ¹¹ E
6	ABK	Abakaliki	Sandstone	EP-105 Broadband Seismometer DR4000 Recorder	06°23'453 ¹¹ N 08° 01'474 ¹¹ E
7	ABJ	Abuja	Granite	No Instrument Installed	08°59'126 ¹¹ N 07° 23'380 ¹¹ E
8	TOR	Toro Central	Gneiss	EP-105 Broadband Seismometers DR4000 Recorder	10°26'303 ¹¹ N 09° 07'089 ¹¹ E
9	KAD	Kaduna	Granite	EP-105 Broadband Seismometers DR4000 Recorder	10° 26'101 ¹¹ N 07° 38'484 ¹¹ E
10	MINN	Minna	Granite Gneiss	EP-105 Broadband Seismometer DR4000 Recorder	09° 30'702 ¹¹ N 06° 26'411 ¹¹ E

Source: Tsalha et al. 2015

In Nigeria's geological history there is an age long conviction that the nation is safe in terms of earthquake occurrence. Nigeria's geological framework is widely believed to be located within the mobile belt of Africa i.e. between the West Africa Craton and the Congo Craton (Figure 2.26). The Pan-African orogeny that occurred 600 ± 100 Ma was the last major deformation and metamorphism witnessed within the belt with slight effect on the adjacent Craton (Turner 1971). This could be related to the reason the country is not experiencing major earthquakes.

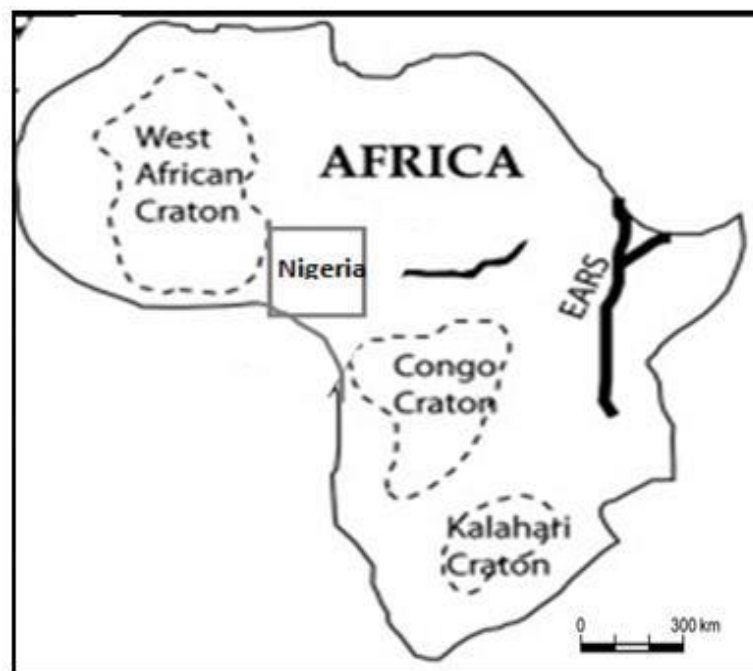


Figure 2.26: Location of Nigeria in square with respect to the Craton (Tsalha et al. 2015)

In the past and recently earthquakes or earth tremors have been recorded to take place in Nigeria. This brings to question the age long belief that Nigeria is safe seismically. (Tsalha et al. 2015).

Two theories were postulated as the origin of seismicity in Nigeria the possible fault systems were assumed based on the spatial distribution of the Earth tremors. The Yola- Dambata Akka-Jushi and Warri – Ijebu Remo systems (Afegbua et al. 2011). Most of these fault systems are inclined along Northwest – Southeast. Figure 2.27 shows the areas where some earth tremors were felt in Nigeria.



Figure 2.27: Map of Nigeria showing the areas where some earth tremors were felt Akpan and Yakubu 2010 [diameter of the solid dot denotes intensity of the events not drawn to scale]

The second postulation was the earlier theory that shows that tremors occurred in the inland extension of the north east through to the south west originating from the Atlantic Ocean and that possibly causes the activities along the Ojebu-Ode and Ibadan axis which is inferred to be associated with the Ifewara- Zungeru fracture systems as shown in Figure 2.28 Adepelumi et al. 2008.

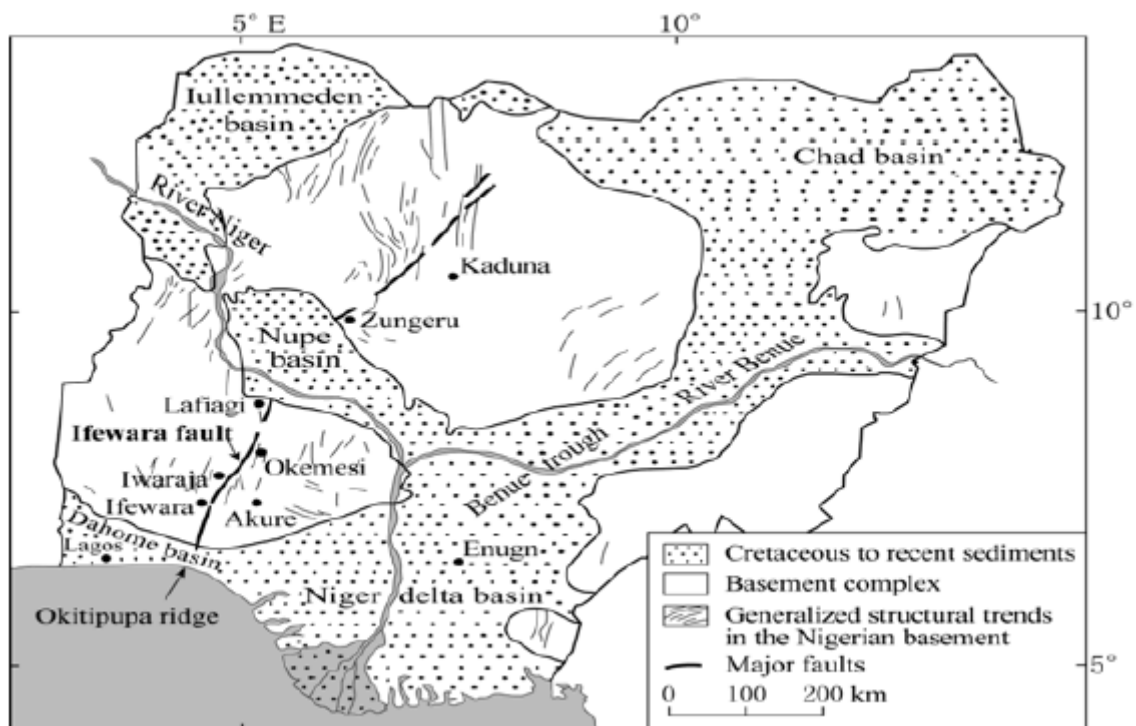


Figure 2.28: Map of Nigeria showing the Zungeru-Ifewara fault Odeyemi 2006

Although Nigeria is not located within the major seismic zones of the world; over the years so many minor earthquakes have been recorded in some parts of the Nigeria. The first widely reported cases of an Earth tremor in Nigeria was in 1933. Other events were reported in 1939 1964 1984 1990 1994 1997 2000 and 2006 as shown in Table 2.13 Akpan and Yakubu 2010.

The intensities of these earthquakes ranged from III to VI based on the modified Mercalli Intensity Scale. Among these occurrences only the 1984 1990 1994 and 2000 occurrences were instrumentally recorded. They had magnitudes ranging from 4.3 to 4.5 Akpan and Yakubu 2010.

Table 2.13: List of historical/Instrumental earthquakes and tremors felt in Nigeria

S/N	Year-Month-Day	Origin Time	Felt Areas	Magnitude	Intensity	Probable Epicentre
1	1933	-	Warri	-	-	-
2	1939-06-22	19:19:26	Lagos Ibadan Ile-Ife	6.5	-	Akwapin fault in Ghana
3	1948-07-28	-	Ibadan	-	-	Close to Ibadan
4	1961-07-2	15:42	Ohafia	-	-	Close Ohafia area

Table 2.13 Contd.

S/N	Year-Month-Day	Origin Time	Felt Areas	Magnitude	Intensity	Probable Epicentre
5	1963-12-21	18:30	Ijebu-Ode	-	V	Close to Ijebu-Ode
6	1981-04 -23	12:00	Kundunu	-	III	At Kundunu village
7	1982-10-16	-	Jalingo Gembu	-	III	Close to Cameroun Volcanic Line
8	1984-07-28	12:10	Ijebu-Ode Ibadan Shagamu Abeokuta	-	VI	Close to Ijebu-Ode
9	1984-07-12	-	Ijebu Remo	-	IV	Close to Ijebu - Ode
10	1984-08-02	10:20	Ijebu-Ode Ibadan Shagamu Abeokuta	-	V	Close to Ijebu-Ode
11	1984-12-08	-	Yola	-	III	Close to Cameroun Volcanic Line
12	1985-06-18	21:00	KombaniYaya	-	IV	KombaniYaya
13	1986- 07-15	10 :45	Obi	-	III	Close to Obi town
14	1987-01-27	-	Gembu	-	V	Close to Cameroun Volcanic Line
15	1987 - 03-19	-	Akko	-	IV	Close to Akko
16	1987-05-24	-	Kurba	-	III	Close to Kurba village
17	1988-05-14	12:17	Lagos	-	V	Close to Lagos
18	1990-06-27	-	Ibadan	3.7		Close to Ijebu-Ode
19	1990-04-5	-	Jerre	-	V	Close to Jerre Village
20	1994-11-07	05:07:51	Ojebu-Ode	4.2		Dan Gulbi
21	1997	-	Okitipupa		IV	Close to Okitipupa Ridge
22	2000-18-15	-	Jushi-Kwari	-	III	Close to JushiKwari village
23	2000-03 -13	-	Benin	-	IV	Benin City 55Km from Benin

Table 2.13 Contd.

S/N	Year-Month-Day	Origin Time	Felt Areas	Magnitude	Intensity	Probable Epicentre
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24	2000-03-07	15:53:54	Ibadan Akure Abeokuta Ijebu- Ode Oyo	4.7	-	Close to Okitipupa
25	2000-05-07	11:00	Akure	-	IV	Close to Okitipupa Ridge
26	2001-05-19	-	Lagos	-	IV	Close to Lagos city
27	2002-08-08	-	Lagos	-	IV	Lagos city
28	2005-03	-	Yola	-	III	Close to Cameroun Volcanic Line
29	2006-03-25	11:20	Lupma	-	III	Close to Ifewara- Zungeru Fault
30	2009-09-11	-	Abomey-Calavi	-	II	Close to Benin
31	2011-11-05	-	Abeokuta	4.4	-	
32	2016-07-10		Saki	2.5	-	Oyo State
33	2016-08-10		Igbogene	2.5	-	Bayelsa
34	2016-09-11		Kwoi	-	III	Jaba Kaduna State
35	2016-09-12		SambangDagi	-	III	Jaba Kaduna State
36	2018-09-07		Mpape	-	II	Abuja

Source: Akpan and Yakubu 2010; Abolarin and Adedeji 2016

2.4 Strawbales

Straw is one of the best renewable building materials available and is found around the world in abundance. Straw is the stalk of tall grain plants such as guinea corn maize wheat hemp rye or rice that remain in the field after the grains are harvested. Its chemical composition is primarily cellulose just like trees. When straws are bundled together into a bale they become a solid block that is highly resistant to decomposition called strawbale. When assembled together and rendered with cement mortar strawbale make a beautiful strong energy efficient and ecologically sound house.

Million tonnes of straws are produced globally from farms inform of agricultural wastes. This is because strawbale walls have been employed in the construction industry as walls made from plastered strawbale blocks have low thermal conductivity and also provide insulation for houses when employed in building construction Adedeji 2008.

With the continuous and increasing concern in regards to global warming and continuous depletion of the earth's natural resources a new focus on environmental sustainability has

arisen throughout the world. According to the Food and Agriculture Organization of the United Nations FAO report of November 17 2005 Nigeria has the world's highest deforestation rate of primary forest. Between 2000 and 2005 the country lost 55.7 percent of its primary forest – defined as forest with no visible signs of past or present human activities. Logging subsistence agriculture and the collection of fuel wood are cited as leading causes of forest clearing in the West African countries FAO 2005. Modern materials used for construction like concrete and steel have greater embodied energy than other alternatives. The amount of energy which is required to produce to produce a particular product is referred to as embodied energy.

2.4.1 Historical Background on Strawbale

Using strawbale as a construction material is not a new idea and according to Amazon-Nails 2001 straws have been used in various forms for building purposes thousands of years ago. In the late 1800s buildings were first constructed using strawbales in the USA when baling machines were invented. The white settlers on Nebraska were growing grain crops in an environment which lack stone or timber for them to build their house and while they were waiting for timber to arrive by train-wagon the spring that follows they built “temporary” houses out of what was to them a waste agricultural material – the baled up straw stalks of grain crops. They built their house directly with the bales as if they were giant building blocks where the bales themselves formed the load bearing structure. This was referred to as the Nebraskan or load bearing style. The Nebraska settlers discovered that these bale houses kept them warm throughout the very cold winter and yet cool during the hot summer with the additional sound-proofing benefits of protection from the howling winds. The positive experience of the Nebraska settlers in building and living in strawbale homes led to the building of permanent houses some of which are still occupied dwellings today. This early building construction method continued until about 1940 when a combination of war and the rise in the popularity and in the use of concrete for construction led to its virtual extinction. Then in the late 1970s Judy Knox and Matts Myrhman among other pioneers of the strawbale revival rediscovered some of those early houses and set about refining the construction method and passing on this knowledge to environmental enthusiasts. Through the green and permaculture movements the ideas spread very rapidly with new buildings springing using the Nebraska/load bearing style. In recent times new techniques were developed to improve the construction method Amazon-Nails 2001.

Lerner *et al.* 2003 reported that things however began to change in the 1970's after long years of strong focus on wood steel stone and concrete within the construction industry as people became aware of the fragility of the environment they began looking for ways of minimizing their personal impact on the earth. This positive thinking about our environment led to the re-discovery of strawbale construction as a viable and environmentally friendly sustainable construction practice. Industry pioneers began building structures and documenting their endeavours. Strawbale construction continued to gain momentum through the 1980's with the beginning of the new millennium seeing this re-emerging technology spreading throughout the world. The strawbale construction continues to improve and extends to other areas today with over 80 buildings currently listed on the Ontario Strawbale Building Coalition's OSBBS's website. The benefits of strawbales are continuously being discovered by Engineers around the world Harvest-Homes 2003.

2.4.2 Advantages of Strawbale Construction

There are a number of purported advantages associated with strawbale construction as reported by House-Energy 2010 and U.S. DOE 1995; however it is the positive environmental impact of strawbale construction that really drives this re-emerging industry. Below are some of the reported advantages of strawbale construction:

- Strawbale walls can be built quickly and easily. This and environmental concerns have fuelled some popular enthusiasm for strawbale home construction as it is reported that strawbales can save around 20% of the wood necessary for the construction of conventional residences.
- Strawbale provides super insulation the R-values defined as the measure of insulation capacity of material of walls constructed with strawbales are double or triple of traditional homes making them excellent for both cold and hot climates.
- Strawbales are particularly cheap since the material is usually considered an agricultural waste and the wall construction does not require specific skill- it is a Do It Yourself DIY job.
- Strawbale modular nature allows a faster wall construction and a flexible design: deep windows deep overhang curves etc.
- Straw is annually renewable has low embodied energy and reduces the reliance on lumber saving valuable forests.

2.4.3 The Fears of Using Straw as a Construction Material

2.4.3.1 Fire Resistance

There are a number of people who are still skeptical regarding the use of straw as a construction material. Some of their concerns are valid while others have been proven to be debatable Chris et al. 2005.

One of the most common concerns is regarding fire performance. Loose straw is a very combustible material but strawbale makes it difficult for oxygen to reach and fuel a potential fire while the plaster coating on the walls provides an insulating barrier keeping exterior flames away from the bales thus elongating the fire resistant period. Evidence of this behaviour has been found in a number of laboratory fire tests. Tests carried out at University of Ilorin showed that plastered strawbale surface withstood fire for 2 hours as reported by Adedeji 2007 and successfully met the conditions of acceptance as outlined in ASTM American Society for Testing Materials Method E 119-05a Fire Tests of Building Construction and Materials for a fire endurance rating of 2 hours.

2.4.3.2 Moisture

Another major concern is with moisture management within strawbale walls. The potential for the straw to rot from within and the related issues are well documented with anecdotal and research information. There have been limited studies in this regard. However with the proper use of cement and lime to prepare the plaster render material penetration of moisture which may lead to rotting of the straws can be prevented Amazon-Nails 2001.

Amazon-Nails 2001 also reported that cement and lime are materials that behave very differently from each other and are used for different reasons. Whereas lime is a breathable material that holds water within itself whilst it is raining and releases it once the rain stops cement is waterproof and works by preventing water from penetrating through it to the surface beneath. Also lime is quite flexible whereas cement is rigid. This means that as long as there are no cracks in the cement it will stop water from reaching the straw. However due to its rigidity it is almost impossible for it not to have cracks in it after a short period of time especially when it is applied to a flexible backing material like straw. This means that when it rains the rain passes through tiny cracks and then filters down the inside face of the cement and stagnate at the bottom of the wall where it is trapped and cannot sip out. The trapped moisture at the base of the wall build-up and causes the straw to rot.

Other primary causes of moisture concerns for strawbale buildings are due to plumbing leakage groundwater and roof leaks. All these and the above can be tackled through proper design foundation design building orientation proper material selection etc detailing and maintenance AmazonNails 2001.

2.4.3.3 Termite/Pest Infestation

Little critters love the open spaces provided by wood frame buildings. Straw is not a good source of food that can attract mice or insects and they would first have to gnaw their way through about an inch of plaster to access the straw. It is ideal for areas at high risk of termites as straw does not suit their palette AmazonNails 2001.

2.4.3.4 Structural Strength

Harvest-Homes 2003 stated that unlike wood from construction that has a low tolerance to extreme lateral and lifts force such as tornadoes or earthquakes strawbale building's continuous monolithic plaster provides extraordinary strength and elasticity in the face of such force. They can actually flex and bend but resist collapse making strawbale building ideal for quake hurricane or tornado-prone areas.

2.4.4 Construction Practices and Materials

The need to construct buildings with low-cost material has become the focus of research in building construction materials in recent times especially the use of materials considered to be wastes.

Strawbale construction is a method of building that incorporates the use of strawbales as structural elements. It has been found to be suitable for living houses offices and other purposes.

In other words or simply put straw is the stalk of grasses like wheat rice sorghum that are high in tensile strength. Straw is not hay used for feeding livestock and includes the grain head. Straw is composed of cellulose hemicellulose liquidand silica. It is non-rigid it is flexible and easy to work with. Bales are compressed masses of straw usually in rectangular blocks and bound with steel wire or propylene twine.

The typical strawbale is bound with two or three strings of polypropylene twine and sometimes wire or fibre twine is used and it is very easy for one man or woman to handle.Bales as made by farmers have are generally good for construction. The biggest

challenge for bale holders is the method used in harvesting the straw conventional method combines the leave long straw fibres which is good for building and rotary method combines the chopped off straw into shortfibre which makes for unstable crumbly bales. With the increased industrialization of farms other types of materials for the strawbales are beginning to predominate in many areas. U.S. Department of Energy 2005 gives types as follows:

- i. Jumbo bale: Rectangular blocks bound with six to ten string of a typical size like 1.2m x 0.9m x 2.4m which can only be handled by mechanized equipment.
- ii. A typical medium-sized three-wire bale may be 0.6m x 0.4m x 1.1m.
- iii. The smaller two-wire bales which are easier to handle are roughly 0.46m x 0.36m x 0.95m.

There are two primary ways of building with strawbales.

- (a) Load bearing or Nebraska style.
- (b) Post and beam / infill / non- load bearing method

In load-bearing strawbale construction bales are stacked and reinforced to provide structural walls that carry the roof load. While in post and beam method a wood metal or masonry structural frame supports the roof and bales are stacked to provide non- structural insulating walls. With either alternative the bale walls are plastered or stuccoed on both the interior and exterior of the wall. This study is based on the former.

The load-bearing strawbale construction employs relatively simple techniques that can be used by any builder and still maintain sufficient flexibility to allow for the design features such as curved walls. Moreover strawbale possess excellent insulate properties such as thermal sound and fire resistance.

It should be added that the technology of strawbale construction is still rapidly evolving. It is highly rated for 'buildability' because it can be very straightforward. And it is important to keep bales during storage and construction and to try and eliminate vermin. During construction strawbale should be kept under tarpaulins or plastic sheets otherwise moisture may get into unprotected walls. Although it may not be dried if bales do get wet slightly they can often be dried out sufficiently to be usable.

Harvest-Homes 2004 reported also that strawbale walls are very resilient and in the event of damage they can be repaired. Wet bales can be taken out and replaced.

2.4.5 Choice of Building Material for Wall Construction

The choice of building materials is determined largely by the properties of the materials such as strength in tension or compression flexural strength thermal insulation capacity which are pertinent to what the builder has in mind. The materials commonly used for wall construction are natural or artificial stones clay bricks sandcrete strawbale concrete timbers plastic glass metal and asbestos cement. Harvest-Homes 2003 posited that before the selection of a material for wall an understanding of the composition advantages and disadvantages of the material is required.

Harvest-Homes 2003 highlighted some factors that affect the choice of materials for wall constructions;

- i. the particular function of the wall which will limit the choice of material by the designer;
- ii. the structural design of the building determines whether a wall is to be load bearing or non-load bearing hence the nature of the loading and the load carrying capacity of the material will restrict the choice of certain materials;
- iii. some special selection of materials may be available from only one supplier which may lead to a long waiting period before an order can be fulfilled. Hence some of the more common materials which are readily available may be used;
- iv. the speed of construction of the wall may lead to the use of materials that would meet the need of that time;
- v. the cost of material may lead to the choice of less costly but suitable material for use; and
- vi. also the cost of labour for the construction of the wall and any subsequent finishing costs.

2.4.6 Function of Walls

As stated by Harvest-Homes 2003 the importance of the functionality of walls will depend upon the type of structure which is to be erected the situation of the building in terms of both macro and micro-environment and the use for which the building is intended. The functions of the walls as highlighted by Harvest-Homes 2003 are:

- i. **Strength and stability:** The wall should be strong enough to carry imposed load without excessive deformation. The strength of a wall depends on the strength of the material of the wall and its wall thickness.
- ii. **Weather exclusion:** It is extremely important to keep the weather out of a building if the internal environment is to remain constant.
- iii. **Thermal insulation:** Heat flows from a region higher to that of a lower temperature. Heat loss from external walls can be reduced by means of insulation; there is a wide difference in the range of thermal conductivity of engineering materials.
- iv. **Durability:** The material from which external walls are constructed must have sufficient resistance to the damaging effects of the climate in the form of erosion atmosphere pollution rain chemical and solar radiation.
- v. **Sound Insulation:** Noise has been defined as unwanted extremely loud sound which can be detrimental to health irritating and can cause loss of concentration. Thus sound transmits less in a medium which is airtight. Walls do not permit the direct passage of successive air pressure through their material.

2.5 Cold-Formed Steel

The use of cold-formed steel as a construction material began from the 1850s but it was not widely used until the first dissipation of American association of iron in 1946 Canada Mortgage and Housing Corporation 2002. According to Azizian 2015 in recent time due to its high quality of construction speed of construction and highest degree of resistance against earthquake movements it is being used in developed countries in buildings. The construction of in-situ concrete cast of beams and columns requires the fixing and removal of formworks and shoring which constitute a considerable part of construction costs. However the cost of construction can be significantly reduced if the conventional reinforcement steel bars are replaced by a thin-walled cold-formed steel channel section as formworks to carry the wet concrete during their construction. The thin-walled cold-formed steel channels also function as reinforcement in composite reinforced concrete beams and columns. Nguyen 1988 posited that replacing standard reinforcement with thin-walled cold-formed steel sections of equal cross-sectional maintains the strength of the beam.

2.6 Review of Past Studies

Azizian2015 in his study of constructional system of cold formed light steel framing LSF presented some major advantages of cold-formed steel. The most important advantages are the lack of need to using thermal operations lack of thermal tensions of residual in sections the possibility of creating sections in various shapes to achieve the maximum resistive return in section lightweight high resistance and rigidity high accuracy in implementing details and rapid and easy installation. The research findings also revealed that the amount of base cut in LSF system compared to reinforced concrete system is reduced up to 55.5% and compared to metal system is reduced to 38.1%. The highest amount of displacement of the roof mass centre in LSF system compared to reinforced concrete system is decreased to 56.2% and compared to metal system is decreased to 38.1%. The total weight of building in LSF system compared to reinforced concrete system is decreased to 572.8% and compared to metal system is decreased to 58.7%. The amount of building skeleton weight in LSF system compared to reinforced concrete system is reduced up to 72.4% and compared to metal system is reduced to 55.8%. The amount of consumed concrete in LSF system compared to reinforced concrete system is reduced up to 78.9% and compared to metal system is reduced to 56%. The amount of consumed steel and reinforcement in LSF system compared to reinforced concrete system is reduced up to 24.5% and compared to metal system is reduced to 54.3%.

Iskhakov and Ribakov2008 in their study dynamic testing of full-scale 11-storey RC building and its 3-storey structural parts: Comparing the results and seismic resistance estimation shows that if the dynamic parameters of both structures are rather close it is possible to obtain the seismic loads acting on the real building using the impulse forces applied to the structural part. This procedure could be used successfully for estimation of seismic resistance of real buildings designed for a given seismic zone.

Tehrani and Maralek 2006 compared nonlinear static pushover and nonlinear dynamic procedures to determine the maximum displacements of an existing steel structure retrofitted with different methods.

Touqan 2008 in his study a comparison of the Response spectrum analysis and Equivalent Static Lateral Load with the elaborate Response Spectrum Method of analysis as they apply to a repertoire of different structural models.

Khan's 2010 performed a response spectrum analysis of 20 story building and compare static and dynamic analysis and design results of buildings up to 40storeys.

Sahoo 2008 observed that the main cause of failure of multi-storey reinforced concrete frames during seismic motion is the column sway mechanism. The seismic inertia forces generated at the floor levels are transferred through the various beams and columns to the ground. His conclusion was that the failure of a column can affect the stability of the whole building but the failure of a beam causes localized effect. Therefore it is better to make beams to be the ductile weak links than columns.

Bhandarkar et al. 2017 performed a seismic analysis and design of a multi-storey building using ETABS. In the Study displacement shear storey driftstorey stiffness modal period and frequency on different floors of an 8 storey reinforced concrete building were analysed and compared to a shear wall structure of the same storey height. The following were discovered.

- The displacement is decreased in shear wall structure as compared to frame structure
- The story stiffness is more in shear structure than the frame structure.
- The story drift is decreased in shear wall structure than the frame structure.
- The modal period and frequency is less in frame structure & more in shear wall structure.
- The story acceleration is more in shear structure than the frame structure.
- The shear structure is suitable in earthquake prone area due to its higher stiffness & less displacement

Kumar et al. 2014 considered both static and dynamic method in the seismic analysis of a multi-storey residential building. In the analysis the response reduction was considered for two cases both ordinary moment resisting frame OMRF and special moment resisting frame SMRF. Equivalent static analysis and response spectrum analysis were the two methods used for the G+15 storey structure by using STAADPRO software. The structure is located in Zone II according to the Indian standard. The response reduction of cases ordinary moment resisting frame and special moment resisting frame values with deflection diagrams in static and dynamic analysis were observed and it was discovered that the special moment of

resisting frame structured is good in resisting the seismic loads. It was concluded that the results of static analysis in OMRF & SMRF values are low when comparing to that of dynamic analysis in OMRF & SMRF values. Hence the performance of dynamic analysis SMRF structure is quiet good in resisting the earthquake forces compared to that of the static analysis OMRF & SMRF.

Ismaeil et al. 2017 analysed and design a reinforced concrete building located in Haql city which is the most active seismic zone region in Saudi Arabia to mitigate seismic loads. A multi-storey building was seismically analysed using the equivalent lateral force procedure with the aid of SAP2000 software. The chosen structure which was ordinary moment resisting frame OMRF was analysed and designed using SBC 301 Saudi Building Code. The result showed that the current design of RC buildings located in the most active seismic zone region in Saudi Arabia Haql city was found unsafe inadequate and unsatisfied to mitigate seismic loads.

Abolarin et al. 2017 investigated the response of three modelled one storey buildings under various earthquake loads. Building A consists of a plastered strawbale walled building building B is made of reinforced concrete shear wall while building C is a reinforced concrete framed building. The three buildings which have the same plan and configuration were subjected to both response spectrum and time history analysis. It was discovered that the plastered strawbale wall building offers an appreciable amount of resistance to earthquake loads and so is a shear wall building. The reinforced concrete framed building is greatly disturbed by an occurrence of earthquake and should not be used in an area of high seismic activities. The plastered strawbale wall despite its light weight can resist earthquake loads and should be employed in low-rise building construction of earthquake resistance structures in place of the reinforced concrete shear wall.

Barnaure and Stoic 2015 in their paper analysed the structural behaviour of masonry in-filled reinforced concrete frame system subjected to in-plane loadings. The result shows that masonry in-fill panels highly increase the stiffness of the structure.

Kara et al 2016 study a multi-mode pushover analysis procedure to estimate the seismic demands of medium rise unsymmetric-plan buildings. In their study a multi-mode adaptive spectra based pushover procedure was presented in order to estimate the structural performance of 3D medium-rise unsymmetric-plan buildings. The result shows that under

ground motion the structural behaviour of the building is linearly elastic and the storey displacements are very small.

Deshmukh and Pajgrade 2015 investigated the effect of infill material on seismic performance of RC Building. The behaviour of RC Frames with both autoclaved aerated concrete AAC and conventional clay brick infill with varying percentage of opening subjected to seismic loads was investigated results shows that deflection and drift of structure with AAC block in all cases was less as compared with the corresponding case of structure with brick masonry.

In view of the available literatures no study had been carried to assess cold formed steel reinforced concrete frame with strawbale infill wall panel under seismic loading.

Tehrani and Maralek 2006 compared the nonlinear static pushover and nonlinear dynamic procedures in the determination of maximum displacements of an existing steel structure retrofitted with different methods.

Patil and Kumbhar 2013 performed a nonlinear dynamic analysis of a 10 storey reinforced concrete building considering different seismic intensities. The building was modelled using SAP2000 and five different time histories were used. The result of the study showed similar variation patterns in seismic responses such as base shear and storey displacement with intensities V and X.

Costa et al. 2005 demonstrated that the use of plasticity theory and the assumption of the rigid-plastic behaviour for the plastic hinges might be of great practical value to assess in a simplified manner the dynamical response of a structure subjected to strong earthquakes.

Chhindam and Autade 2018 carried out seismic analysis of a six storey shear building by subjecting it to a base ground motion excitation of the Bhuj earthquake. Modal analysis was performed using transformation of coordinates so that the MDOF system is analysed like SDOF system. Modal analysis by transformation of coordinates was performed for the MDOF system to investigate the vibration shape vectors and modal time periods. The time history analysis performed using the algorithm based on theoretical time stepping method called Newmark's Linear Acceleration Method was compared with the linear modal time history analysis performed using ETABS software. Results shows that Time history of top displacement record obtained from Newmark's method is similar to that of ETABS response time history.

Bhandarkar et al. 2017 performed a seismic analysis and design of a multi-storey building using ETABS. In the research storey drift displacement shear storey stiffness modal period and frequency on different floors of an 8 storey reinforced concrete building were analysed and compared to a shear wall structure of the same storey height. The following were discovered.

- The displacement is decreased in shear wall structure as compared to frame structure
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- The story acceleration is more in shear structure than the frame structure.
- The shear structure is suitable in earthquake prone area due to its higher stiffness & less displacement

Abolarin et al. 2017 investigated the response of three modelled one storey buildings under various earthquake loads. Building A consists of a plastered strawbale walled building building B is made of reinforced concrete shear wall while building C is a reinforced concrete framed building. The three buildings which have the same plan and configuration were subjected to both response spectrum and time history analysis of El Centro and Petrolia earthquakes. It was discovered that the plastered strawbale wall building offers an appreciable amount of resistance to earthquake loads and so is a shear wall building. The reinforced concrete framed building is greatly disturbed by an occurrence of earthquake and should not be used in an area of high seismic activities. The plastered strawbale wall despite its light weight can resist earthquake loads and should be employed in low-rise building construction of earthquake resistance structures in place of the reinforced concrete shear wall.

In view of the available literatures no study had been carried on time history analysis and numerical design of cold-formed steel frame building with strawbale infill panel under seismic loading.

CHAPTER THREE

3.0 METHODOLOGY

This chapter highlights the steps involved in the modeling and analysis of a four 4 storey cold-formed steel frame building structure with strawbale infill panel under seismic loading using finite element software called CSI ETABS. This study concerns the development of a

finite element model a dissimilar model of a four 4 storey cold-formed steel frame building structure with strawbale infill panel under seismic loadings using the Haiti earthquake data which occurred in 2010 for response spectrum and 5 different historical earthquake records of seismic ground excitation loads time history. Time history analysis which is a non-linear dynamic numerical model analysis was used for this study because it can determine the actual performance of a structure during earthquake occurrence it is based on the direct numerical integration of the differential equations of motion by considering the elasto-plastic deformation of the structural elements Patil and Kumbbhar2013.

3.1 Assessment of Specimen Building

The criteria used in selecting the specimen building - the CSC frame with strawbale infill panels - includes identifying the capabilities of the combination CFS and strawbale materials to resist seismic loading activities as shown by Parastesh and Hosseinjani 2012 and Ashour 2003 respectively. The geometry material of construction location of building soil conditions location of building with respect to seismicity and ductility parameters were considered for the modeling of the building. The applied loading on the building were in accordance to Shelton 2004. In this study materials properties of CSC frame and the storey height of 4-storey and inter storey height of 3.0m for all floors were used in the modeling and analysis.

4-storey building was used because they are classified as medium or low rise i.e. building heights below 35m as defined by Emporis GmbH 2016 and study have shown that light frame cold formed steel is an effective solution for low and medium rise building structures Nakata et al. 2012.

Building details:

Nos. of stories = G+4

C/C dist. in x-direction = 22125mm

C/C dist. in y-direction =9675mm

C/C dist. in z-direction =15000mm total height

3.2 Numerical Modelling of the Building

Response spectrum data of the Haiti earthquake that occurred in 2010 with a magnitude of 7.0 was used for the analysis and were obtained from literature and earthquake agencies.

Haiti earthquake was used for the analysis because it has one of the highest casualties in recent time. The materials used for modelling the building were sourced from past literatures and their properties. The software used for modelling and analysis is Computers and Structures Incorporation CSI ETABS a building module of CSI SAP2000. The general procedure for modelling a building structure in CSI ETABS is shown in Appendix A.

Finite element method FEM was used in this research by assuming a linear elastic behaviour of the building structure. Four types of forces will be involved at any point/node 'i' of a structure. These forces include; the externally applied load $p_i t$ and also the forces resulting from the motion that is inertia f_{Ii} damping f_{Di} and elastic f_{Si} . Thus for each of multiple degrees of freedom the dynamic equilibrium may be expressed as

$$\begin{aligned} f_{I1} + f_{D1} + f_{S1} &= p_1 t \\ f_{I2} + f_{D2} + f_{S2} &= p_2 t \\ f_{I3} + f_{D3} + f_{S3} &= p_3 t \end{aligned} \quad 3.1$$

The force vectors can be represented in matrix form

$$f_I + f_D + f_S = p t \quad 3.2$$

Each of the resisting forces can be expressed most conveniently by means of an appropriate set of influence coefficients. Consider for example the elastic force component developed at any point 1; this depends on the displacement components developed at all points of the structure in general

$$f_{S1} = k_{11}v_1 + k_{12}v_2 + k_{13}v_3 + \dots + k_{1N}v_N \quad 3.3$$

Similarly the elastic force that corresponds to the degree of freedom v_2 is

$$f_{S2} = k_{21}v_1 + k_{22}v_2 + k_{23}v_3 + \dots + k_{2N}v_N \quad 3.4$$

Also in general

$$f_{Si} = k_{i1}v_1 + k_{i2}v_2 + k_{i3}v_3 + \dots + k_{iN}v_N \quad 3.5$$

The coefficients ' k_{ij} ' are called *stiffness influence coefficients*

k_{ij} = a force corresponding to coordinate i due to a unit displacement of coordinate j .

The complete set of elastic force may be written in matrix form as

$$\begin{Bmatrix} f_{S1} \\ f_{S2} \\ \vdots \\ f_{Si} \\ \vdots \end{Bmatrix} = \begin{bmatrix} k_{11} & k_{12} & k_{13} & \cdots & k_{1i} & \cdots & k_{1N} \\ k_{21} & k_{22} & k_{23} & \cdots & k_{2i} & \cdots & k_{2N} \\ \vdots & \vdots & \vdots & \ddots & \vdots & \ddots & \vdots \\ k_{i1} & k_{i2} & k_{i3} & \cdots & k_{ii} & \cdots & k_{iN} \\ \vdots & \vdots & \vdots & \ddots & \vdots & \ddots & \vdots \end{bmatrix} \begin{Bmatrix} v_1 \\ v_2 \\ \vdots \\ v_i \\ \vdots \end{Bmatrix} \quad (3.6)$$

Figure 3.1 shows the link element internal forces and moments acting at a joint.

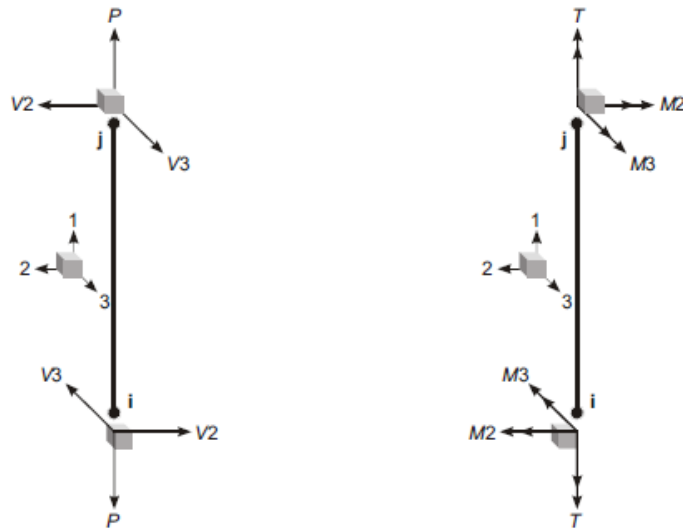


Figure 3.1: Link/Support Element Internal Forces and Moments Acting at a Joint

The Link/Support element internal forces P $V2$ $V3$ and the internal moments T $M2$ $M3$ have the same meaning as for the Frame element.

- Axial: $P = f_{u1}$
- Shear in the 1-2 plane: $V2 = f_{u2} M3s = d - dj2 f_{u2}$
- Shear in the 1-3 plane: $V3 = f_{u3} M2s = d - dj3 f_{u3}$
- Torsion: $T = f_{r1}$
- Pure bending in the 1- 3 plane: $M2_b = f_{r2}$
- Pure bending in the 1- 3 plane: $M3_b = f_{r3}$

Where d is the distance from joint j . The total bending moment resultants $M2$ and $M3$ composed of shear and pure bending parts:

$$M2 = M2s + M2b$$

$$M3 = M3s + M3b$$

The element internal forces and displacement at joint ij for one joint element is expressed as

$$\begin{Bmatrix} P \\ V2 \\ V3 \\ T \\ M2 \\ M3 \end{Bmatrix}_j = \begin{bmatrix} k_{u1} & 0 & 0 & 0 & 0 & 0 \\ & k_{u2} & 0 & 0 & 0 & -dj2 k_{u2} \\ & & k_{u3} & 0 & -dj3 k_{u3} & 0 \\ & & & k_{r1} & 0 & 0 \\ sym. & & & & k_{r2} + dj3^2 k_{u3} & 0 \\ & & & & & k_{r3} + dj2^2 k_{u2} \end{bmatrix} \begin{Bmatrix} u_1 \\ u_2 \\ u_3 \\ r_1 \\ -r_2 \\ r_3 \end{Bmatrix}_j \quad (3.7)$$

This relationship also holds for a two joint element if all the displacements at joint I are zero.

The cold formed steel reinforced concrete frame with strawbale infill panel contains concrete beams and columns with infill straw bale panels a cold formed concrete composite slab deck and a plastered strawbale wall. Figure 3.2 shows the typical floor plan of the building. The front and side view of the building were presented in Figures 3.3 and 3.4 respectively.

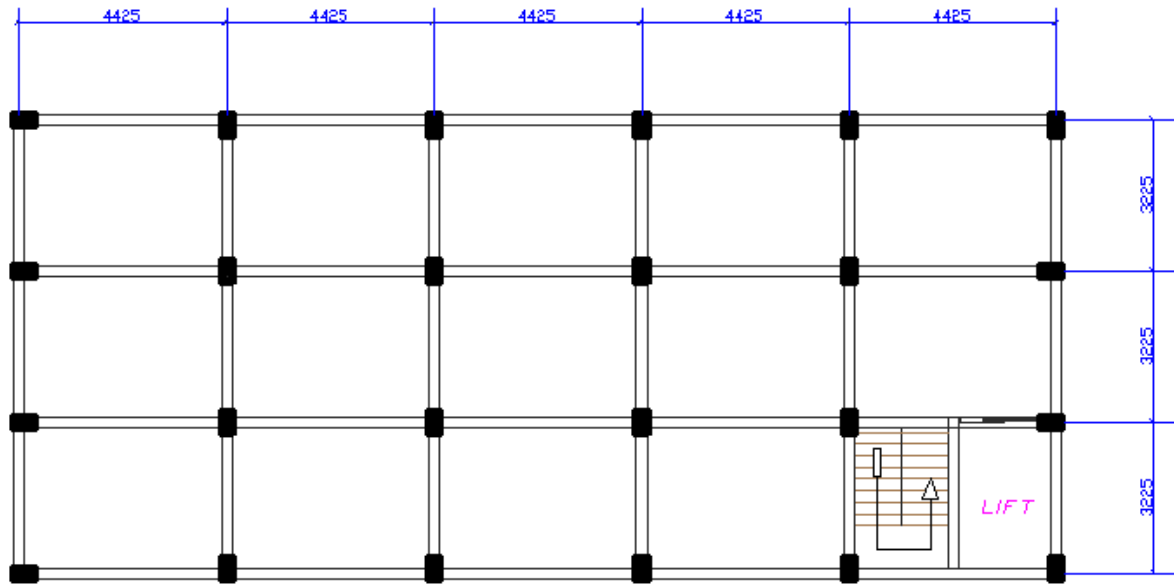


Figure 3.2: Typical Floor Plan for the Building

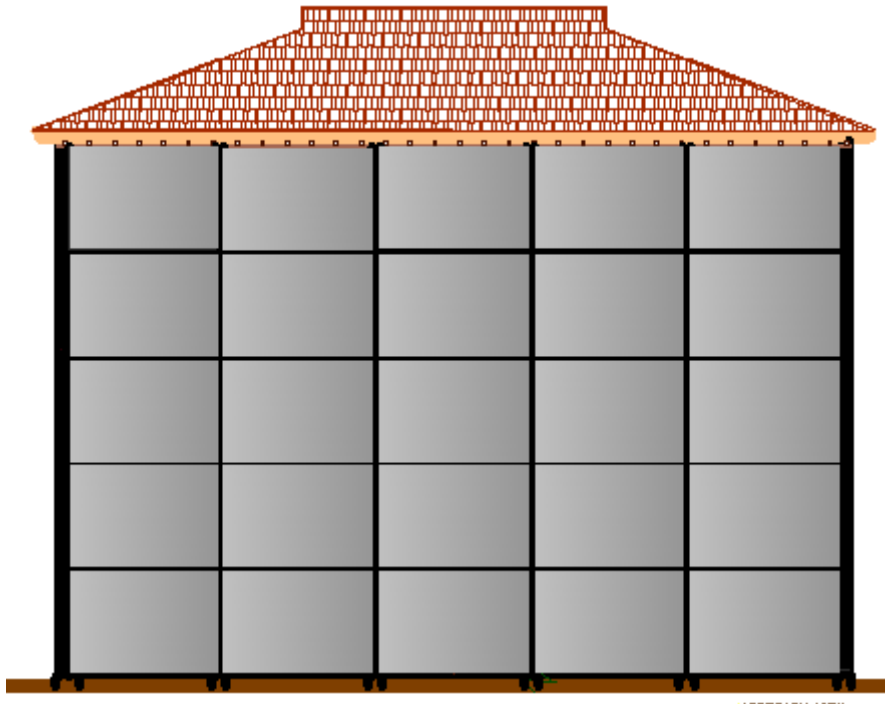


Figure 3.3: Front View of the 4 Storey Building

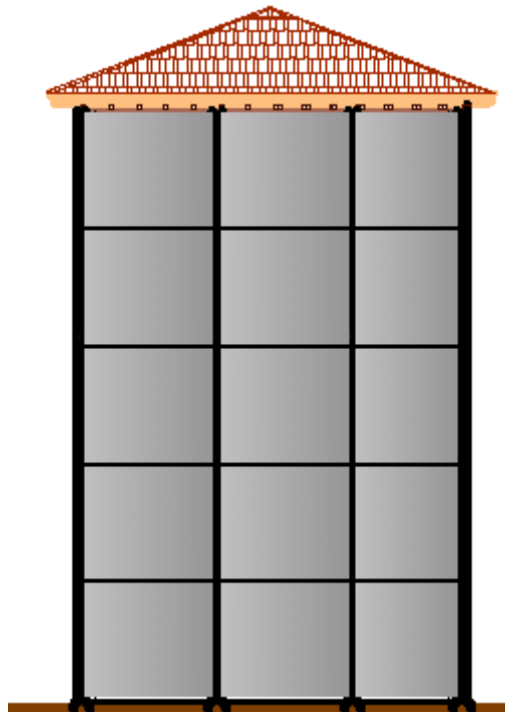


Figure 3.4: Side View of the Building

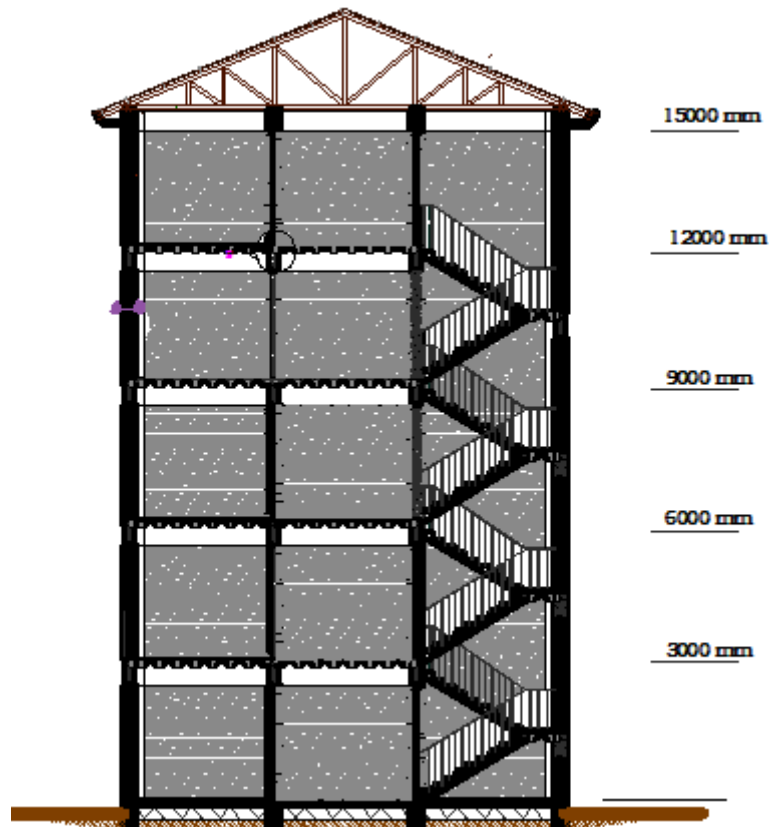


Figure 3.5: Sectional View of Building.

3.2.1 Analytical Simulation of the 4 Storey Building

The general process for the modelling of a building structure in CSI ETABS is shown in Appendix A. The flowchart that is used for the analysis of the 4-storey cold-formed steel reinforced concrete frame with plastered strawbale wall is shown in Figure 3.6.

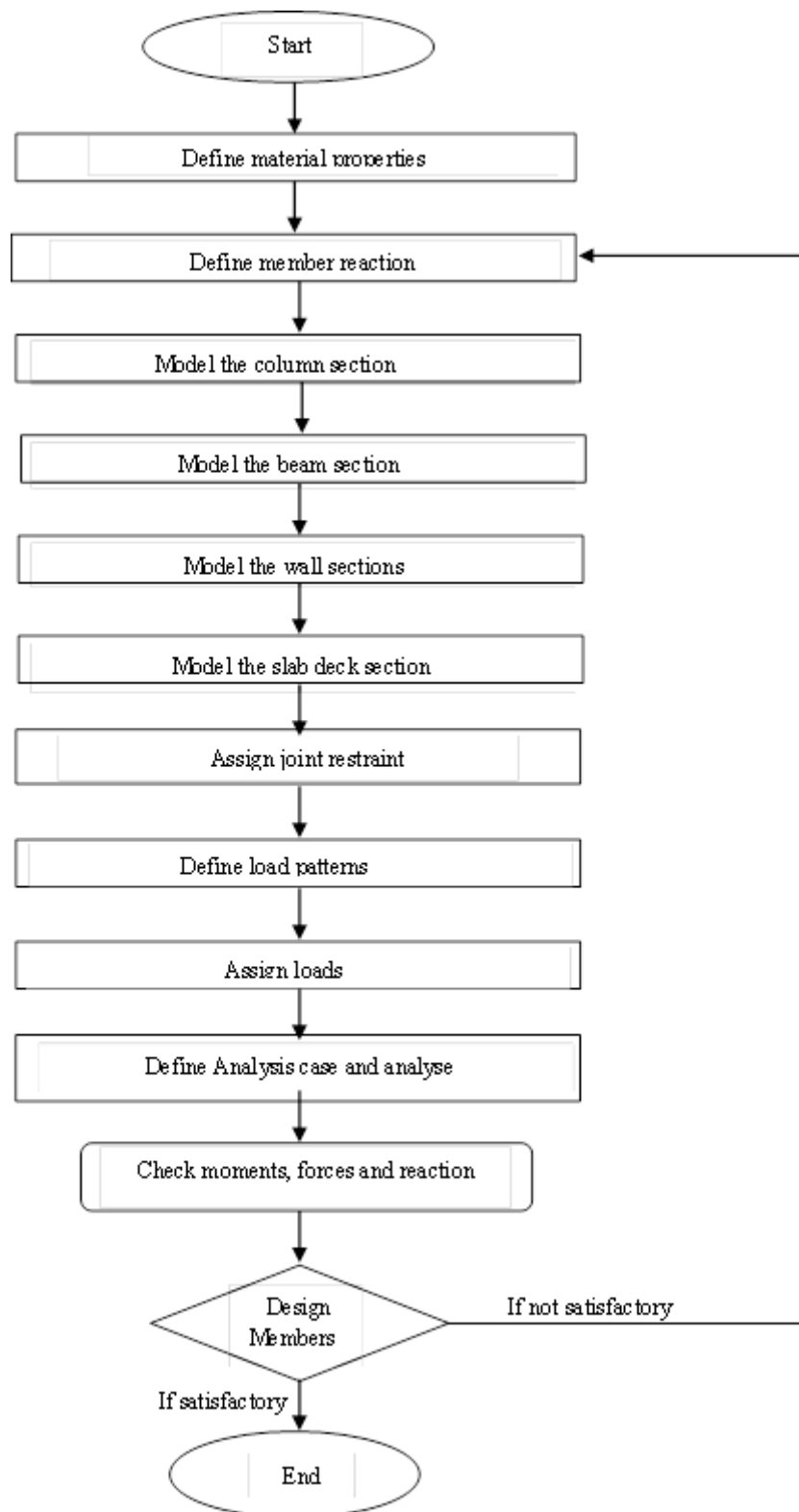


Figure 3.6: CSI ETABS Flowchart for Building Analysis and Design

3.2.2 Material Properties Definition

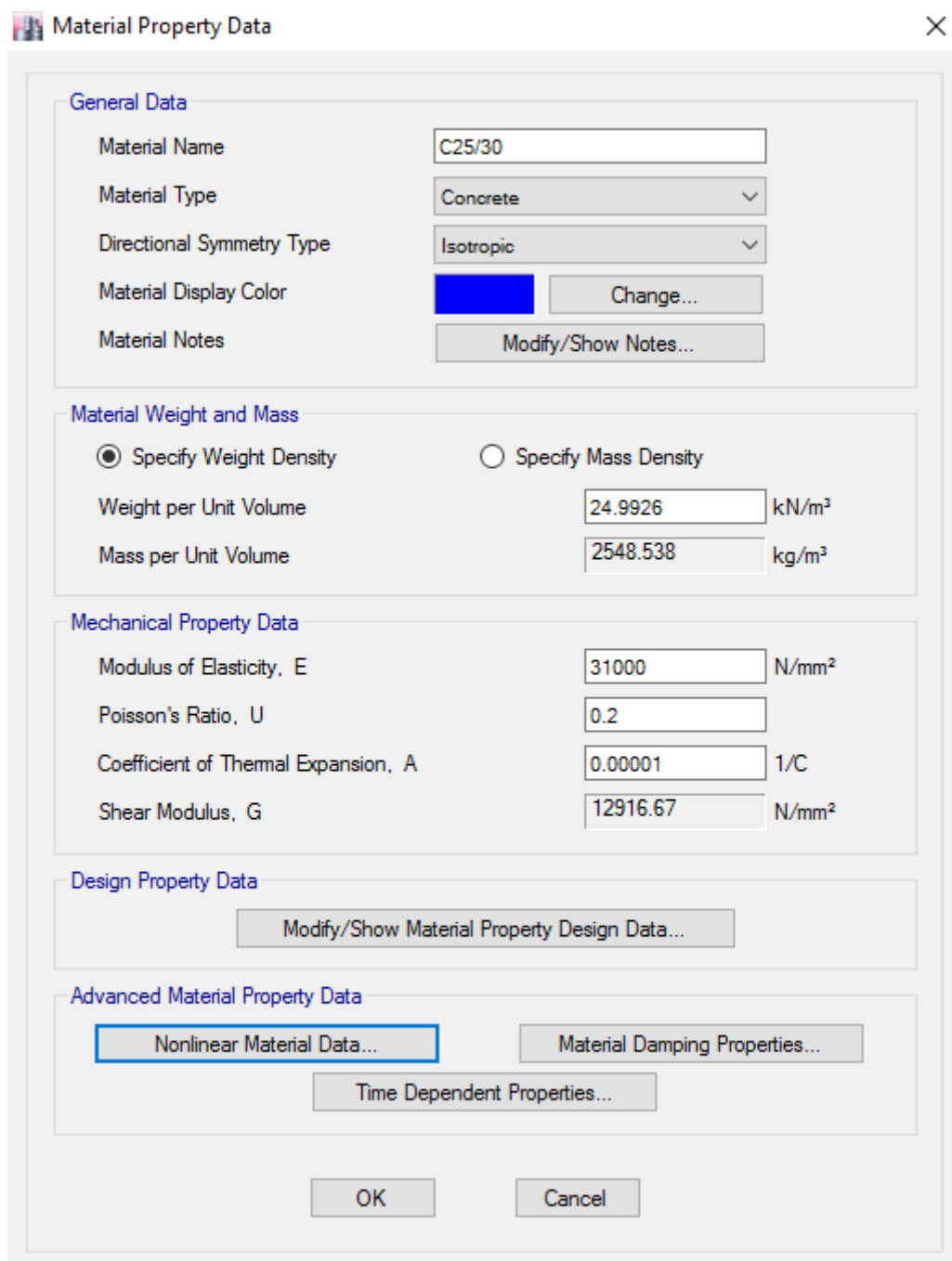
The material properties like modulus of elasticity density shear modulus compressive strength concrete Poisson's ratio and coefficient of thermal expansion were modelled in this

research work. These properties of the materials in Table 3.1 were used for the analysis of the building.

Table 3.1: Material properties

Material	Property	Value	Source
Cold Formed Steel	Density KN/m^3	76.97	ETABS 2016
	Modulus of Elasticity KN/m^2	203396360	
	Poisson Ratio	0.33	
Concrete	Density KN/m^3	24.9926	Asiz and Ahmed 2013
	Modulus of Elasticity KN/m^2	31000	
	Poisson Ratio	0.2	
	Compressive Strength N/mm^2	25	
Strawbale	Density KN/m^3	22.3	Bruce 2003
	Modulus of Elasticity KN/m^2	22990	
	Poisson Ratio	0.3	
Cement Plaster	Density KN/m^3	19	Ashour 2003
	Modulus of Elasticity KN/m^2	20250	
	Poisson Ratio	0.2	

The interface of CSI ETABS with the material properties of reinforced concrete is shown in Figure 3.7. The weight per unit volume of the concrete the modulus of elasticity Poisson's ratio coefficient of thermal expansion shear modulus and the concrete strength were defined. The non-linear material properties and the stress-strain plot are shown in Figure 3.8 and 3.9 respectively.



The image shows a 'Material Property Data' dialog box with a close button (X) in the top right corner. The dialog is organized into several sections: 'General Data', 'Material Weight and Mass', 'Mechanical Property Data', 'Design Property Data', and 'Advanced Material Property Data'. Each section contains various input fields and buttons for defining material properties.

General Data

Material Name	C25/30
Material Type	Concrete
Directional Symmetry Type	Isotropic
Material Display Color	 Change...
Material Notes	Modify/Show Notes...

Material Weight and Mass

☒ Specify Weight Density ☐ Specify Mass Density

Weight per Unit Volume	24.9926	kN/m ³
Mass per Unit Volume	2548.538	kg/m ³

Mechanical Property Data

Modulus of Elasticity, E	31000	N/mm ²
Poisson's Ratio, U	0.2	
Coefficient of Thermal Expansion, A	0.00001	1/C
Shear Modulus, G	12916.67	N/mm ²

Design Property Data

Modify/Show Material Property Design Data...

Advanced Material Property Data

Nonlinear Material Data... Material Damping Properties...
Time Dependent Properties...

OK Cancel

Figure 3.7: Material Property Definition Interface for Concrete

Nonlinear Material Data

Material Name and Type

Material Name: C25/30

Material Type: Concrete, Isotropic

Miscellaneous Parameters

Hysteresis Type: Concrete

Modify/Show Hysteresis Parameters...

Drucker-Prager Parameters

Friction Angle: 0 deg

Dilatational Angle: 0 deg

Acceptance Criteria Strains

	Tension	Compression	
IO	0.01	0.003	mm/mm
LS	0.02	0.006	mm/mm
CP	0.05	0.015	mm/mm

☒ Ignore Tension Acceptance Criteria

Stress Strain Curve Definition Options

☒ Parametric: Mander

Convert to User Defined

☐ User Defined

Parametric Strain Data

Strain at Unconfined Compressive Strength, f'_c : 0.001613

Ultimate Unconfined Strain Capacity: 0.005

Final Compression Slope (Multiplier on E): -0.1

Show Stress-Strain Plot...

OK Cancel

Figure 3.8: Nonlinear Material Data for Concrete

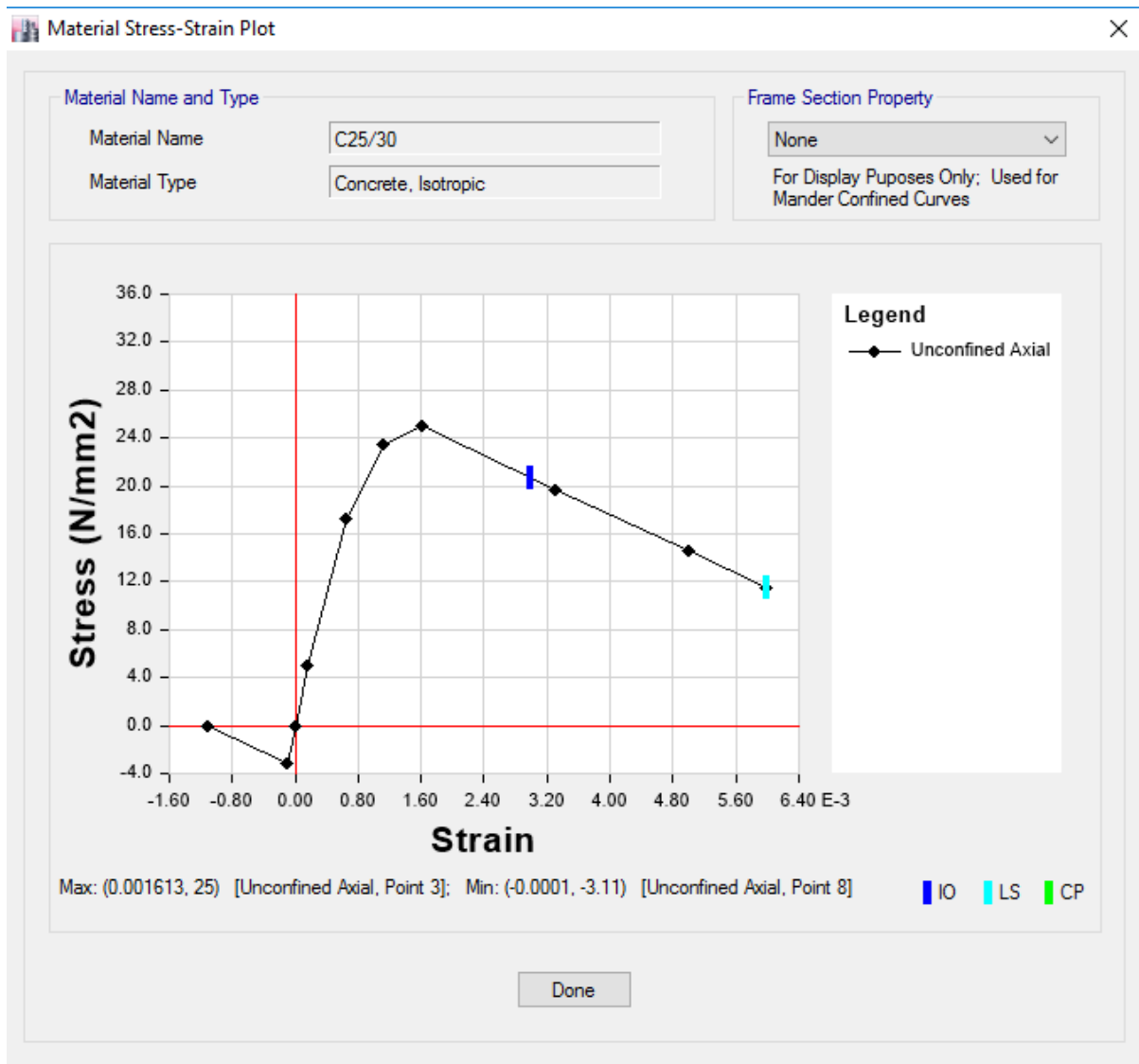


Figure 3.9: Stress-Strain Plot for Concrete

The plot of the stress-strain curve for the concrete used in this research were obtained from the concrete property in ETABS. It has its maximum value at 0.001613 strain at 25 N/mm² stress and its minimum value at -0.0001 strain at -3.11 N/mm² stress.

Figure 3.10 shows the material properties definition interface for the cold-formed steel and its non-linear material properties in Figure 3.11. The cold-formed steel was modelled as an isotropic material. The weight per unit volume Poisson's ratio coefficient of thermal expansion and shear modulus were defined.

Material Property Data

General Data

Material Name:

Material Type:

Directional Symmetry Type:

Material Display Color:

Material Notes:

Material Weight and Mass

☒ Specify Weight Density ☐ Specify Mass Density

Weight per Unit Volume: kN/m³

Mass per Unit Volume: kg/m³

Mechanical Property Data

Modulus of Elasticity, E: N/mm²

Poisson's Ratio, U:

Coefficient of Thermal Expansion, A: 1/C

Shear Modulus, G: N/mm²

Design Property Data

Advanced Material Property Data

Figure 3.10: Material property Definition Interface for Cold-Formed Steel

Material Name and Type

Material Name
ColdFormed

Material Type
ColdFormed, Isotropic

Miscellaneous Parameters

Hysteresis Type
Kinematic

Acceptance Criteria Strains

	Tension	Compression	
IO	0.01	0.005	mm/mm
LS	0.02	0.01	mm/mm
CP	0.05	0.02	mm/mm

Stress Strain Curve Definition Options

☐ Parametric

☒ User Defined

Convert to User Defined

User Stress-Strain Curve Data

Number of Points in Stress-Strain Curve
3

Point Number	Strain	Stress (N/mm2)	Point ID
1	-0.001695	-344.74	
2	0	0	A
3	0.001695	344.74	

Show Plot...

OK

Cancel

Figure 3.11: Nonlinear Material Data for Cold-Formed Steel

Figures 3.12 and 3.13 show the material definition interfaces for strawbale and cement plaster respectively. The weight per unit volume Poisson's ratio coefficient of thermal expansion and shear modulus were defined and also modelled as an isotropic material.

Material Property Data

General Data

Material Name

Straw bale

Material Type

Other

Directional Symmetry Type

Isotropic

Material Display Color

Change...

Material Notes

Modify/Show Notes...

Material Weight and Mass

☒ Specify Weight Density
☐ Specify Mass Density

Weight per Unit Volume

22.3

kN/m³

Mass per Unit Volume

0

N-s²/mm⁴

Mechanical Property Data

Modulus of Elasticity, E

22990

kN/m²

Poisson's Ratio, U

0.3

Coefficient of Thermal Expansion, A

0.0000099

1/C

Shear Modulus, G

8.84

MPa

Design Property Data

Modify/Show Material Property Design Data...

Advanced Material Property Data

Nonlinear Material Data...

Material Damping Properties...

Time Dependent Properties...

OK

Cancel

Figure 3.12: Interface for the Material Properties of Straw-bale

Material Property Data

General Data

Material Name
Cement plaster

Material Type
Other

Directional Symmetry Type
Isotropic

Material Display Color
Change...

Material Notes
Modify/Show Notes...

Material Weight and Mass

☒ Specify Weight Density
☐ Specify Mass Density

Weight per Unit Volume
19
kN/m³

Mass per Unit Volume
0
N-s²/mm⁴

Mechanical Property Data

Modulus of Elasticity, E
20250
kN/m²

Poisson's Ratio, U
0.2

Coefficient of Thermal Expansion, A
0.0000099
1/C

Shear Modulus, G
8.44
MPa

Design Property Data

Modify/Show Material Property Design Data...

Advanced Material Property Data

Nonlinear Material Data...
Material Damping Properties...

Time Dependent Properties...

OK
Cancel

Figure 3.13: Interface for the Material Properties for Cement Plaster

3.2.3 Member Sections Definition

The column supporting the structure is modelled as a composite column with the concrete being the fill material for the cold-formed steel encasement. The section of the column is shown in Figure 3.14 and the interface for the column property in Figure 3.15.

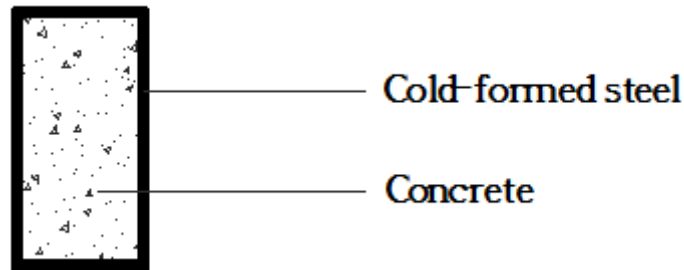


Figure 3.14: Section of the Cold-Formed Concrete Composite Column

The cold formed steel has a depth of 450 mm and a width of 225 mm. The gauge of the cold formed steel has a thickness of 8 mm. The 8 mm thickness was used since pre-galvanized material is not required for its application Kumar and Kumar 2002.

Frame Section Property Data

General Data

Property Name: Concrete Composite Column 450x225mm

Material: ColdFormed

Display Color: Change...

Notes: Modify/Show Notes...

Shape

Section Shape: Filled Steel Tube

Section Property Source

Source: User Defined

Section Dimensions

Total Depth: 450 mm

Total Width: 225 mm

Flange Thickness: 8 mm

Web Thickness: 8 mm

Corner Radius: 0 mm

Show Section Properties...

Fill

Fill Material: C25/30

Property Modifiers

Modify/Show Modifiers...
Currently Default

OK
Cancel

Figure 3.15: Interface for the Frame Section Property for the Cold-Formed Steel Concrete Column

A total depth of 450 mm and width of 225 mm were defined for the column. The flange thickness cold-formed steel encasing the concrete was defined as 8 mm.

The beams also have the same configuration as the columns by having a cold-formed steel tube with the filled material as concrete. The frame section definition of the beam is shown in Figure 3.16. The beams also have the same configuration as the columns by having a cold-formed steel tube with concrete as the filled material. The flange thickness is reduced to 4 mm for the beams.

Frame Section Property Data

General Data

Property Name: Cold Formed Concrete Composite Beam 450x225

Material: Cold Formed

Display Color: Change...

Notes: Modify/Show Notes...

Shape

Section Shape: Filled Steel Tube

Section Property Source

Source: User Defined

Section Dimensions

Total Depth: 450 mm

Total Width: 225 mm

Flange Thickness: 4 mm

Web Thickness: 4 mm

Corner Radius: 0 mm

Show Section Properties...

Fill

Fill Material: C25/30

Property Modifiers

Modify/Show Modifiers...
Currently Default

OK
Cancel

Figure 3.16: Frame Section Property Definition for the Cold-Formed Steel Concrete Beam

The composite slab is made up of concrete placed on a cold-formed steel deck and held together by shear studs. The section of the cold-formed beam is shown in Figure 3.17a while the steel hollow section for the cold-formed column with welded seam for concrete cold-formed interaction is shown in Figure 3.17b. Figure 3.18 shows the cold-formed strawbale interaction

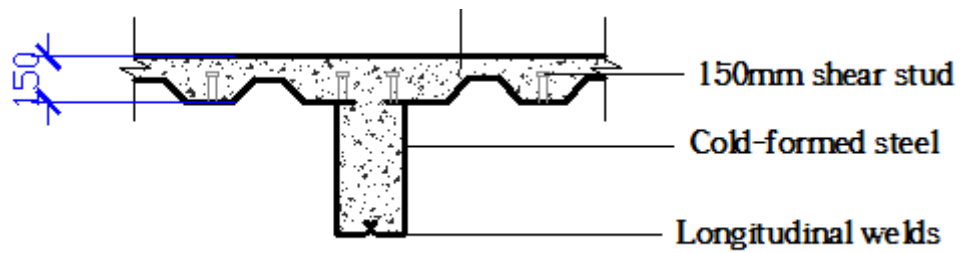


Figure 3.17a: Section of the Cold-Formed Beam



Figure 3.17b: Steel Hollow Section for Cold-Formed Column EN10219 1997

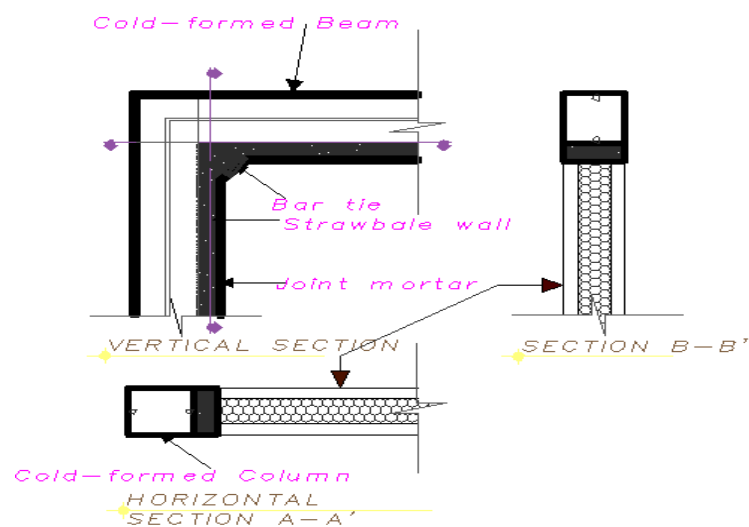


Figure 3.18: Cold-formed frame Strawbale interaction

General Data	
Property Name	ad Concrete Composite Slab
Type	Filled
Slab Material	C25/30
Deck Material	ColdFormed
Modeling Type	Membrane
Modifiers (Currently Default)	Modify/Show...
Display Color	Change...
Property Notes	Modify/Show...

Property Data	
Slab Depth, tc	75 mm
Rib Depth, hr	75 mm
Rib Width Top, wrt	175 mm
Rib Width Bottom, wrb	125 mm
Rib Spacing, sr	300 mm
Deck Shear Thickness	4 mm
Deck Unit Weight	0.0001 N/mm ²
Shear Stud Diameter	19 mm
Shear Stud Height, hs	130 mm
Shear Stud Tensile Strength, Fu	400 MPa

OK Cancel

Figure 3.19: Interface of the Composite Deck Property Section

The shear wall was modelled with concrete material to support the elevator. The definition of the shear wall is shown in Figure 3.20.

Figure 3.20: Interface for the Shear Wall Property

Oduola 2006 posited that for a triangular element of the wall prism the element stiffness matrix $[K^e]$ would be a 6 x 6 matrix for the plane elasticity triangle as shown in Figure 3.21. Because of the existence of two degrees of freedom DOF at each node of the triangular element the Nodal force vector $[F^e]$ can be related to the displacement vector $[\delta^e]$ in Equation 3.8.

$$\{F^e\} = [K^e]\{\delta^e\} \quad 3.8$$

For each element there are six independent nodal coordinates thus the assumed displacement function at a node i with coordinate x_i, y_i will be:

$$\delta_{xi} = \alpha_1 + \alpha_2 x_i + \alpha_3 y_i \quad 3.9$$

$$\delta_{yi} = \alpha_4 + \alpha_5 x_i + \alpha_6 y_i \quad 3.10$$

In matrix form equations 3.9 and 3.10 can be written for a node as

$$\begin{bmatrix} \delta_{xi} \\ \delta_{yi} \end{bmatrix} = \begin{bmatrix} 1 & xi & yi & 0 & 0 & 0 \\ 0 & 0 & 0 & 1 & xi & yi \end{bmatrix} \begin{bmatrix} \alpha_1 \\ \alpha_2 \\ \alpha_3 \\ \alpha_4 \\ \alpha_5 \\ \alpha_6 \end{bmatrix} \quad 3.11$$

Where $\alpha_1 \dots \alpha_6$ are constants which may be determined from the coordinates of the nodal points of the elements.

Figure 3.21 shows a section of the plastered strawbale wall. The strawbale covers about 89% of the total wall breadth 201 mm while the cement plaster which bounds the strawbale on both sides is about 11% of the total wall breadth 12 mm thickness on both faces to make 24 mm.

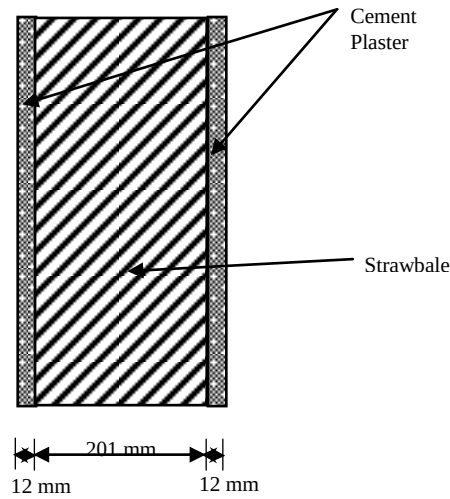


Figure 3.21: Section of the Plastered Strawbale Wall

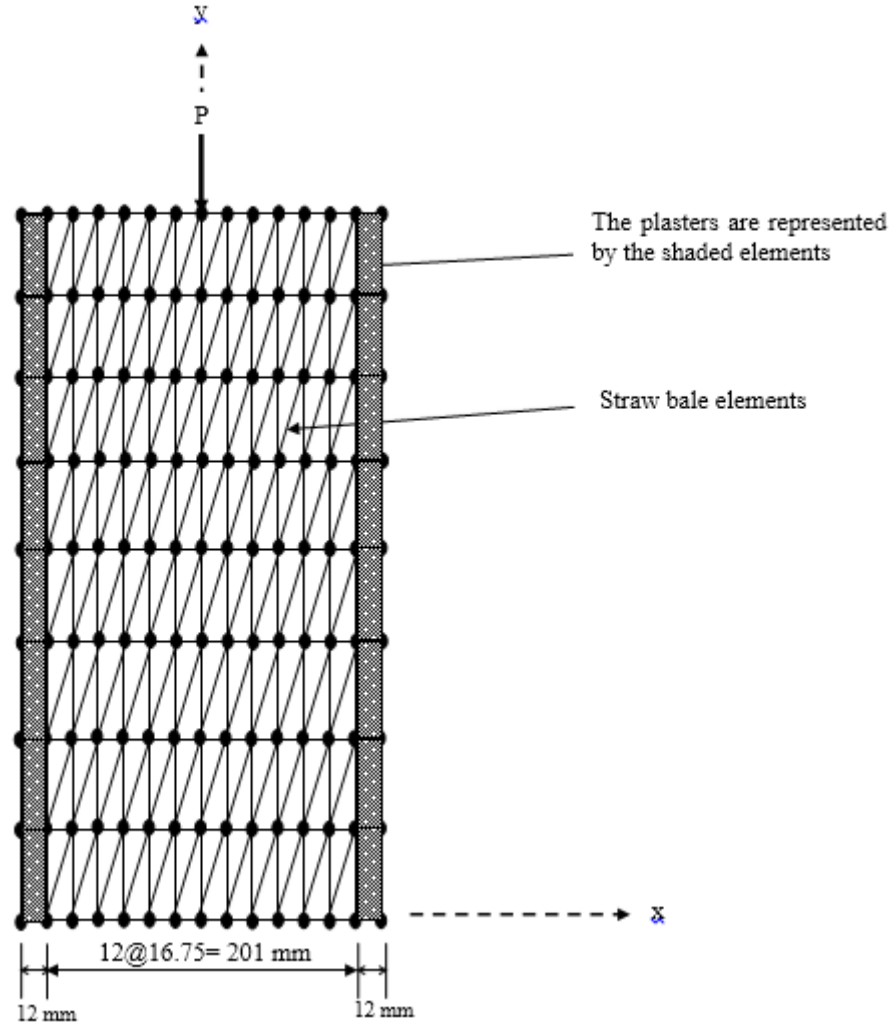


Figure 3.22: Finite Element Model of the Plastered Strawbale wall

For the three nodes $i j k$ of the element Equation 3.11 can be written as follows

$$\begin{aligned}\delta_{xi} &= \alpha_1 + \alpha_2 xi + \alpha_3 yi \\ \delta_{xj} &= \alpha_1 + \alpha_2 xj + \alpha_3 yj \\ \delta_{xk} &= \alpha_1 + \alpha_2 xk + \alpha_3 yk \\ \delta_{yi} &= \alpha_4 + \alpha_5 xi + \alpha_6 yi \\ \delta_{yj} &= \alpha_4 + \alpha_5 xj + \alpha_6 yj \\ \delta_{yk} &= \alpha_4 + \alpha_5 xk + \alpha_6 yk\end{aligned}$$

This can be written as

$$\begin{bmatrix} \delta_{xi} \\ \delta_{xj} \\ \delta_{xk} \\ \delta_{yi} \\ \delta_{yj} \\ \delta_{yk} \end{bmatrix} = \begin{bmatrix} 1 & xi & yi & 0 & 0 & 0 \\ 1 & xj & yj & 0 & 0 & 0 \\ 1 & xk & yk & 0 & 0 & 0 \\ 0 & 0 & 0 & 1 & xi & yi \\ 0 & 0 & 0 & 1 & xj & yj \\ 0 & 0 & 0 & 1 & xk & yk \end{bmatrix} \begin{bmatrix} \alpha_1 \\ \alpha_2 \\ \alpha_3 \\ \alpha_4 \\ \alpha_5 \\ \alpha_6 \end{bmatrix} \quad 3.12$$

$$\{\delta\} = [C]\{\alpha\} \quad 3.13$$

Where $[C]$ is 6 x 6 matrix for the three nodal displacements

$$[C]^{-1}\{\delta\} = \{\alpha\} \quad 3.14$$

$$\{\varepsilon\} = [H]\{\alpha\} \quad 3.15$$

And

$$\{\varepsilon\} = [H][C]^{-1}\{\delta\} \quad 3.16$$

Hence

$$\{\varepsilon\} = [B]\{\delta\} \quad 3.17$$

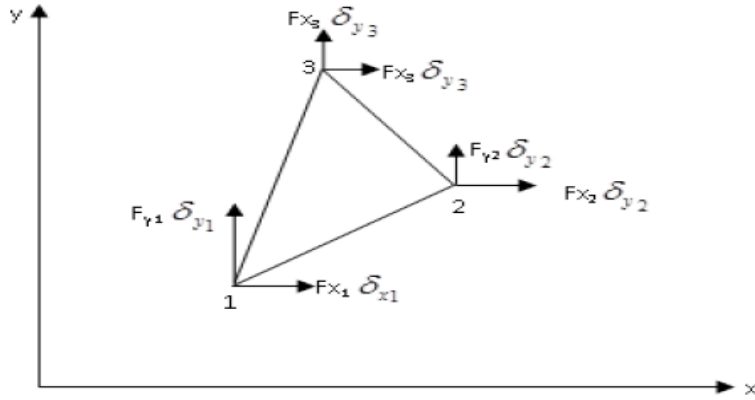


Figure 3.23: A two- dimensional triangular element

The stress-strain relationship for an element depends on the material of the element; hence the mechanical properties of the material the matrix D can be obtained as follows:

$$D = \begin{bmatrix} \frac{E_x}{1-\nu_{xy}\nu_{yx}} & \frac{E_x\nu_{yx}}{1-\nu_{xy}\nu_{yx}} & 0 \\ \frac{E_y\nu_{xy}}{1-\nu_{xy}\nu_{yx}} & \frac{E_y}{1-\nu_{xy}\nu_{yx}} & 0 \\ 0 & 0 & \frac{E_y}{2(1+\nu_{yx})} \end{bmatrix} \quad 3.18$$

Where E_x and E_y represent moduli of elasticity in the x and y-direction respectively ν_{xy} and ν_{yx} represents Poisson's ratio in the xy and yx plane respectively.

For isotropic element Equation 3.15 can be written as:

$$D = \begin{bmatrix} \frac{E}{1-\nu} & \frac{E\nu}{1-\nu^2} & 0 \\ \frac{E\nu}{1-\nu^2} & \frac{E}{1-\nu^2} & 0 \\ 0 & 0 & \frac{E}{2(1+\nu)} \end{bmatrix} \quad 3.19$$

Where E is the modulus of elasticity and ν is the Poisson's ratio. The interrelationship between nodal forces and applied stresses may be written in simplified matrix form as:

$$\{F\} = [A][\sigma] \quad 3.20$$

The stiffness matrix for the triangular element can be written as:

$$[K^e] = [A][D][B] \quad 3.21$$

The plastered strawbale wall was modelled in layers in ETABS with a total section thickness of 225 mm as shown in Figure 3.24.

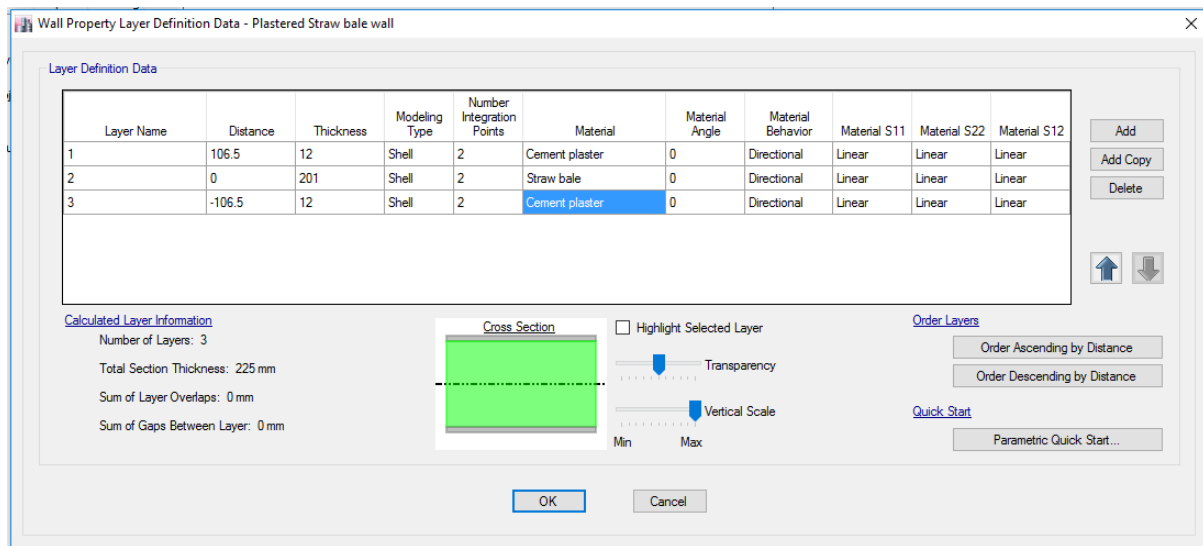


Figure 3.24: Interface of the Wall Property Layer Definition of the Plastered Strawbale Wall

3.2.4 Modelling of the Building

The structural members defined were used to model the cold-formed steel concrete composite building with strawbale walls. Figure 3.25 shows the plan view of a typical floor model. Likewise the 3D view of the model is shown in Figure 3.26.

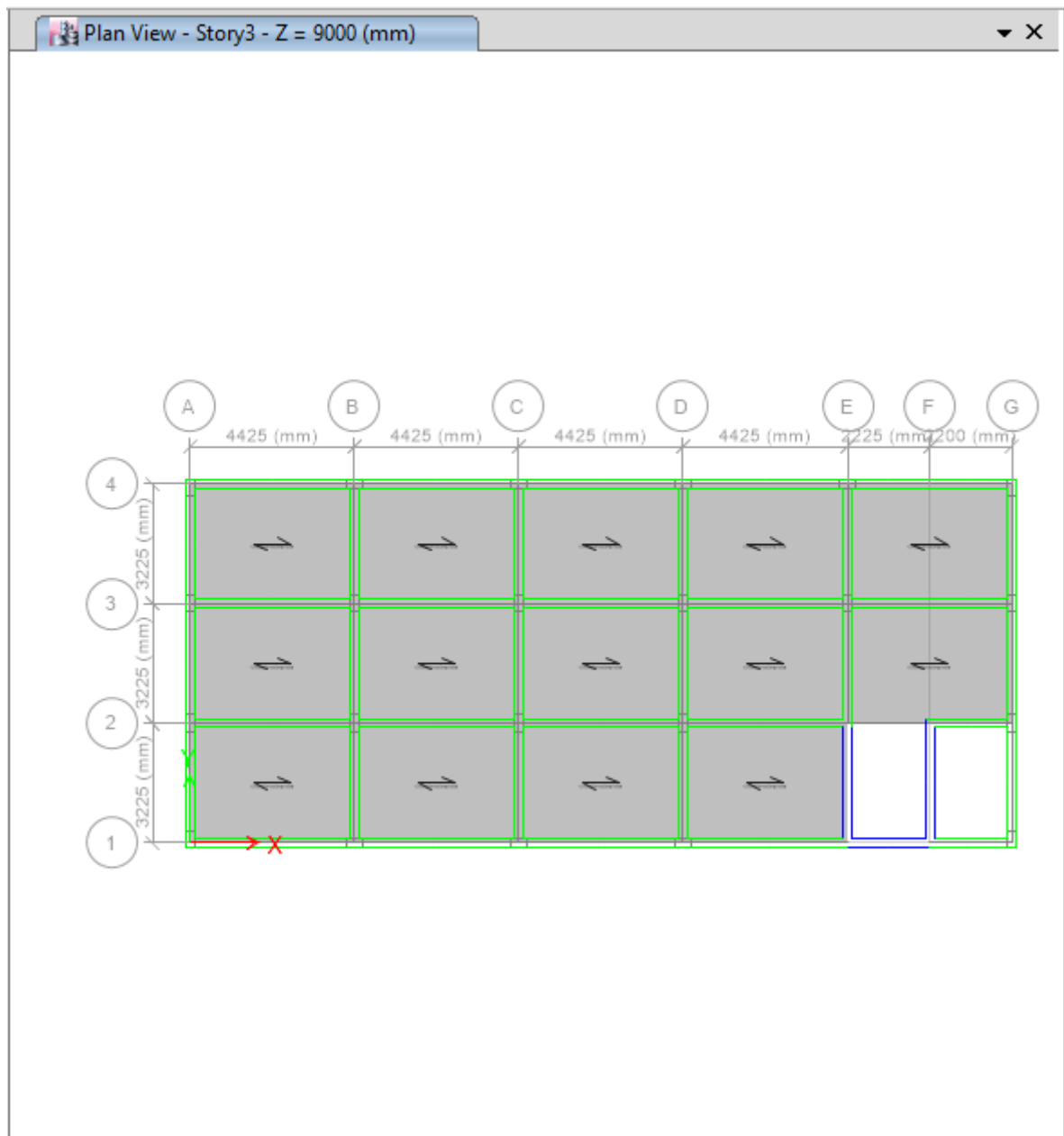


Figure 2.25: Plan View of the Modelled Building.

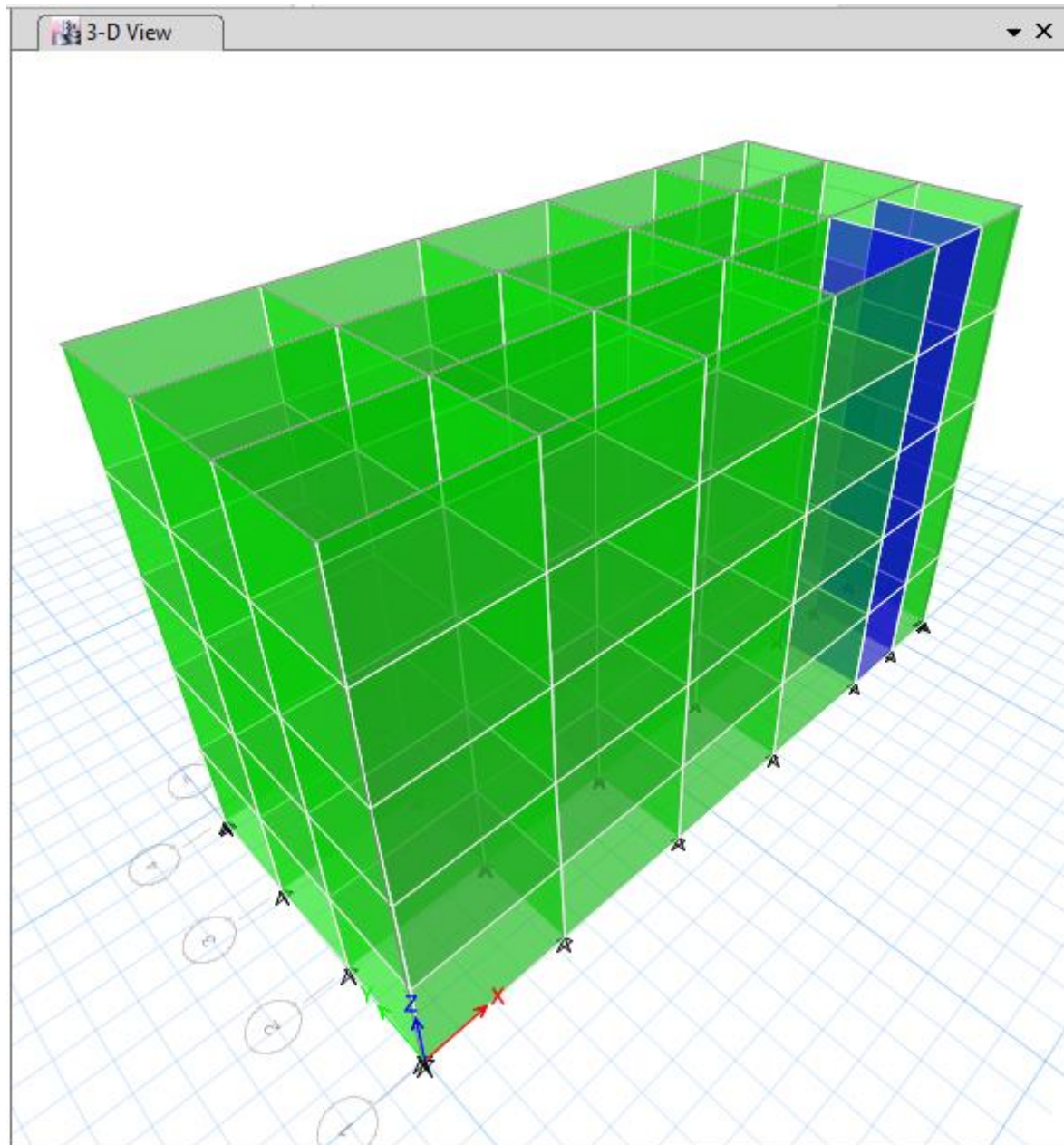


Figure 3.26: 3D View of the Modelled Building

The columns were assumed to be fixed at the base and hence fixed support was assigned to all the columns bases with restraints assigned to all the joints in global directions.

3.2.5 Loading Definitions

The building under consideration was analyzed based on the following load cases based on the specifications of Eurocode 8.

- i. Dead Load: This is the self-weight of all the building components.
- ii. Live Load: This is the imposed load on the structural element slab deck

- iii. Roof Live: This is the imposed roof load on the roof beams.
- iv. Wind Load: This is the load caused by wind. The wind load was applied to the plastered strawbale walls.
- v. Response Spectrum: In this project the horizontal elastic response spectrum obtained from Eurocode8 EN 1998-1:2005 Clause 3.2.2.2 was used to determine the response of the building structure to ground acceleration.
- vi. Time History: These are historical records of earthquake occurrences. There are many earthquake records but in this project five different earthquake accelerogram were used for the Time History analysis of the building structure.

3.2.6 Response Spectrum

The response spectrum analysis was loaded using the Haiti earthquake model. The earthquake which occurred on 12 January 2010 has an epicentre near the town of Leogane which is about 25 kilometres west of Port-au-Prince Haiti capital. It has a magnitude of 7.0 M_w with a peak acceleration of 0.5 g. In Port-au-Prince strong shakings with intensity IX on the Modified Mercalli scale were recorded. The damage from the earthquake was severe and catastrophic leaving thousands of buildings collapsed with an unknown number of people trapped and hundreds of thousands of people homeless Margesson and Taft-Morales 2010. The shakings were also felt in several surrounding regions and countries like Cuba Jamaica Venezuela Puerto Rico and the bordering Republic Wikipedia 2018. The earthquake was the most powerful earthquake in over 200 years causing a record death over 300 000 people injuries and displacements. Figure 3.27 shows the map of Haiti depicting the intensity of shaking and the degree of damage incurred by January 12 2010 earthquake.

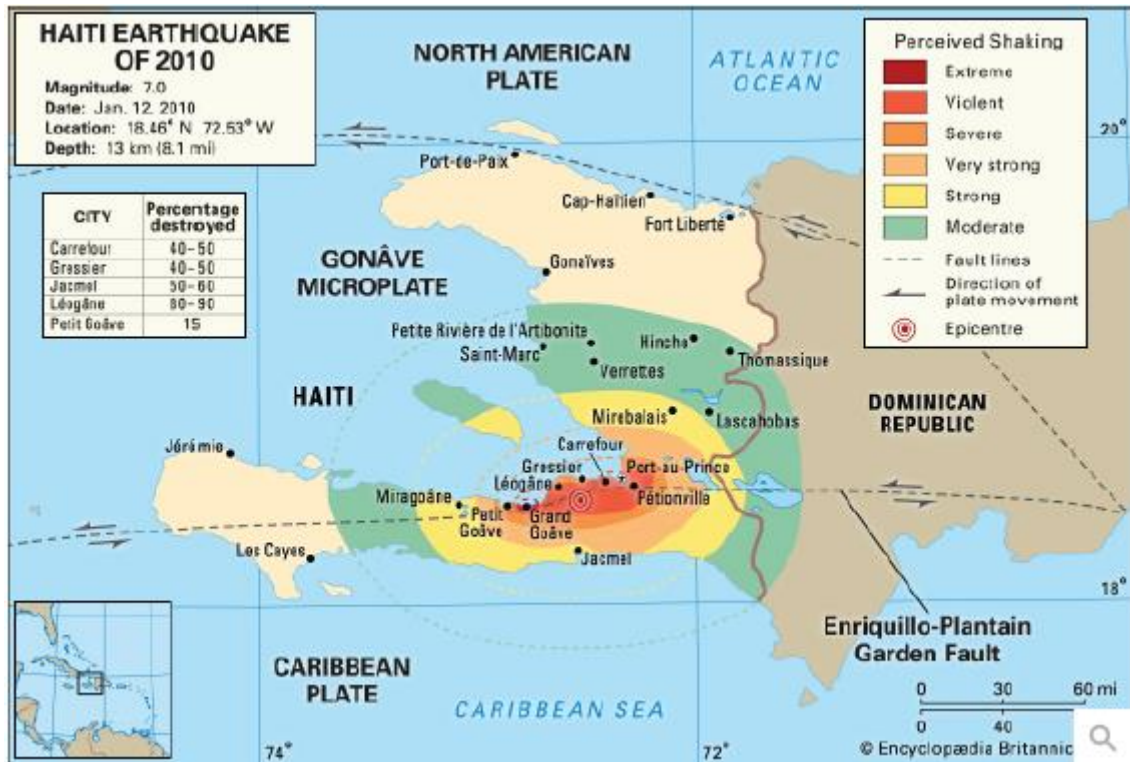


Figure 3.27: Map of Haiti Showing the Intensity of Shaking and the Degree of Damage Caused by the 2010 Earthquake MacLeod et al. 2016

The structure was assumed to be in Haiti and thus all the environmental conditions of Haiti were used to model the loadings. Figure 3.27 shows the average annual temperature and precipitation in Haiti based on climate station data for 1960 to 1990.

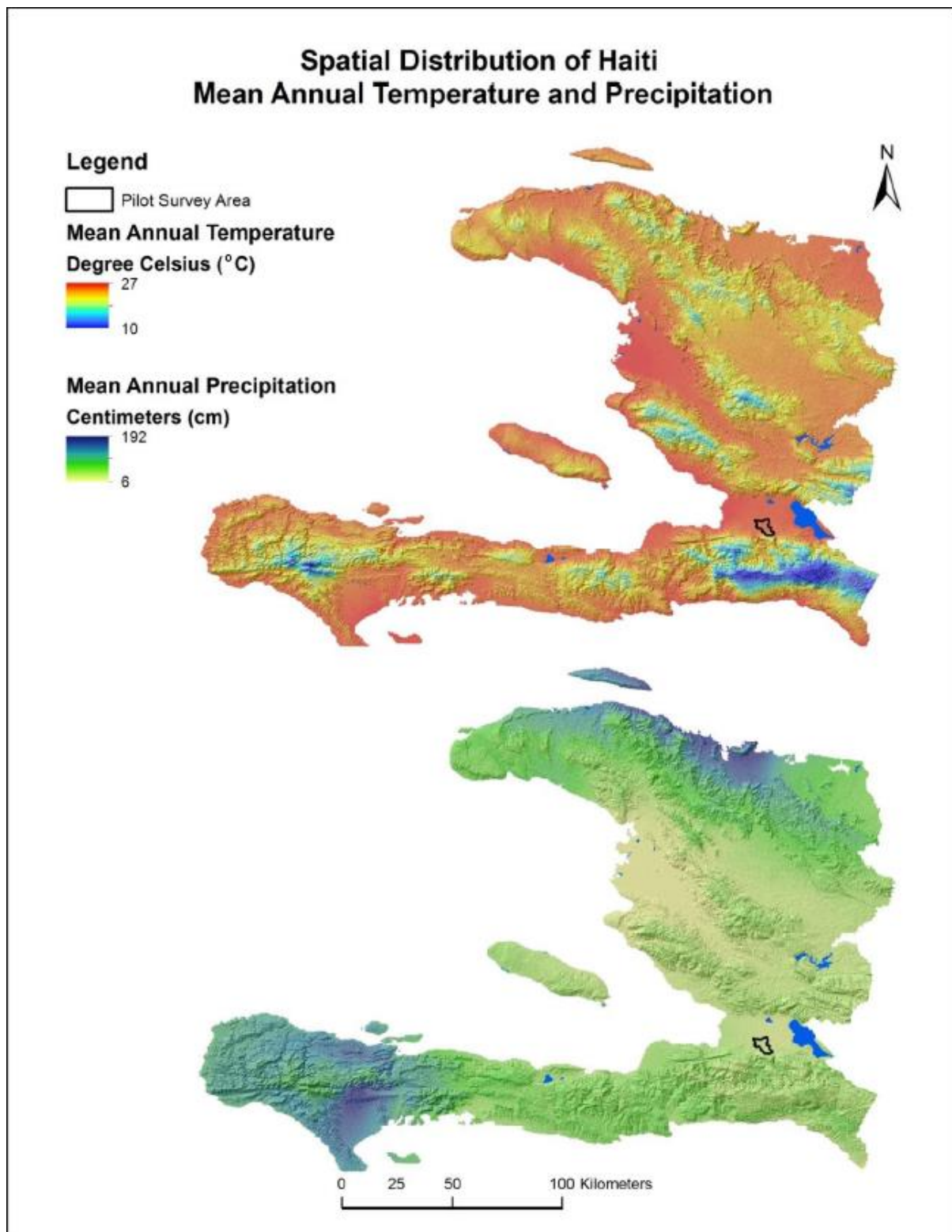


Figure 3.28: Spatial Distribution of Haiti Mean Annual Temperature and Precipitation for 1960 to 1990 Rolfes et al. 2014

Haiti has a humid warm tropical climate that is characterized by diurnal temperature variations which are much greater than the annual variations; temperatures are modified by

elevation. Average temperatures range from about 25 °C in January and February to about 30 °C in July and August. The village of Kenscoff at some 1430 metres has an average temperature of about 16 °C whereas Port-au-Prince at sea level has an average of 26 °C MacLeod et al. 2016. In winter frost are of higher percentage to occur at high elevations. Haiti can be found on the leeward side of the island and this means that the humid trade winds influence is not as great as it is in the Dominican Republic.

A wind velocity of 9.5 m/s was loaded to the structure which depicts the wind condition of Haiti. NREL 2014 shows in Figure 3.29 the annual average wind speed of Haiti at 80 m above sea level.

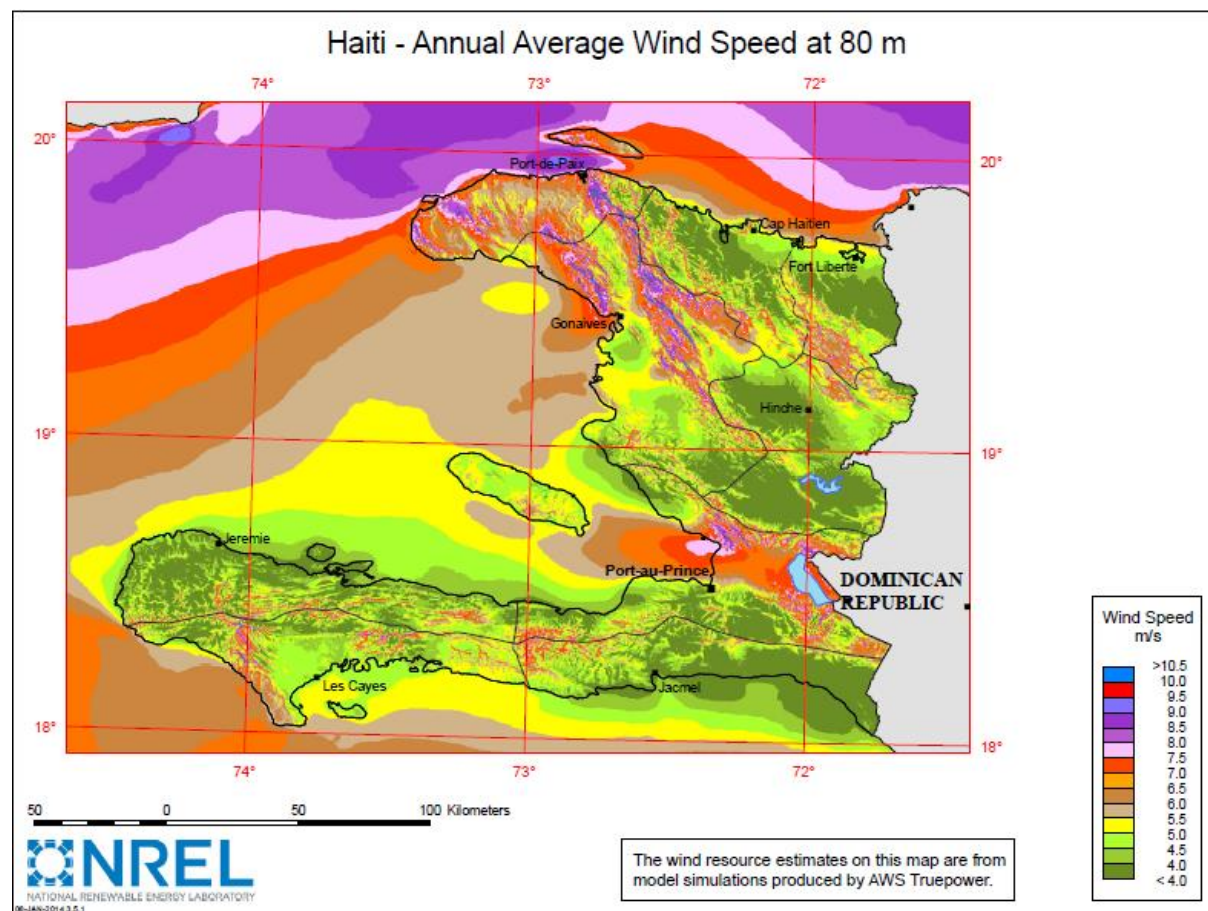


Figure 3.29: Annual Average Wind Speed of Haiti at 80 m NREL 2014

Figure 3.30 shows the load patterns that were defined and used to analyze the structure.

Various loads were added to the building. Live loads of 2.0 kN/m² was added to the slab as an imposed load and a roof load of 0.75 kN/m² added to the roof beams as the load coming on top of the roof beams.

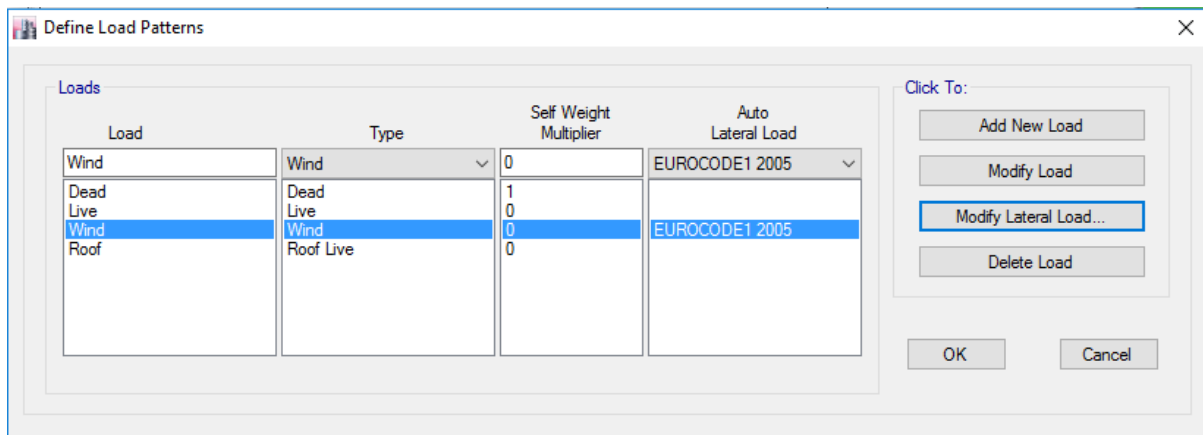


Figure 3.30: Load Patterns of the Building

The wind parameters were defined based on the specifications in Eurocode 1 EN 1991-1-4:2005 as shown in Figure 3.31. The wind velocity of 9.5 m/s which is the observed wind of Haiti was used and the terrain category of II was also used according to the Eurocode 1 specification.

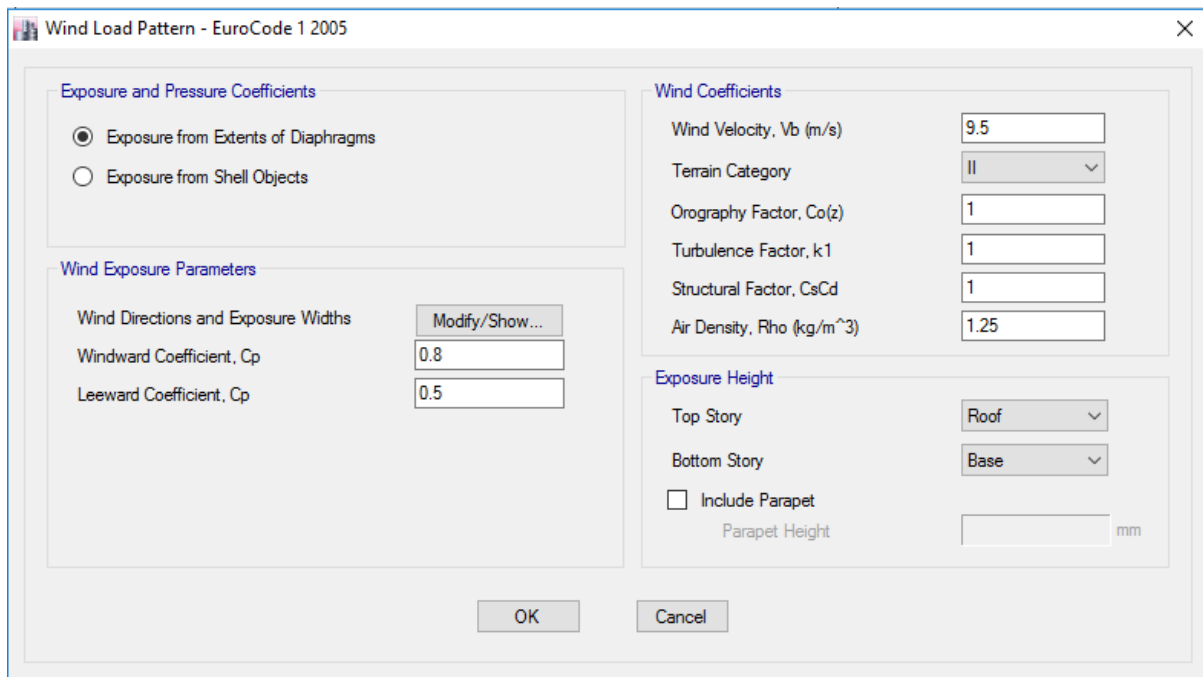


Figure 3.31: Wind load pattern definition

Response spectrum analysis is a method always used for performing seismic analysis of structures as it computes the maximum response values that is in each mode of a structure from a response curve and then combines these responses using modal superposition. Since a building response is not made up of a single mode but the response of many modes the response from many modes must be combined. Combining the modes takes statistical

approaches like Square Root Sum of the Square SRSS Complete Quadratic Combination CQC Absolute sum General Modal Combination GMC.

Eurocode8 EN 1998-1:2005 Clause 3.2.2.2 gives the elastic response spectrum $S_e T$ of the horizontal components of the seismic action as follows

$$0 \leq T \leq T_B : S_e(T) = a_g \cdot S \cdot \left[1 + \frac{T}{T_B} \eta^{2.5} - 1 \right] \quad 3.22$$

$$T_B \leq T \leq T_C : S_e(T) = a_g S \eta^{2.5} \quad 3.23$$

$$T_C \leq T \leq T_D : S_e(T) = a_g S \eta^{2.5} \left[\frac{T_C}{T} \right] \quad 3.24$$

$$T_D \leq T \leq 4s : S_e(T) = a_g S \eta^{2.5} \left[\frac{T_C T_D}{T^2} \right] \quad 3.25$$

Where

$S_e(T)$ is the elastic response spectrum;

T is the vibration period of a linear single-degree-of-freedom system;

a_g is the design ground acceleration on type A ground;

T_B is the lower limit of the period of the constant spectral acceleration branch;

T_C is the upper limit of the period of the constant spectral acceleration branch;

T_D is the value defining the beginning of the constant displacement response range of the spectrum;

S is the soil factor;

η is the damping correction factor with a reference value of $\eta = 1$ for 5% viscous damping.

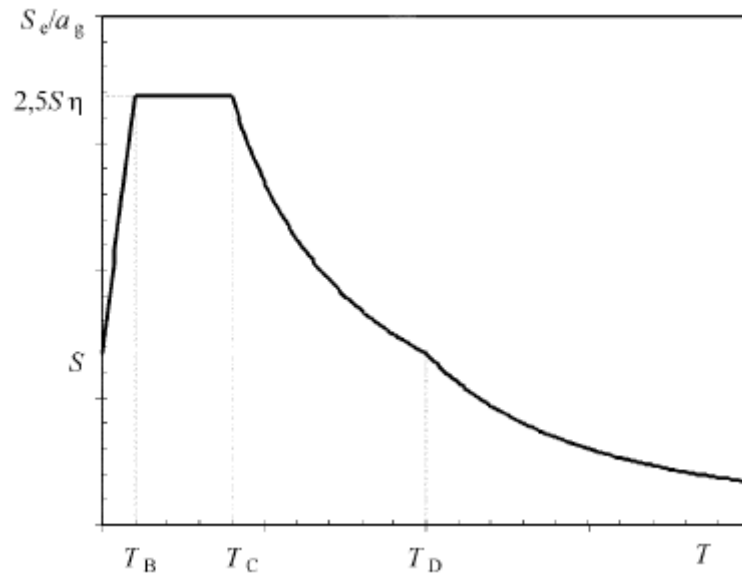


Figure 3.32: Shape of the elastic response spectrum

Figure 3.32 shows the shape of the elastic response spectrum as given by Eurocode. However the basic shape of the horizontal elastic response spectra normalized that is recommended and used in this project is presented in Figure 3.33.

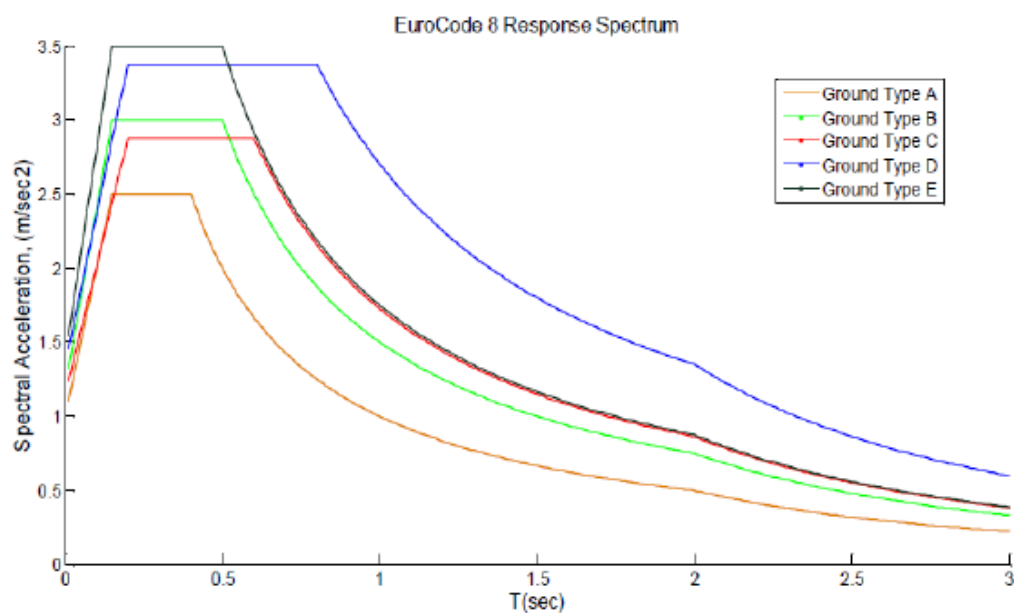


Figure 3.33: Eurocode 8 Design Response Spectrum S_eT Showing a Plot of Spectral Acceleration against Period

The horizontal linear elastic response spectrum was used in this project and the ground type B describing a soil with deposits of very dense sand gravel or very stiff clay at least several tens of metres in thickness and characterised by a gradual increase of its mechanical properties

with depth. responsespectrum parameters were used which corresponds to the Type I stiff soil which is the soil dominant in Haiti. The values of the periods T_B , T_C and T_D and of the soil factor S describing the shape of the elastic response spectrum is dependent on the ground type. Therefore according to Eurocode 8;

- Behavior Factor q 1.5
- Damping Ratio 0.05
- Ground Acceleration a_g/g 0.5 g
- Ground Type B
- Spectrum Type 1
- S 1.5
- $T_B S$ 0.15
- $T_C S$ 0.5
- $T_D S$ 2

Figure 3.34 shows the interface of the response spectrum parameters definition according to Eurocode standard. Figure 3.35 shows the interface of the load case for response spectrum which describes the defined acceleration in U1 x and U2 y direction.

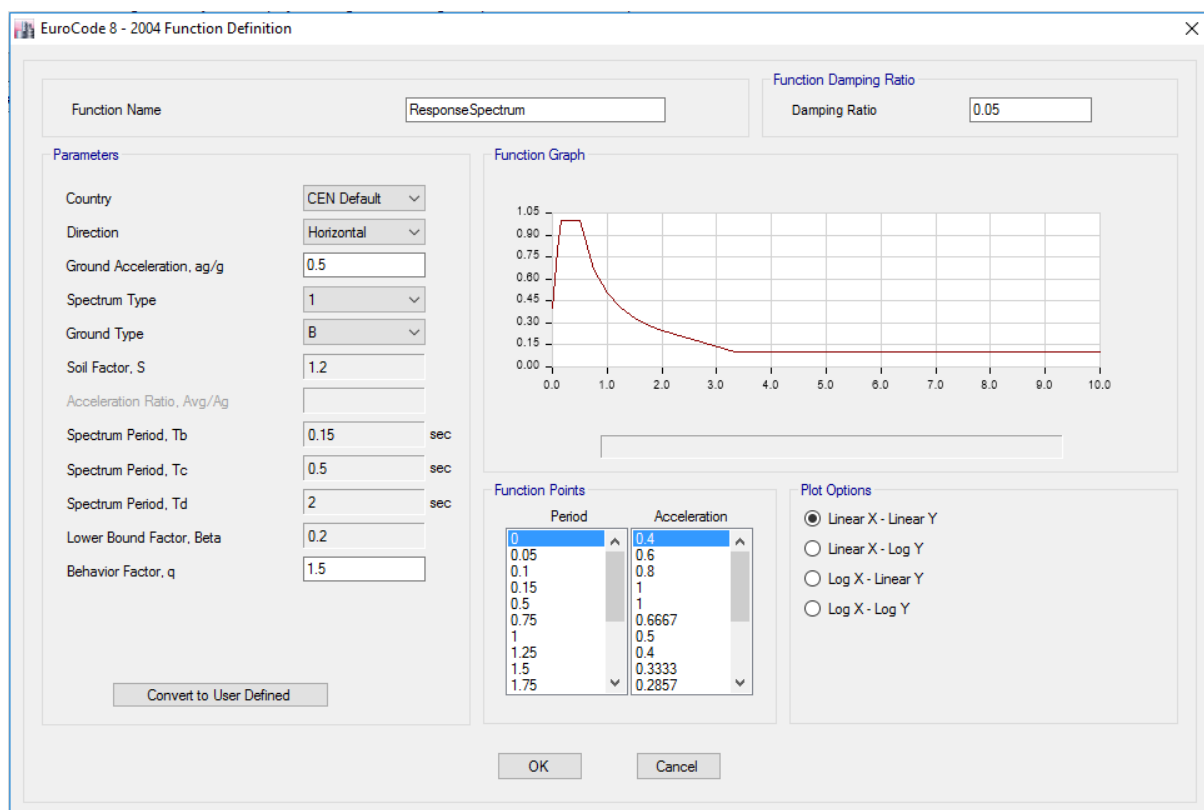


Figure 3.34: Interface of the Response Spectrum Definition According to Eurocode 8

Load Case Data

General

Load Case Name: RS

Load Case Type: Response Spectrum

Exclude Objects in this Group: Not Applicable

Mass Source: Previous (MsSrc1)

Design... Notes...

Loads Applied

Load Type	Load Name	Function	Scale Factor
Acceleration	U1	ResponseSpectrum	9806.65
Acceleration	U2	ResponseSpectrum	9806.65

Add Delete

☐ Advanced

Other Parameters

Modal Load Case: Modal

Modal Combination Method: CQC

☐ Include Rigid Response

Rigid Frequency, f1:

Rigid Frequency, f2:

Periodic + Rigid Type:

Earthquake Duration, td:

Directional Combination Type: SRSS

Absolute Directional Combination Scale Factor:

Modal Damping: Constant at 0.05

Diaphragm Eccentricity: 0 for All Diaphragms

Modify/Show... Modify/Show...

OK Cancel

Figure 3.35: Interface for the Load-case Definition for the Response Spectrum

3.2.7 Time History Analysis

There are many recorded historical earthquake records but in this research five different earthquake records were used to analyse the structure. The descriptions of the earthquake are given in 3.2.7.1 to 3.2.7.5 and the plots of the accelerogram are also shown.

3.2.7.1 El Centro Earthquake

The May 1940 El Centro earthquake Imperial Valley earthquake which was the first major earthquake to be recorded by a strong-motion seismograph located next to a fault rupture was used in this project

Date	May 18 1940
Origin Time	21:35 PST
Magnitude	6.9 Mw
Epicentre	32.733°N 115.5°W
Areas Affected	United States Mexico
Maximum Intensity	X

To understand ground motions feature in near-source regions Kikuchi et al. 2000 used a number of El Centro Array and discovered that the PGA of the vertical motions of El Centro Array increases significantly in accordance with approaching the fault. A PGA of 1490 Gal was recorded in Array Number 6 and this value was tremendously greater than the rest and thus it was used in this project. Figure 3.36 and 3.37 shows the Array06 El Centro earthquaketime history plot in Imperial Valley for x 1 and y 2 direction respectively.

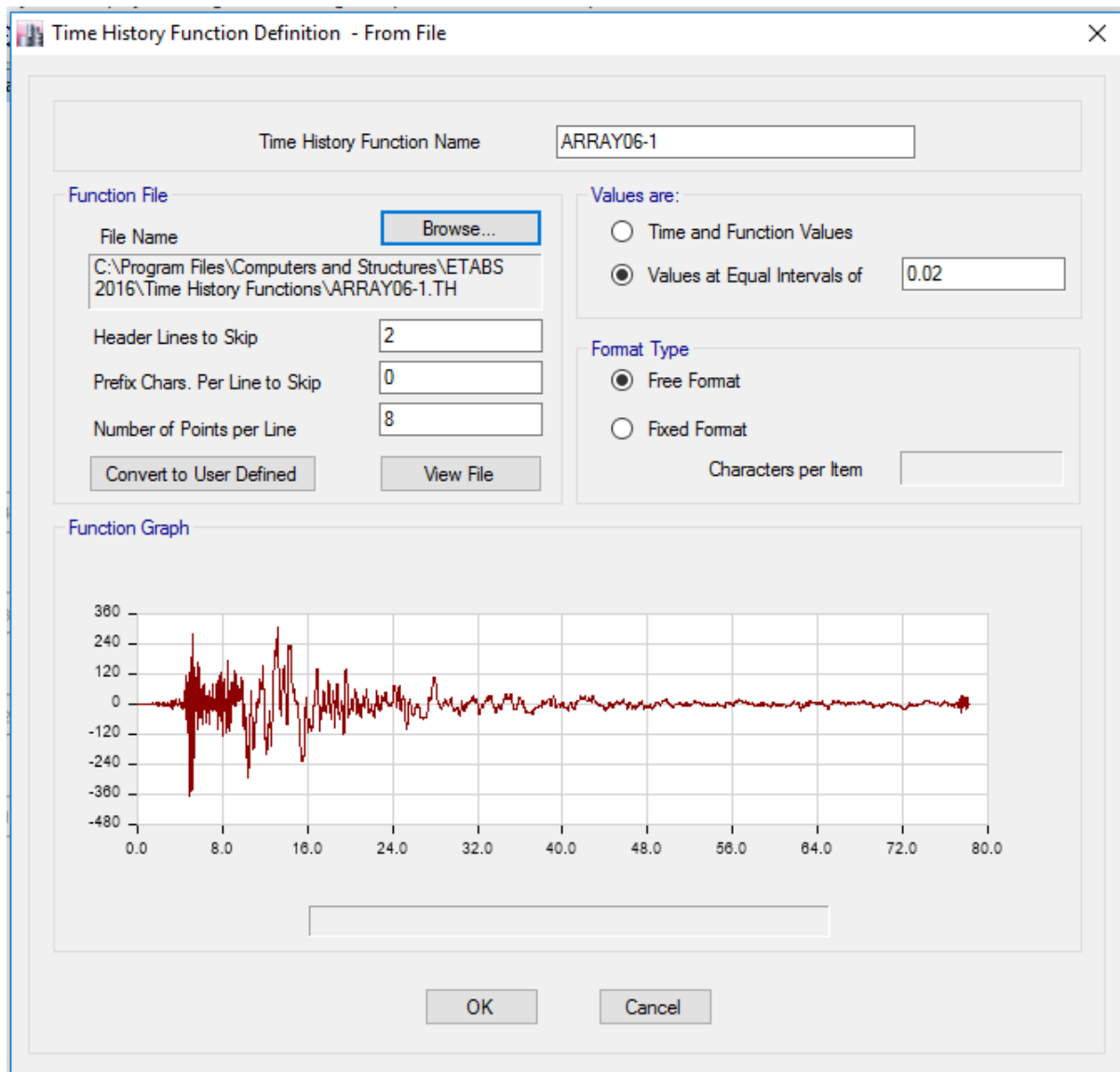


Figure 3.36: Time History Function for Array06 in x Direction

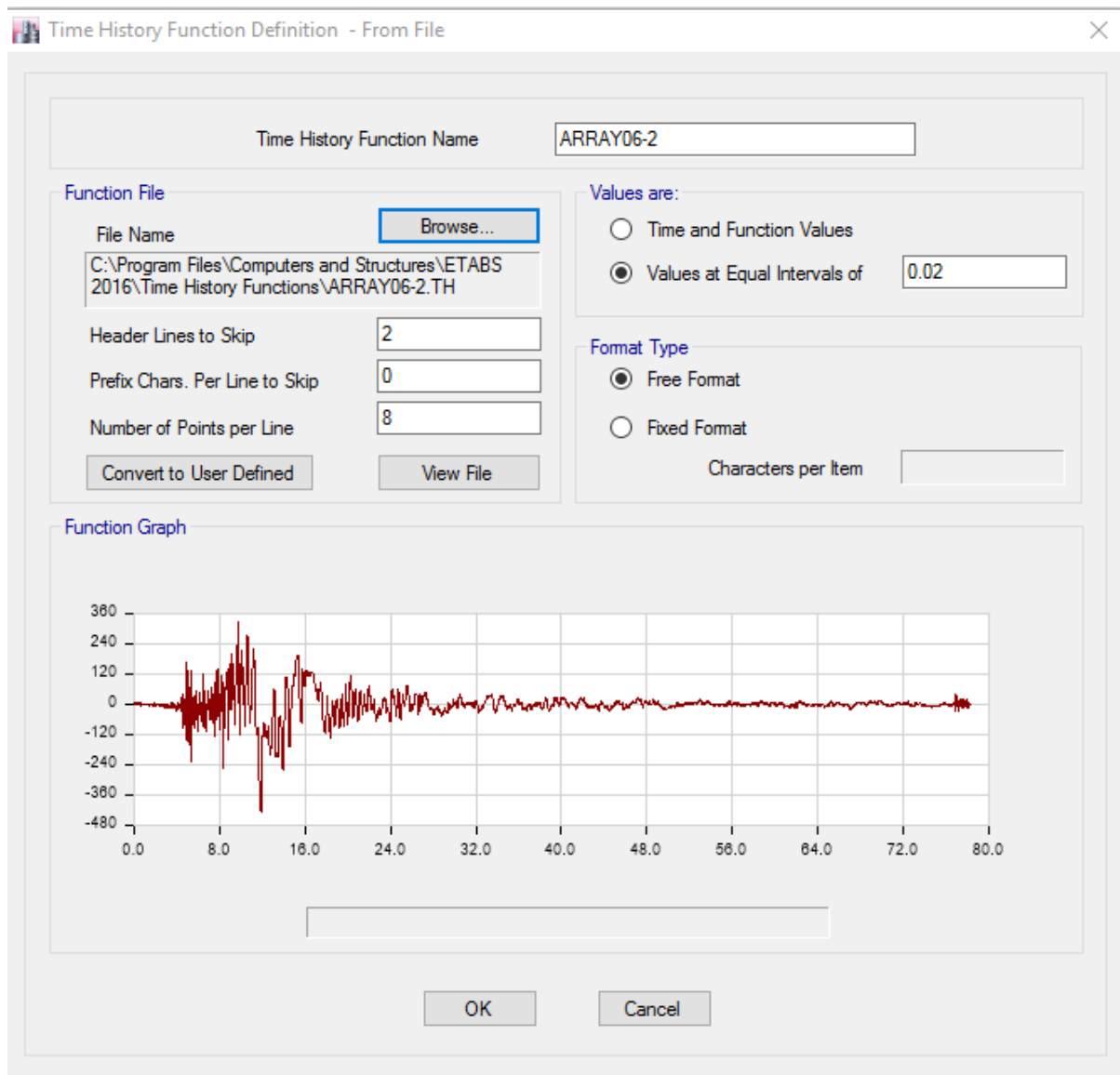


Figure 3.37: Time History Function for Array06 in y Direction

The Array06 functions in the x and y direction were combined in a load case as shown in Figure 3.38 and used to analyse the building. The number of output time steps was set to 120 and the output time step size set as 0.5. This was done to set the total time of the earthquake occurrence/span at 60 seconds product of 120 and 0.5.

Load Case Data

General

Load Case Name: Array06 TH Records [Design...]

Load Case Type/Subtype: Time History / Nonlinear Direct Integration [Notes...]

Exclude Objects in this Group: Not Applicable

Mass Source: MsSrc1

Initial Conditions

☒ Zero Initial Conditions - Start from Unstressed State

☐ Continue from State at End of Nonlinear Case (Loads at End of Case ARE Included)

Nonlinear Case:

Loads Applied

Load Type	Load Name	Function	Scale Factor
Acceleration	U1	ARRAY06-1	1
Acceleration	U2	ARRAY06-2	1

[Add] [Delete] [Advanced]

Other Parameters

Geometric Nonlinearity Option: P-Delta

Number of Output Time Steps: 120

Output Time Step Size: 0.5 sec

Proportional Damping: Mass: 0; Stiff: 0 [Modify/Show...]

Time Integration: Hilber-Hughes-Taylor [Modify/Show...]

Nonlinear Parameters: Default [Modify/Show...]

[OK] [Cancel]

Figure 3.38: Load Case Data Definition for the Array06 earthquake of El Centro

3.2.7.2 Petrolia Earthquake

The 1992 Petrolia earthquake or 1992 Cape Mendocino earthquake which occurred on April 25 1992 along the Lost Coast of Northern California was used.

Date	April 25 1992
Magnitude	7.2 Mw
Epicentre	40.33°N 124.23°W

Areas Affected North Coast California United States

Maximum Intensity IX

Figure 3.39 and 3.40 shows the time history plot for the 1992 Cape Mendocino earthquake for x 1 and y 2 direction respectively.

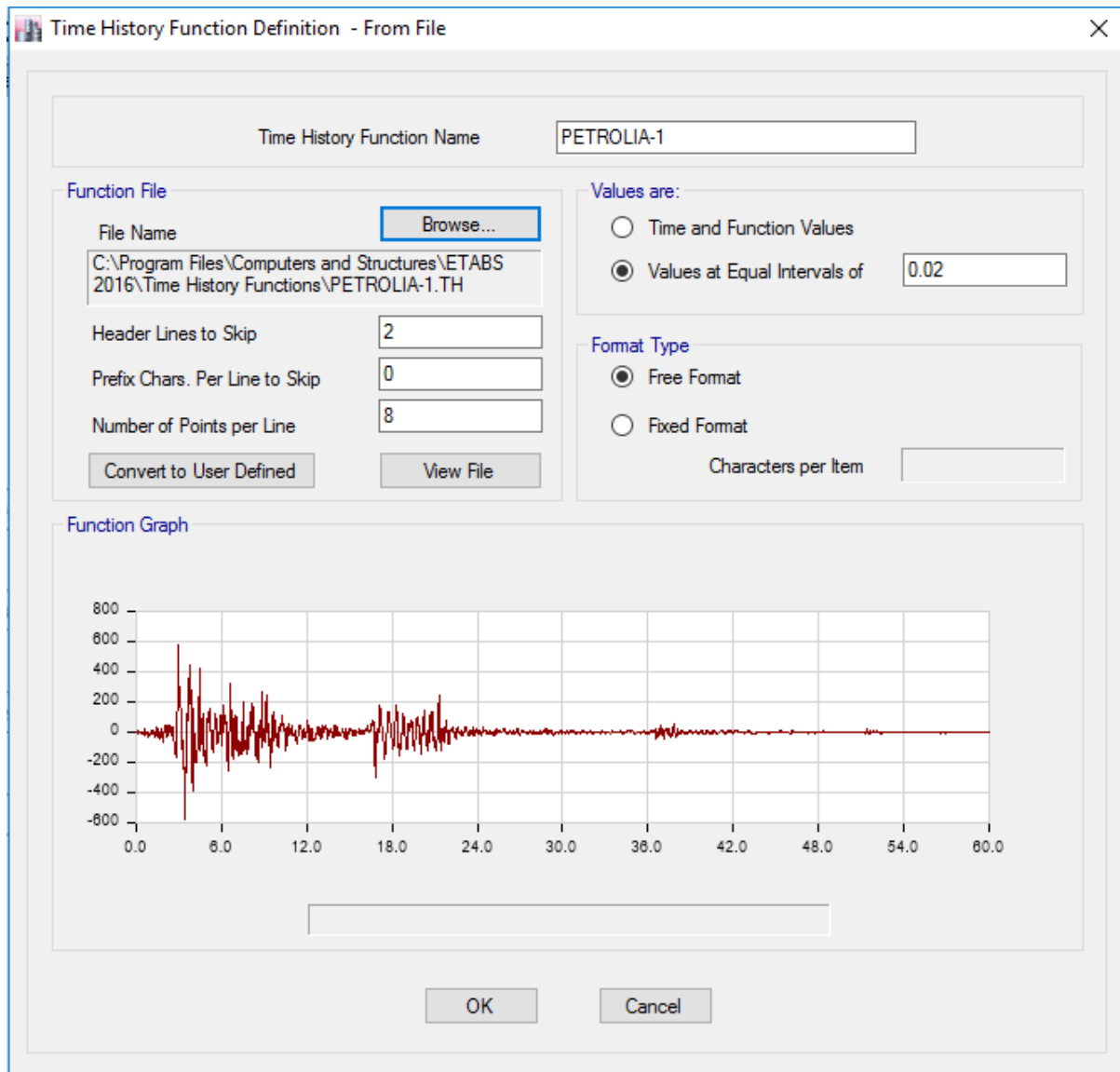


Figure 3.39: Time History Function for Petrolia Earthquake in x Direction

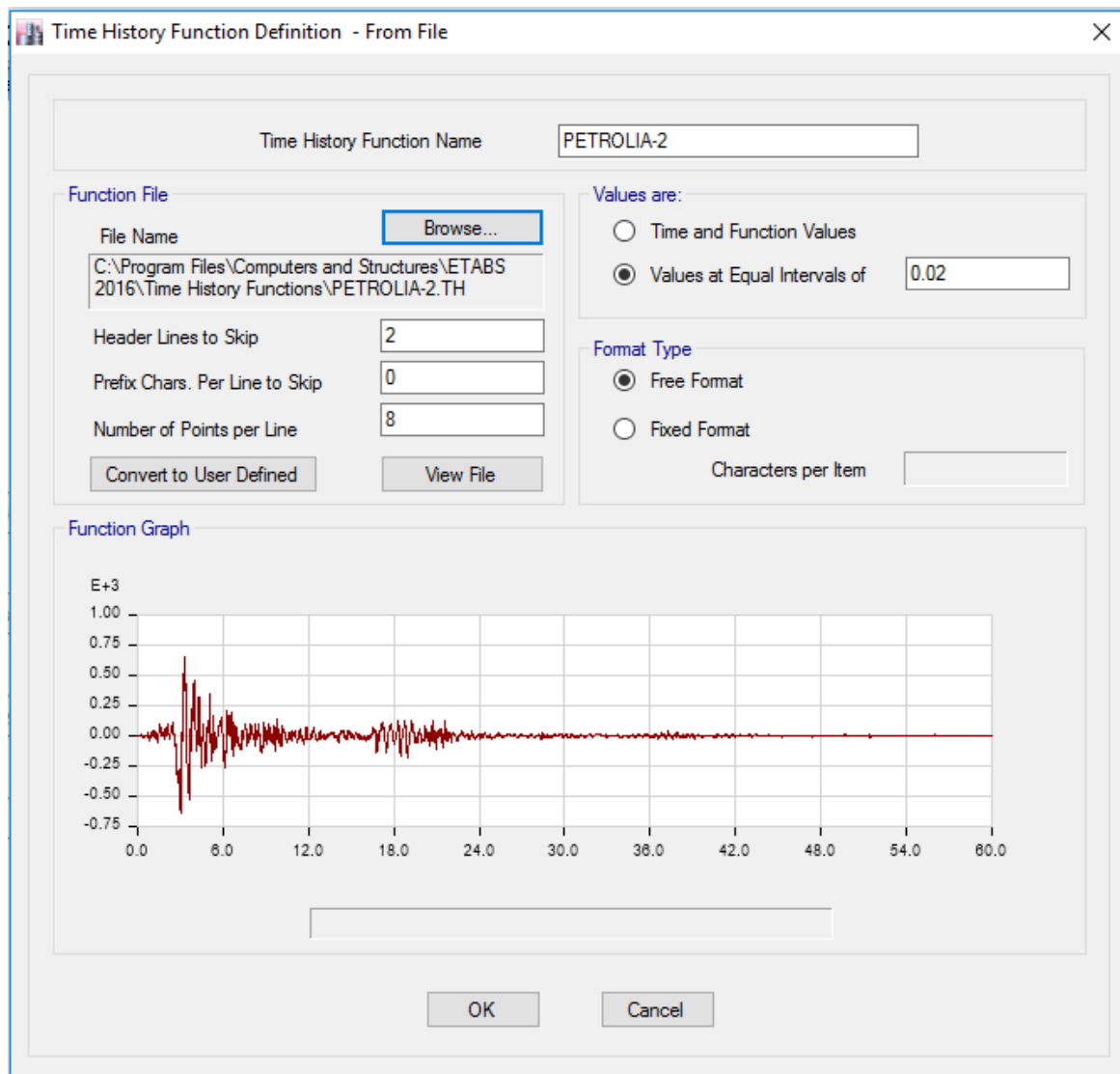


Figure 3.40: Time History Function for Petrolia Earthquake in y Direction

The Petrolia earthquake functions in both the x and y direction were also combined in a load case and used for the time history analysis of the building. The number of output time steps was also set to 120 and the output time step size set as 0.5. This was done to set the total time of the earthquake occurrence/span at 60 seconds product of 120 and 0.5.

3.2.7.3 Lacc_North Earthquake

The 1994 Northridge earthquake which occurred on January 17 was the highest instrumentally recorded ever in an urban area in North America and it was also used.

Date	January 17 1994
Magnitude	6.7 Mw

Epicentre 34.213°N 118.537°W

Areas affected Los Angeles Area Southern California United States

Maximum intensity IX

The interface of ETABS for the 1994 Northridge earthquake definition are shown in Figure 3.41 and 3.42 for x and y direction respectively.

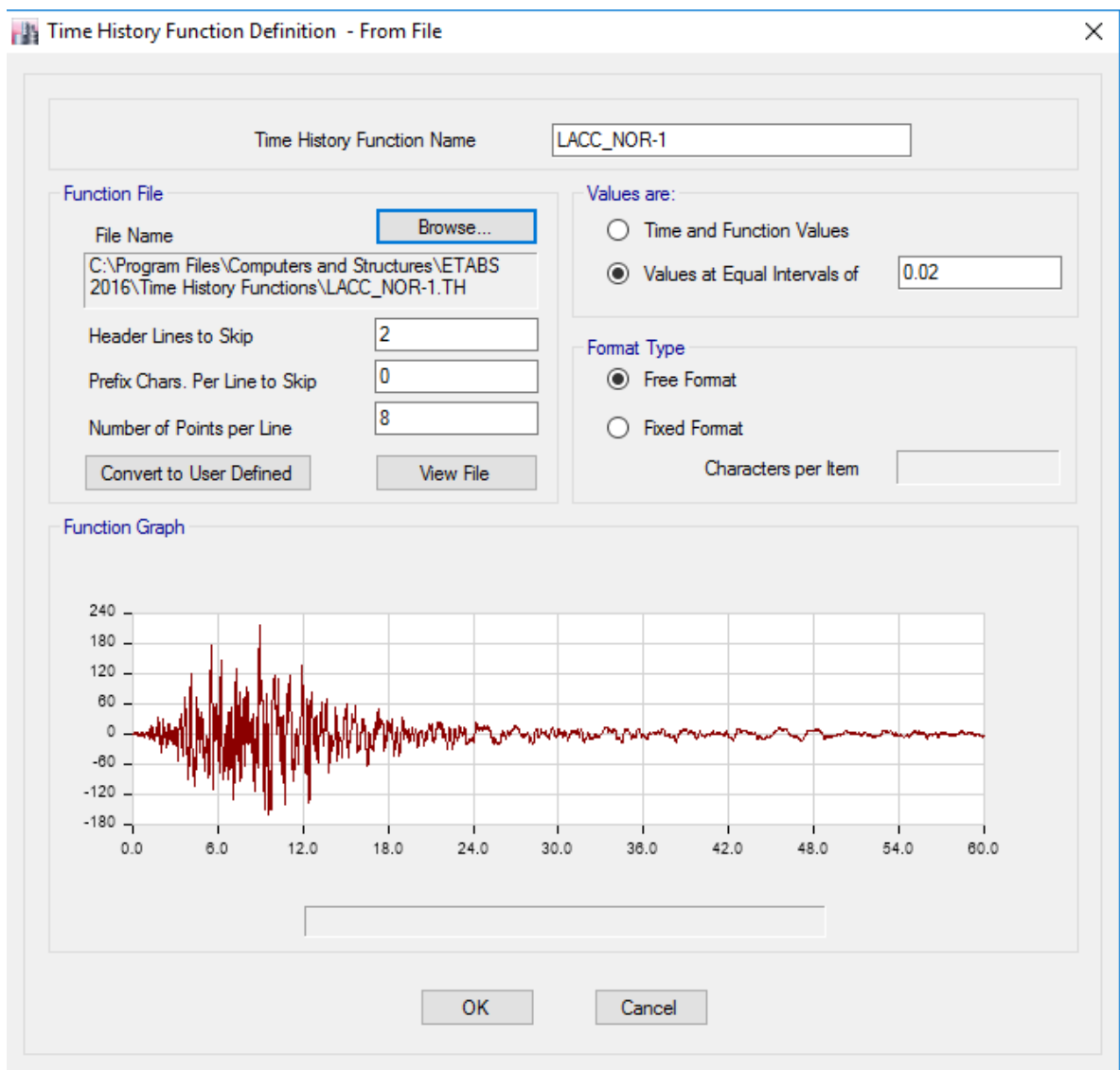


Figure 3.41: Time History Function for LACC_NOR Earthquake in x Direction

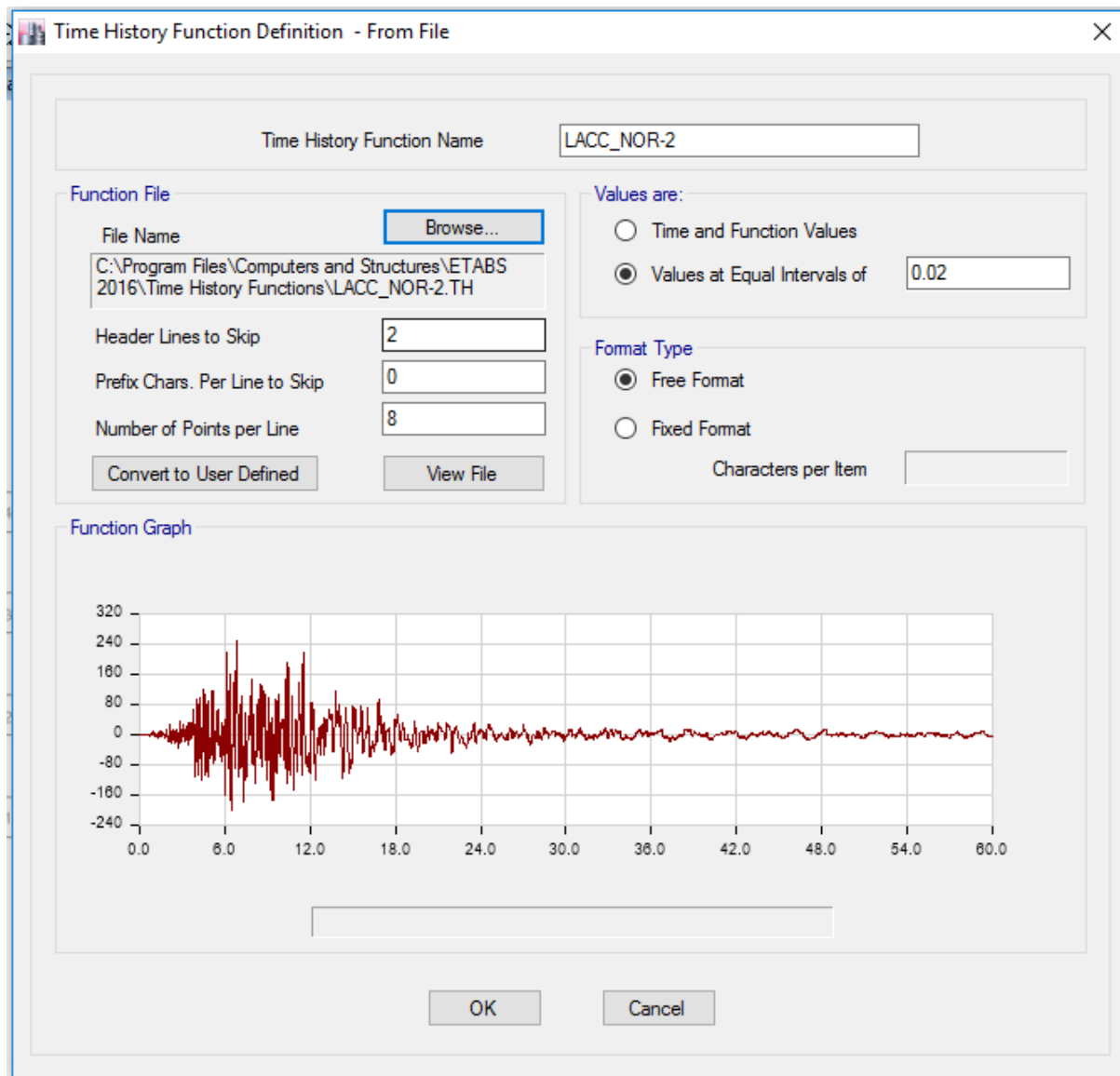


Figure 3.42: Time History Function for LACC_NOR Earthquake in y Direction

The x and y direction functions of the LACC_NORTH earthquake were then combined in a load case. The output step was also set to span for 60 seconds when used for the time history analysis.

3.2.7.4 Lucerne Earthquake

The 1992 Landers earthquake occurred on June 28 with an epicentre near the town of Landers California.

Date	June 28 1992
Magnitude	7.3 Mw
Epicentre	34.217°N 116.433°

Areas affected Southern California United States

Maximum intensity IX

The interface of the CSI ETABS for the 1992 Landers earthquake definition are shown in Figure 3.43 and 3.44 for x and y direction respectively.

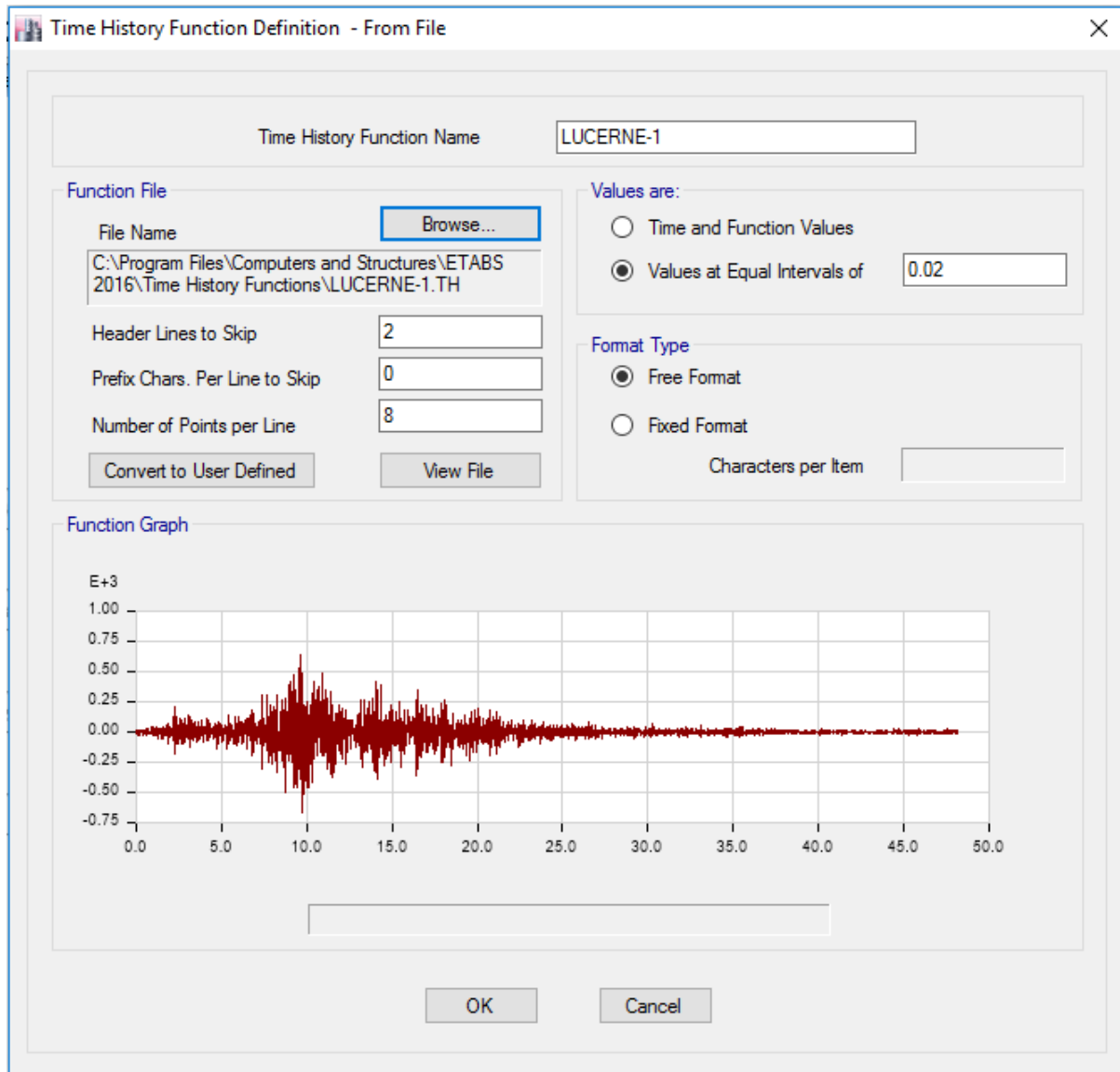


Figure 3.43: Time History Function for Lucerne Earthquake in x Direction

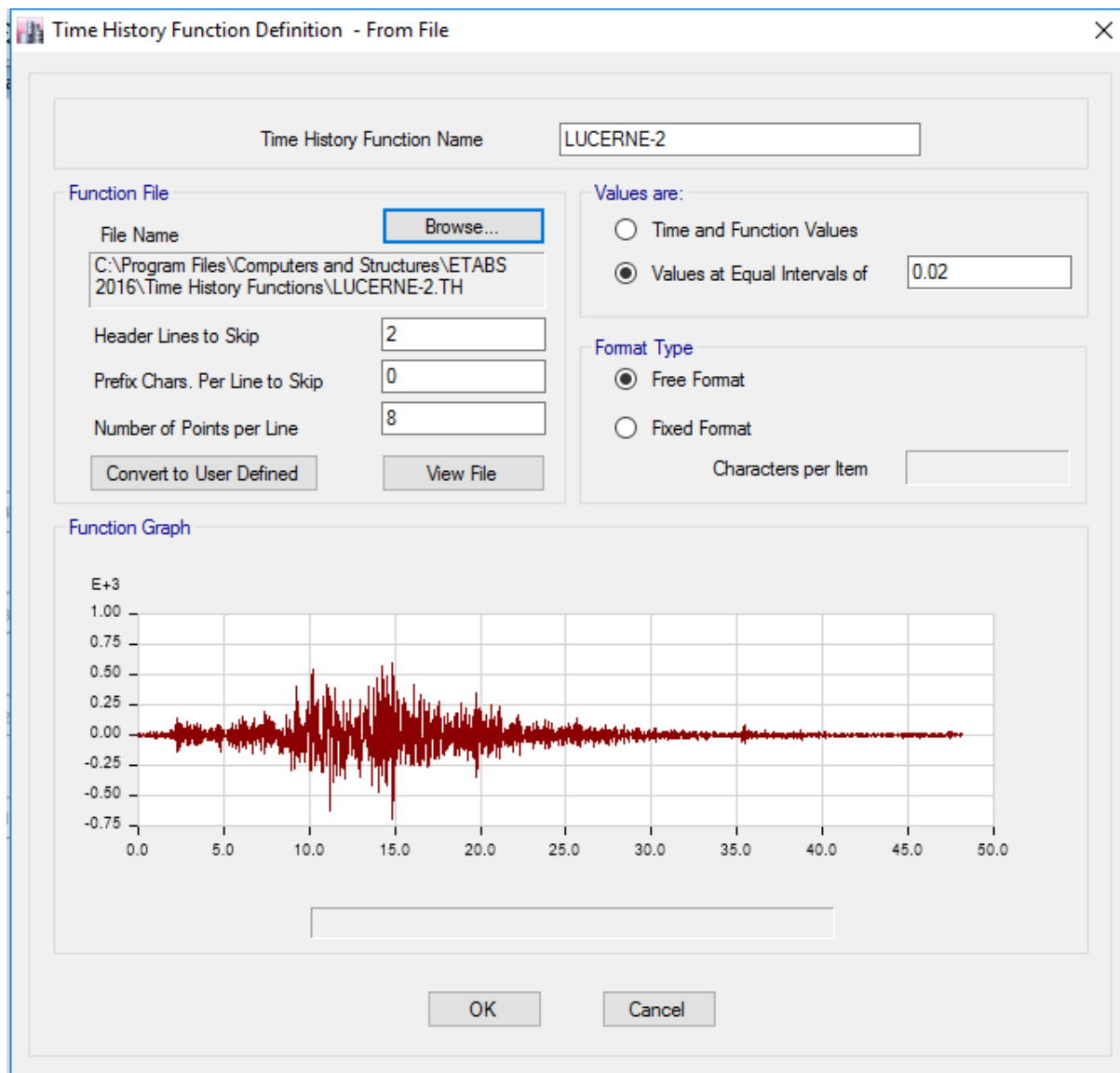


Figure 3.44: Time History Function for Lucerne Earthquake in y Direction

The functions for the x and y direction of the Lucerne earthquake were also combined in a load-case. However the time history functions were set to span for 50 seconds. This was achieved by setting the number of output time steps to 100 and the output time step size to 0.5.

3.2.7.5 Slymarff Earthquake

The Slymar earthquake also known as the 1971 San Fernando earthquake occurred in the early dawn of February 9 in the foothills of the San Gabriel Mountains in southern California was used.

Date February 9 1971

Magnitude	6.6 Mw
Epicentre	34.41°N 118.40°W
Areas affected	Greater Los Angeles Area Southern California United States
Maximum intensity	XI
Casualties	58–65 dead and 200–2000 injured

Figure 3.45 and 3.46 shows the time history plot for the 1971 San Fernando earthquake for x and y direction respectively.

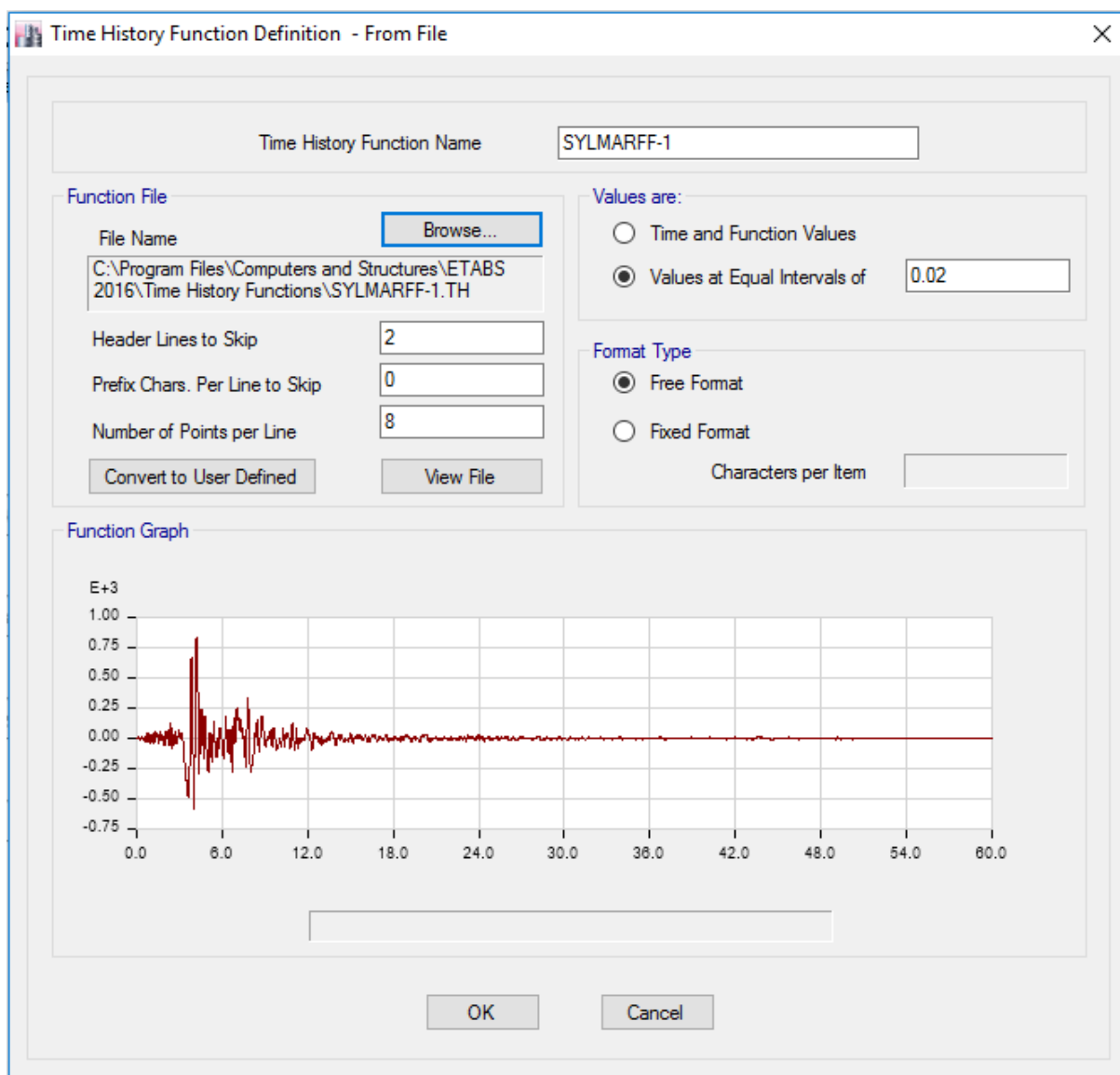


Figure 3.45: Time History Function for Sylmarff Earthquake in x Direction

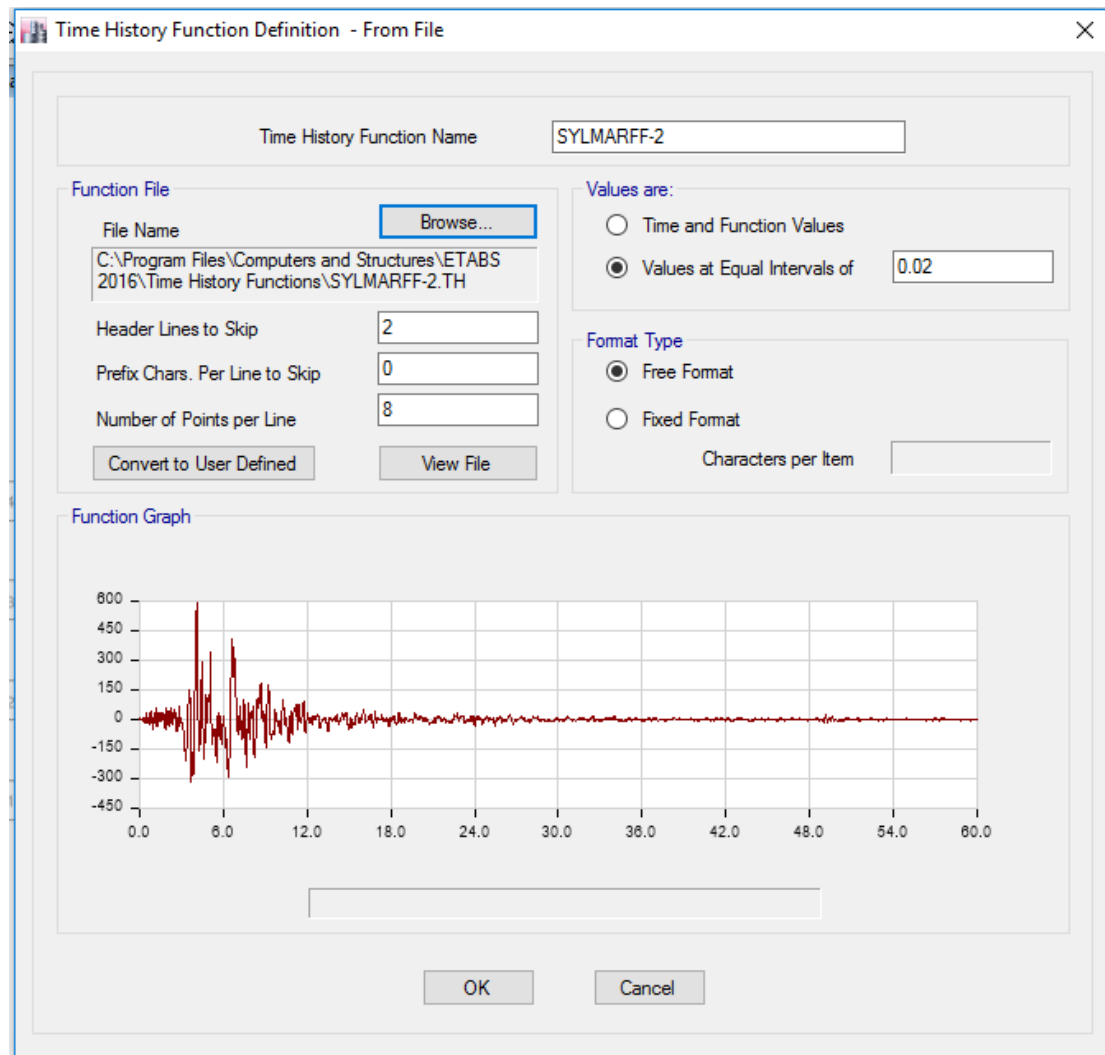


Figure 3.46: Time History Function for Slymarff Earthquake in y Direction

The x and y direction functions of the Slymarff earthquake were also further combined in a load case and also set to span for 60 seconds when used for the time history analysis.

3.3 Building Response to Seismic Loadings

After subjecting the building to the various historical earthquakes its response to displacement drift stiffness shear and overturning moments were recorded and compared.

3.3.1 Safety Verification

The damage limitation requirement was verified in terms of the inter-storey drift d_r EN 1998-1: 20004 /4.4.3.2 using equation

$$d_r \cdot v \leq \alpha \cdot h \Rightarrow \frac{d_r}{h} \leq \frac{\alpha}{v} \quad 3.26$$

Eurocode 8 has made provisions for the safety verification of seismic resistance building in EN 1998-1/4.4.3.2 limitation of interstorey drift and it is used to test verify the safety limit of the study building.

The most severe drift limit $d_r v \leq 0.005h$ for building having non-structural elements of brittle materials attached to the structure is used to verify the safety of the building.

Storey drift d_r is defined and computed as the difference of the average lateral displacements d_s at the top and bottom of the storey EN 1998-1: 2004 /4.4.2.22. In EN 1998 it is not defined how the “average” value should be calculated. Note storey drifts have to be determined for each vibration mode and combined according to a combination rule. For example Complete Quadratic Combination method CQC reduces errors in modal combination in samples studied Clough and Warburton 1981. h is the storey height. v is the reduction factor which takes into account the lower return period of the seismic action associated with the damage limitation requirement. It depends on the important class of the building.

The study building is classified as importance class III EN 1998-1/Table 4.3 and the corresponding reduction factor v is 0.4 EN 1998-1/4.4.3.22. α is inter-storey drift ratio limit is a factor which takes into account the type of the non-structural elements and their arrangements into the structure and their respective values are; 0.005 0.0075 and 0.01 EN 1998-1 equations 4.31 4.32 and 4.33 this is shown in equation 3.27 to 3.39.

a) for buildings having non-structural elements of brittle materials attached to the structure: $d_r v \leq 0.005h$; 3.27

b) for buildings having ductile non-structural elements: $d_r v \leq 0.0075h$; 3.28

c) for buildings having non-structural elements fixed in a way so as not to interfere with structural deformations or without non-structural elements:

$$d_r v \leq 0.010h \quad \text{3.29}$$

3.4 Comparison between the Responses of CSC with CFB

The CSC frame model was compared to a reinforced concrete frame building CFB of the same plan and configuration. The time history model of the Petrolia earthquake was used for the analysis and their response in terms of displacement and drifts were recorded.

CHAPTER FOUR

4.0 RESULTS AND DISCUSSION

After subjecting the building to response spectrum and time history analysis the response of the building to the seismic load were recorded in this chapter. The response of the cold-formed steel concrete CSC frame with plastered strawbale PSB panel were studied under the influence of seismic ground excitations and recorded. Finite element with the aid of ETABS the building module of SAP2000 engineering software was used to perform the analysis.

4.1 Seismic Modelling and Analysis of the CSC Frame with PSB Panels

The total weight of the building was estimated using material list by section criteria. Table 4.1 shows the material list by section. The weight of the cold-formed concrete composite column and beam the shear wall the cold-formed concrete composite slab and the plastered strawbale wall were added together to form a total of 16067.11 kN.

Table 4.1: Material list by section

Section	Element Type	Total Weight kN
Coldformed Concrete Composite Column 450x225mm	Column	323.2511
Coldformed Concrete Composite Column 225x450mm	Column	692.681
Cold-formed Concrete Composite Beam 450x225mm	Beam	1827.7473
Cold-formed Concrete Composite Slab	Floor and Metal Deck	2334.966
Plastered Strawbale wall	Wall	10156.7251
Shear Wall	Wall	731.737
Total		16067.1078 kN

The building was taken as a single unit and its response to deflection and drifts were recorded. Modal analysis was also done to determine the periods of the first six modes of the building and these modes are shown in Appendix B.

4.2 Building Response to Seismic Loadings

4.2.1 Response Spectrum Analysis

The storey response plot for maximum displacement drift stiffness shear pseudo-spectral acceleration per period for the base and all the storeys of the buildings are studied for the building. These values are tabulated in Appendix C. Table 4.2 shows the base reactions of the CSC Frame with PSB wall during seismic loading. These reactions include the force and moment in x y and z directions. FX FY and FZ are the force in the x y and z-direction respectively while MX MY and MZ are the moments at the base in the x y and z directions.

Table 4.2: Base reaction

Load Case/Combo	FX	FY	FZ	MX	MY	MZ
	kN	kN	kN	kN-m	kN-m	kN-m
Dead	0	0	16167.1954	78698.5188	- 176577.5618	0
Live	- 0.000009691	0	1926534.375	9664780.781	-19417861	0.0001
RS Max	10967.9076	9193.1722	129.2271	92852.9554	109205.0639	125228.2774

The reactions at each joint of the column base were recorded according to their joint labels under the action of the response spectrum analysis. The joint labels were shown on the plan view in Figure 4.1 and its reactions were tabulated in Table 4.3.

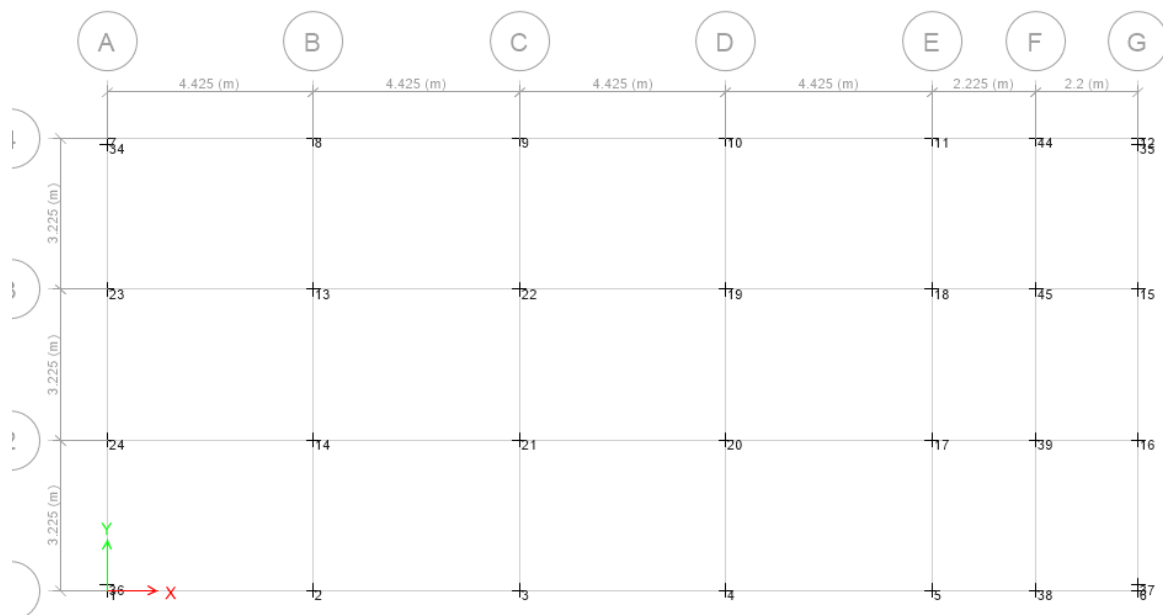


Figure 4.1: Joint label at the base

Table 4.3: Joint reactions at the base

Story	Joint Label	Load Case/Combo	FX	FY	FZ
			kN	kN	kN
Base	1	RS Max	267.1752	87.5783	15928.8715
Base	2	RS Max	74.2534	167.2654	1399.932
Base	3	RS Max	77.4341	79.0221	989.7875
Base	4	RS Max	69.3455	54.0763	730.1806
Base	5	RS Max	4019.0342	2722.1707	14401.6592
Base	6	RS Max	129.8927	13.834	3032.5856
Base	7	RS Max	285.974	87.593	15904.1062
Base	8	RS Max	286.9386	163.9362	1357.0175
Base	9	RS Max	264.9904	81.5565	976.2002
Base	10	RS Max	269.4496	54.4364	665.8632
Base	11	RS Max	245.283	11.1418	512.4409
Base	12	RS Max	124.3234	15.2231	2910.0655
Base	13	RS Max	102.3118	350.9095	144.7814
Base	14	RS Max	86.9305	350.4045	131.1127
Base	15	RS Max	69.0996	85.7523	673.3811
Base	16	RS Max	35.467	86.2752	1350.6753
Base	17	RS Max	132.3798	4435.2976	13830.171
Base	18	RS Max	87.5288	35.992	783.4161
Base	19	RS Max	113.7601	132.1392	209.6431
Base	20	RS Max	81.4618	119.7	364.5789
Base	21	RS Max	69.1007	236.2779	147.1684
Base	22	RS Max	116.4691	231.9925	142.6629
Base	23	RS Max	53.1855	436.6814	733.3938
Base	24	RS Max	72.066	437.0072	449.7663
Base	34	RS Max	155.8226	1017.5511	14383.4787
Base	35	RS Max	46.5008	196.3282	3141.1675
Base	36	RS Max	240.0566	1013.9582	13831.2375
Base	37	RS Max	126.1276	180.6167	2965.1707
Base	38	RS Max	5460.4712	1835.4521	20928.3008
Base	39	RS Max	109.7669	2023.59	6383.2631
Base	44	RS Max	42.8804	2.7216	1.7152
Base	45	RS Max	31.0161	2.448	0.956

Figure 4.2 shows the maximum storey displacement of the 4-storey cold-formed steel concrete with plastered strawbale wall under the action of the response spectrum analysis in both the global x and y-directions. The building was displaced in both global x and y directions with the maximum displacement of 127.85 mm this occurred at the top of the building in the global y-direction. The building displacement in the x-direction also occurred at the top of the

building with a value of 83.60mm. A building's storey displacement is a measure of its total displacement from the ground floor to the top of the building roof.

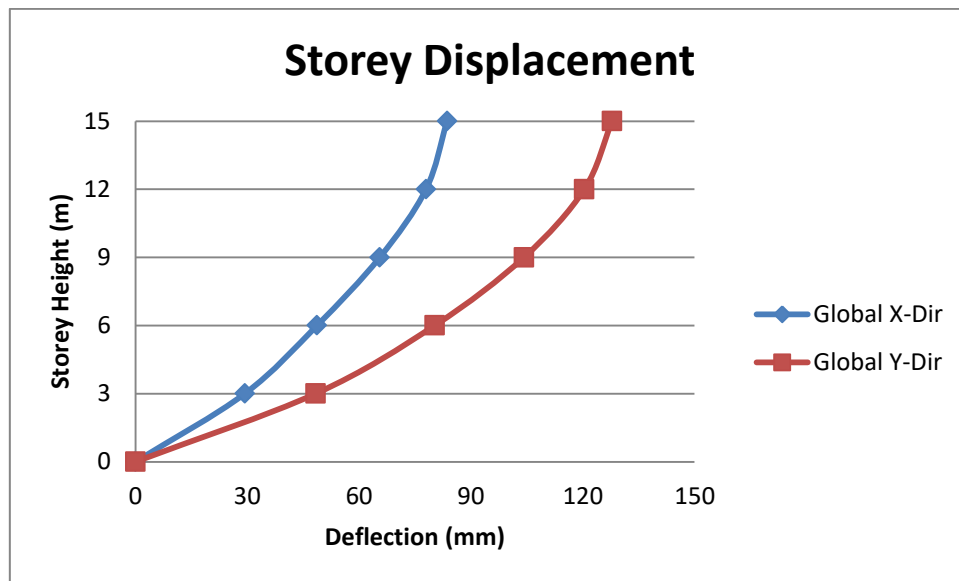


Figure 4.2: Storey response displacement of the CSC Frame with PSB wall

The drift of the building which describes the displacement of the building from storey to storey is shown in Figure 4.3. For each storey the maximum drift occurs at the 1st storey in the global x and y-directions. The building maximum drift has a value of 0.016 in the global y-direction; the storey drift in the x-direction has a value of 0.0098. The maximum drift at storey 1 gradually reduces along the storey height with different values in both directions. The drift values of the building in both the x and y directions are different for each storey levels but the same at the top of the building in both directions with a value of 0.0027 at the top.

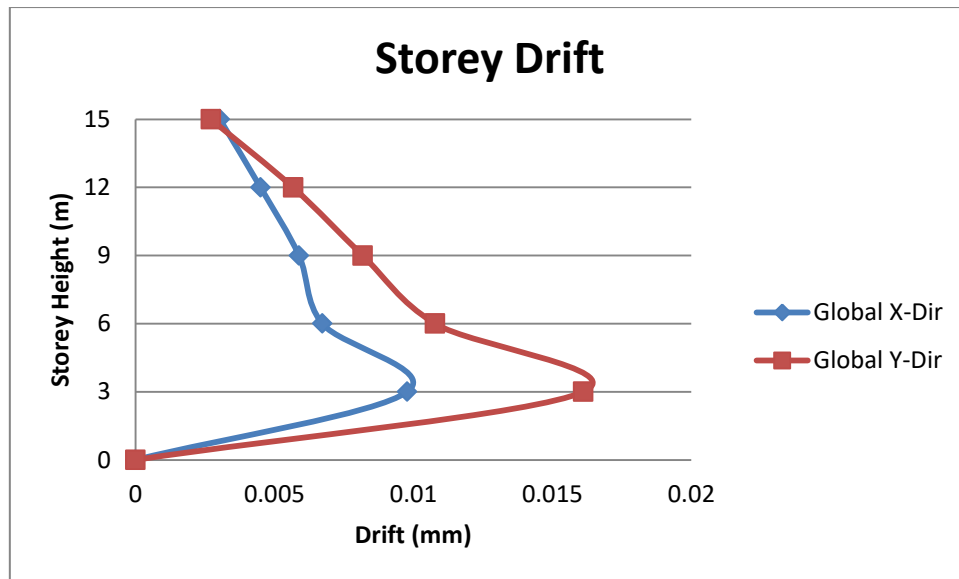


Figure 4.3: Storey response drifts of the CSC Frame with PSB wall

The storey shears of the building in both global x and y-directions is the seismic forces lateral loads applied at each storey and results are shown in Figure 4.4. The structure has a maximum shear of 10967.91 kN at the 1st storey in the x direction and 9193.17 kN in the y direction. The storey shear values of the building in both the x and y directions are different for each storey level and it is maximum at storey 1 and gradually reduces as the storey height increases with different values in both directions.

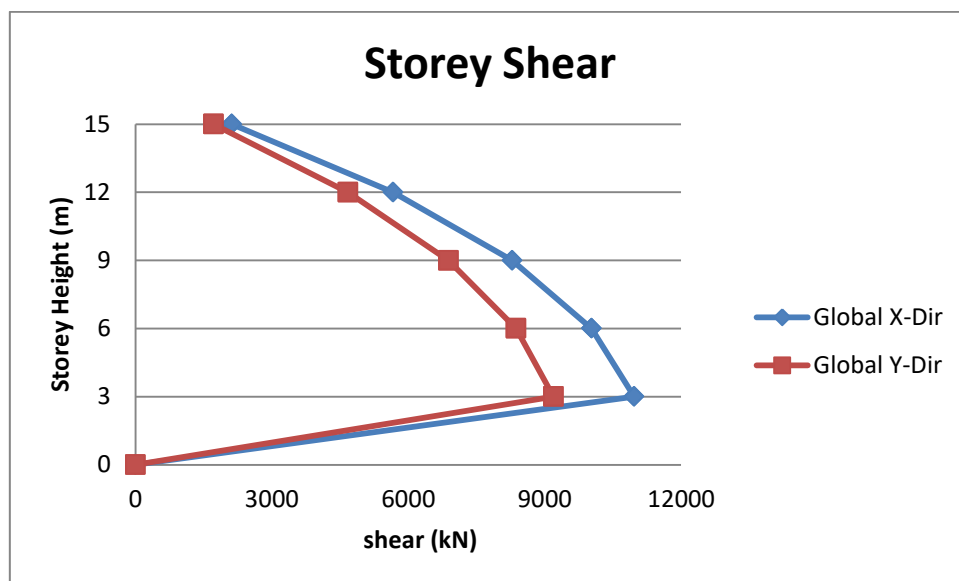


Figure 4.4: Storey response shear of the CSC Frame with PSB wall

The overturning moment of the building decreases as the height of the building increases in both global x and y-directions as shown in Figure 4.5. The maximum overturning moment occurs at the base with a value of 109205 kN-m in the y-direction and also a value of 92852.96 kN-m in the x-direction.

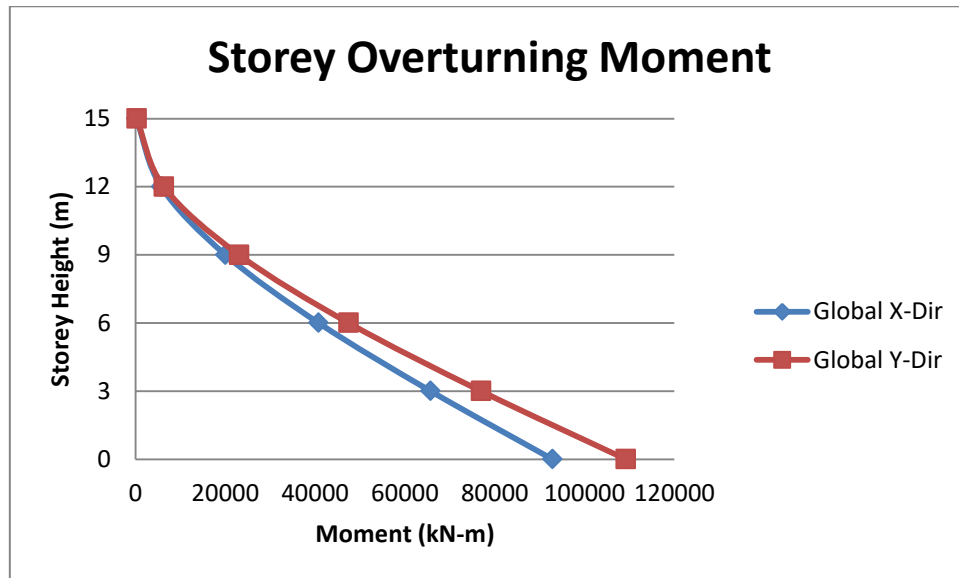


Figure 4.5: Storey response overturning moment of the CSC Frame with PSB wall

The stiffness of the building which depicts the resistance strength of the building is shown in Figure 4.6. The building has a maximum stiffness value of 645117 kN/m on the 2nd storey in the global x-direction. This shows that the building is stiffer in the global x-direction than the global y-direction. And that is why more drift and displacement are experienced in the y-direction.

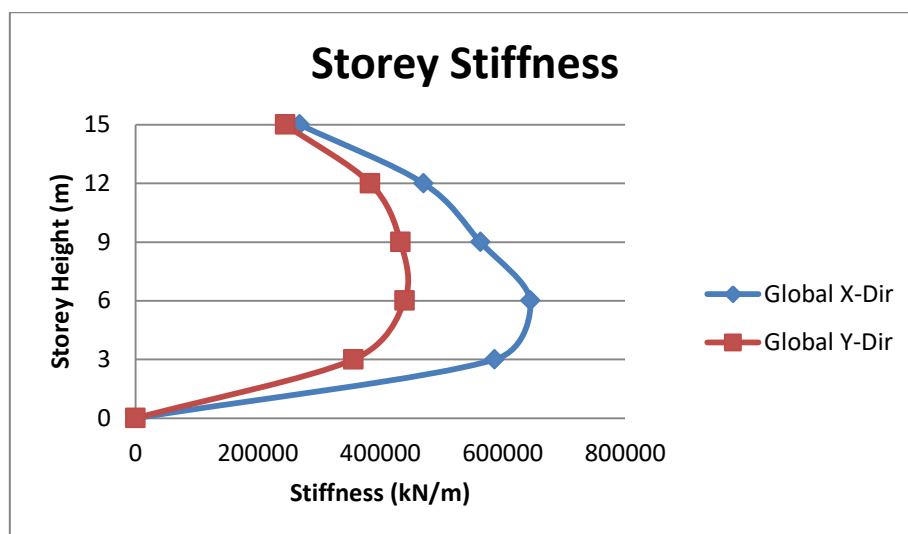


Figure 4.6: Storey response stiffness of the CSC Frame with PSB wall

4.2.2 Time History Analysis

The storey response plot for maximum displacement drift shear and moment for the base and all the storeys of the buildings under the various historical earthquake records are studied and plotted in this section.

4.2.2.1 El Centro Earthquake

Figure 4.7 shows the maximum storey displacement of the building under the action of the El Centro earthquake data in both the x and y directions. The maximum storey displacement of the building is at the roof with a value of 24.57 mm and it occurs at the top of the building in the global y-direction. The building also deflects in the x-direction but with a lesser displacement value of 16.98mm. A building's storey displacement is a measure of its total displacement from the ground floor to the top of the building roof.

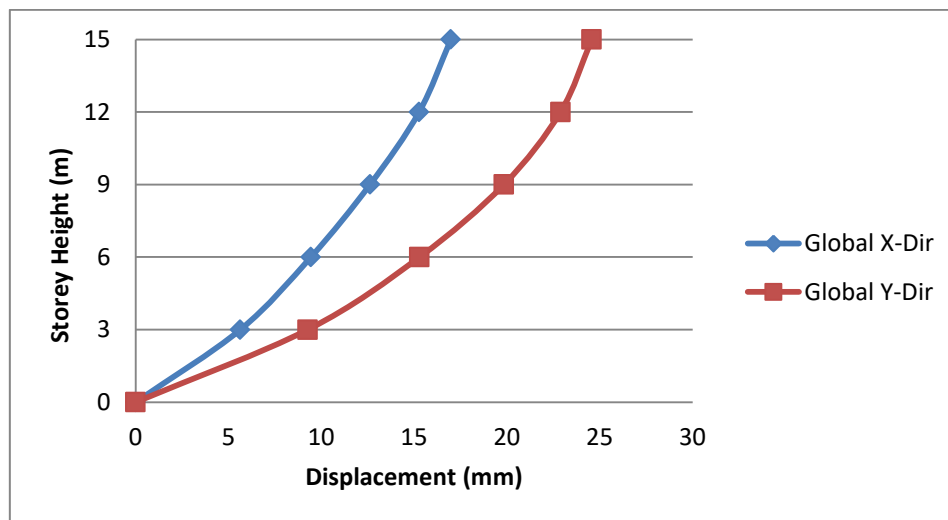


Figure 4.7: Storey response maximum displacement of CSC Frame with PSB wall under the El Centro earthquake

The maximum storey drift of the building describes the displacement of the building from floor to floor under the action of the El Centro earthquake data is shown in Figure 4.8. For each storey the maximum drift occurs at the top for both the x and y directions. The maximum drift of the building is at the 1st storey with a value of 0.0031 in the global y direction and 0.00188 in the global x direction. The drift values of the building in both the x and y directions are different for each storey level but the values are the same at the top of the building in both directions with a value of 0.000705.

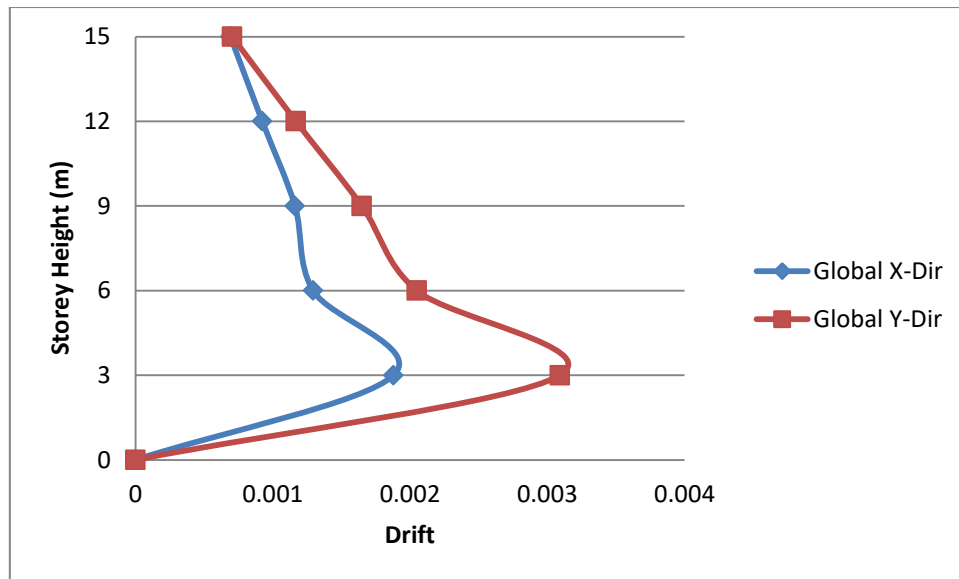


Figure 4.8: Storey response maximum drifts of the CSC Frame with PSB Wall under the El Centro earthquake

The maximum storey shears and the maximum overturning moments of the building in both x and y direction are shown in Figure 4.9 and 4.10 respectively. The maximum shear occurs at storey 2 with a value of 1283.699 kN and the minimum on storey 2 with a value of -1573.94 kN both in the global x-direction. In the global y-direction the maximum and minimum values are 1142.734 kN and -1402.18 kN respectively in the global y-direction. For the overturning moment the maximum moment occurs at the base with a value of 16788.84 kN-m and the minimum at storey 1 with a value of -11889 kN-m both in the global y-direction.

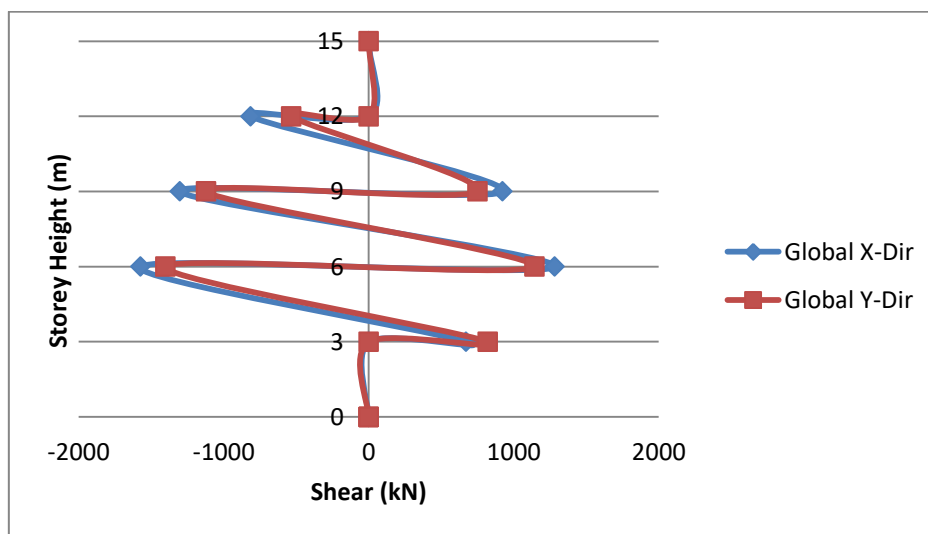


Figure 4.9: Storey response (shears) of the CSC Frame with PSB Wall under the El Centro earthquake

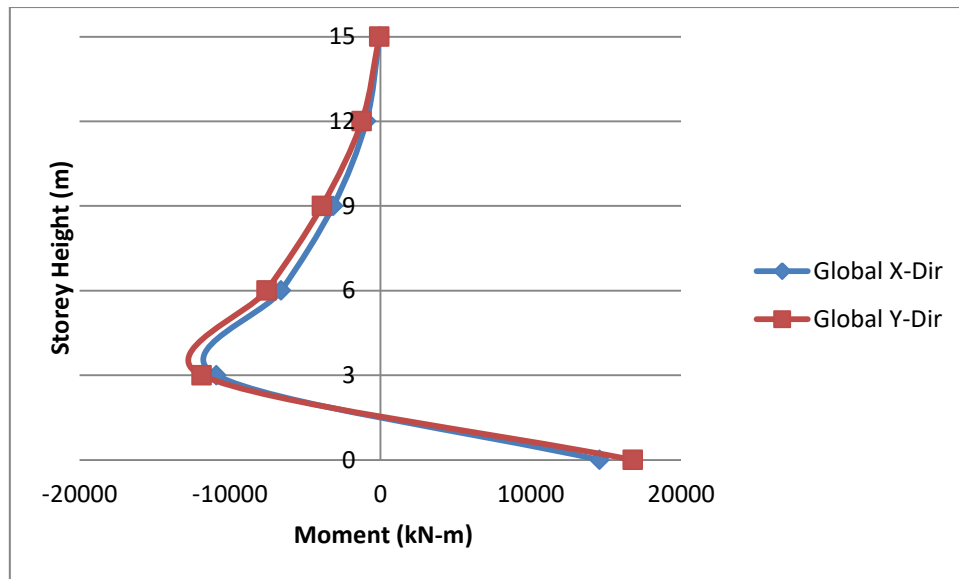


Figure 4.10: Storey response (overturning moments) of the CSC Frame with PSB Wall under the El Centro earthquake

4.2.2.2 Petrolia Earthquake

The Petrolia earthquake effects on the CSC Frame with PSB Wall are recorded in this section. Figure 4.11 shows the maximum storey displacement of the CSC Frame with PSB Wall under the Petrolia earthquake. The maximum displacement of the building is observed at the roof with a value of 50.35 mm in the global y-direction. The building deflects in the x-direction with a value of 29.97mm and occurs at the roof of the building.

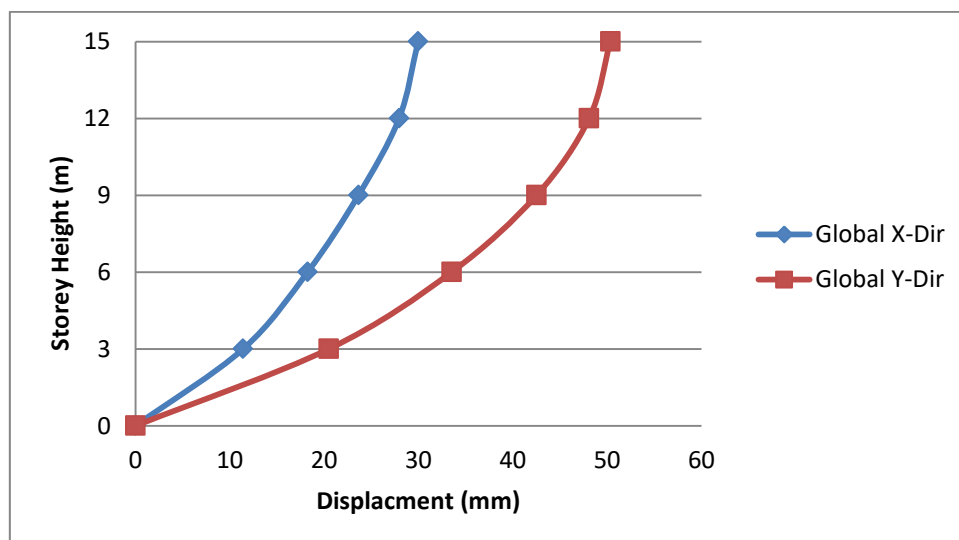


Figure 4.11: Storey response (maximum displacement) of the CSC Frame with PSB Wall under the Petrolia earthquake

The response of the building to drift under the Petrolia earthquake is shown in Figure 4.12. For each storey the minimum drift occurs at the top for both the x and y direction. And also the maximum drift of the overall building occurs at storey one with a sway value of 0.0068 in the global y direction. The drift values of the building in both the x and y directions are different for each storey level but the values are the same at the top of the building in both directions with a value of 0.0012.

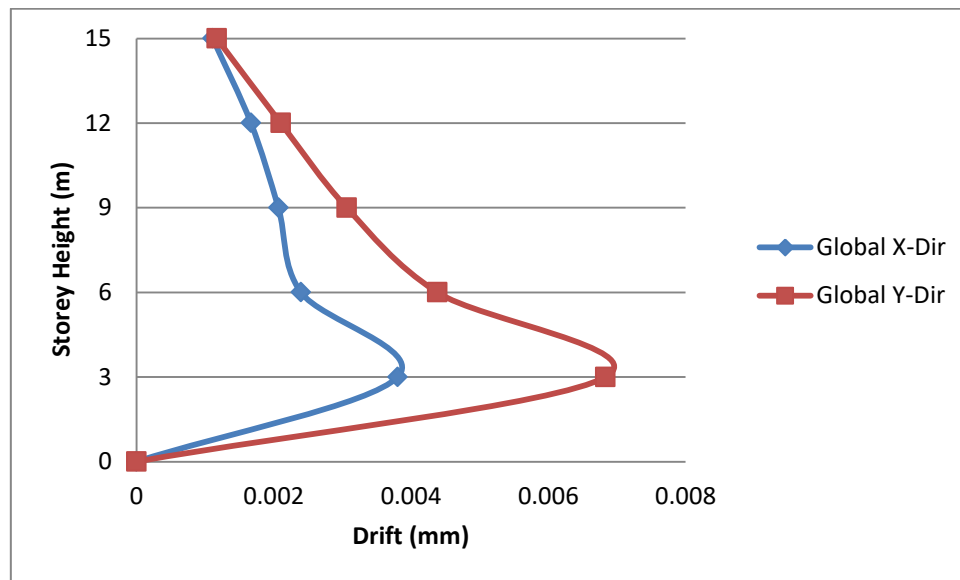


Figure 4.12: Storey response (maximum drift) of the CSC Frame with PSB Wall under the Petrolia earthquake

The maximum storey shears and the maximum overturning moments of the CSC Frame with PSB Wall under the Petrolia earthquake in both x and y direction are shown in Figure 4.13 and 4.14 respectively. The maximum shear occurs at the base with a value of 3420.004 kN in global x-direction and the minimum on storey 1 with a value of -3156.37 kN in global y-direction. For the overturning moment the maximum moment occurs at the base with a value of 30013.54 kN-m in global x-direction and the minimum at storey 1 with a value of -21519 kN-m in global y-direction.

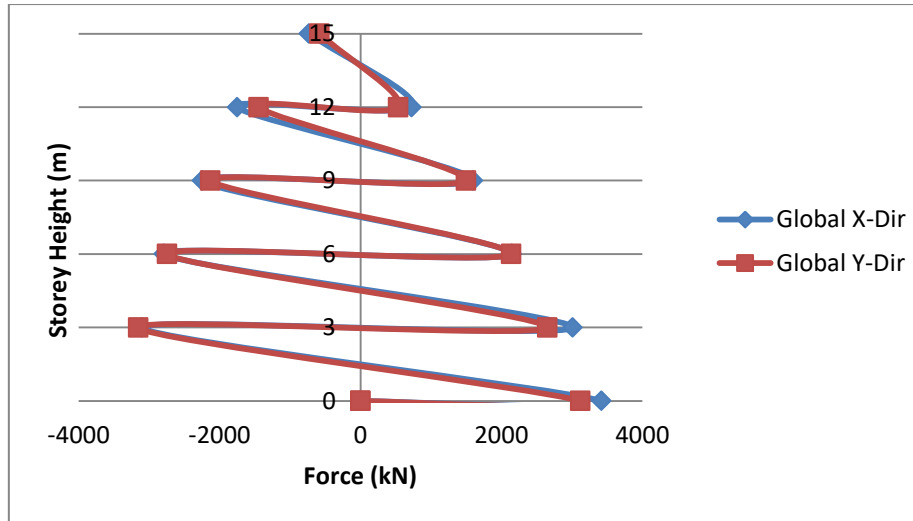


Figure 4.13: Storey response (shears) of the CSC Frame with PSB Wall under the Petrolia earthquake

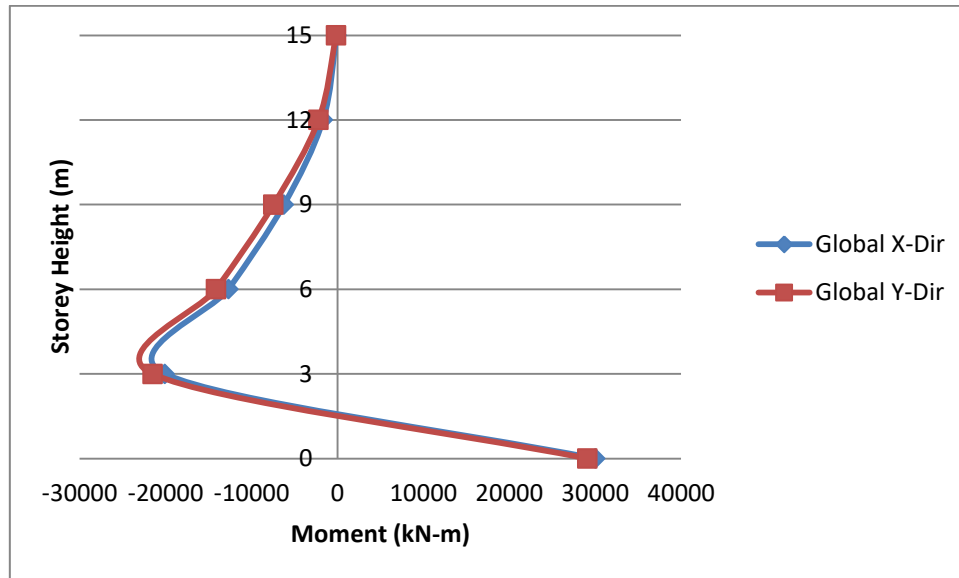


Figure 4.14: Storey response (overturning moments) of the CSC Frame with PSB Wall under the Petrolia earthquake

4.2.2.3 Lacc_North Earthquake

The effects of Northridge earthquake on the CSC Frame with PSB Wall were recorded in this section. Figure 4.15 shows the maximum storey displacements of the CSC Frame with PSB Wall under the Lacc_North earthquake in the global x and y directions. The maximum storey displacement of the building is observed at 12.37 mm in the y-direction. In the x-direction the maximum displacement also occurs at the top with a value of 6.37 mm.

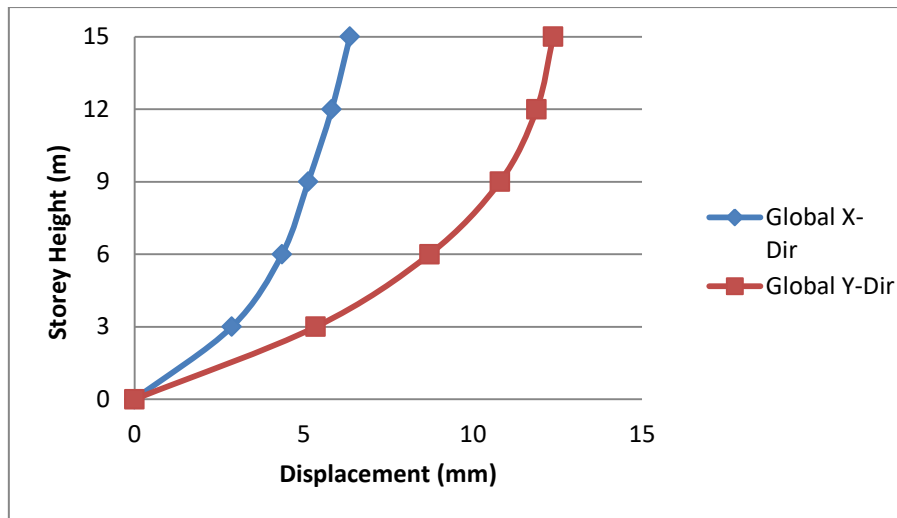


Figure 4.15: Storey response (maximum displacement) of the CSC Frame with PSB Wall under the Lacc_North earthquake

The response of the building to drift under the Lacc_North earthquake is shown in Figure 4.16. For each storey the minimum drift occurs at the top for both the global x and y direction. And also the maximum drift of the overall building occurs at storey one with a sway value of 0.0018 in the global y-direction and 0.00096 in x direction. The drift values of the building in both the x and y directions are different for each storey level and do not follow the sway patterns of the response spectrum El centro and Petrolia earthquake records. The drift values become reduced at the top to 0.00038 and 0.00028 in the global y and x direction respectively.

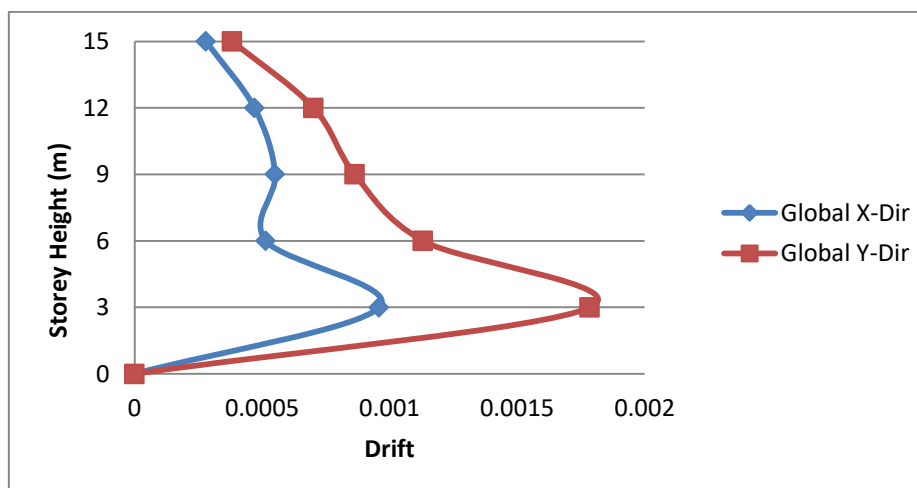


Figure 4.16: Storey response (maximum drift) of the CSC Frame with PSB Wall under the Lacc_North earthquake

The maximum storey shears and the maximum overturning moments of the CSC Frame with PSB Wall under the Lacc_North earthquake in both global x and y direction are shown in Figure 4.17 and 4.18 respectively. The maximum shear occurs at the base with a value of 1102.18 kN in global y-direction and the minimum on storey 1 with a value of -1275.88 kN in global y direction. For the overturning moment the maximum moment occurs at the base with a value of 10876.99 kN-m in global x-direction and the minimum at storey 1 with a value of -8704.48 kN-m also in global x-direction.

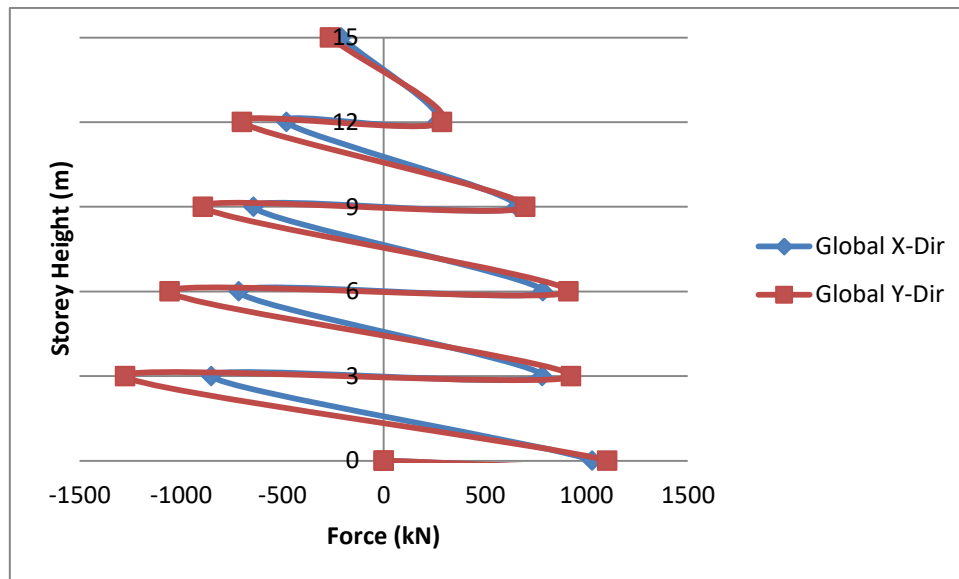


Figure 4.17: Storey response (shears) of the CSC Frame with PSB Wall under the Lacc_North earthquake

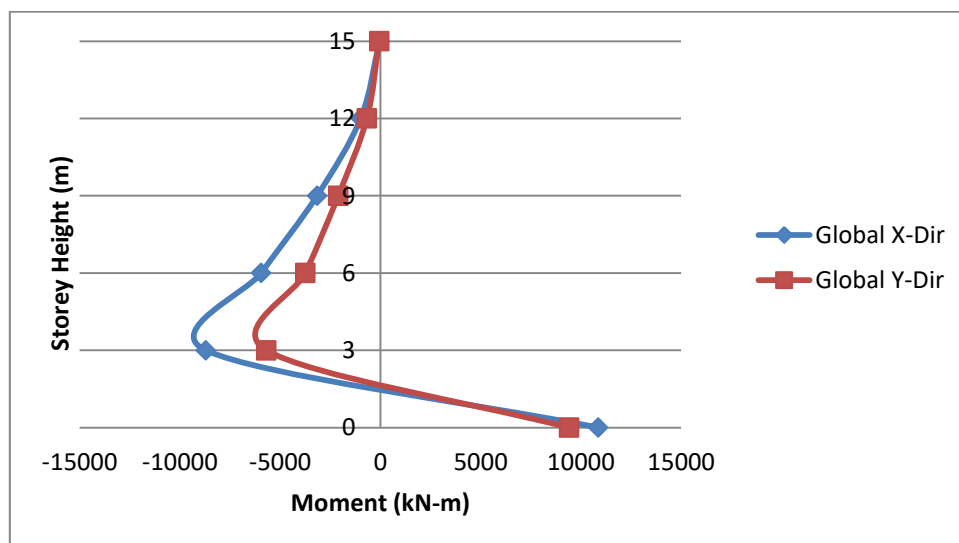


Figure 4.18: Storey response (overturning moments) of the CSC Frame with PSB Wall under the Lacc_North earthquake

4.2.2.4 Lucerne Earthquake

The 1992 Landers earthquake's effects on the CSC Frame with PSB Wall are documented in this section. The maximum displacement of the CSC Frame with PSB Wall under the Lucerne earthquake is shown in Figure 4.19 with a maximum displacement of 13.25 mm at the roof in the global y direction. In the x direction it also has maximum displacement of 8.83 mm at the top.

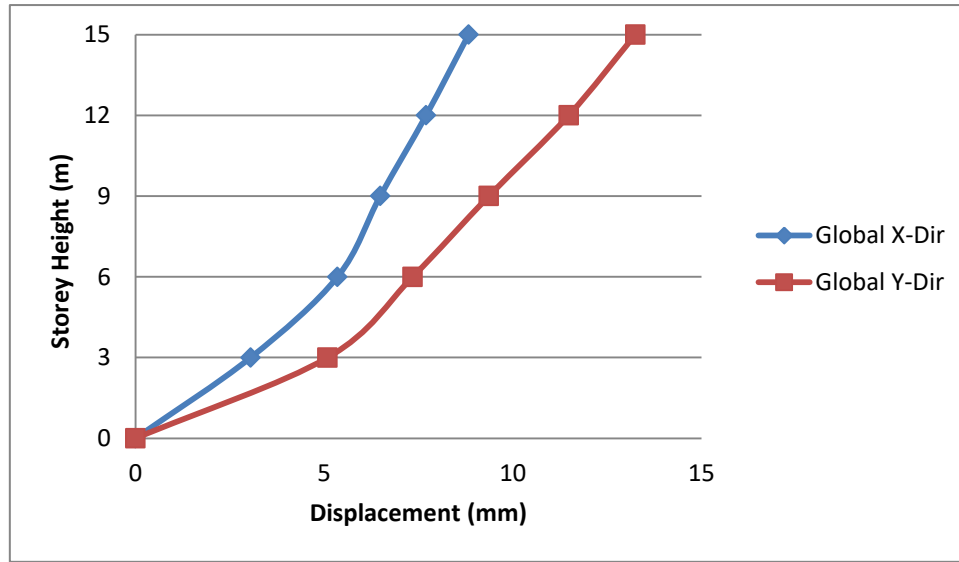


Figure 4.19: Storey response (maximum displacement) of the CSC Frame with PSB Wall under the Lucerne earthquake

The maximum storey drift of the CSC Frame with PSB Wall under the Lucerne earthquake is shown in Figure 4.20 with a maximum drift of 0.0017 at storey one in the global y direction. The drift values of the building in both the x and y directions are different for each storey level and do not follow the sway patterns of the response spectrum El centro and Petrolia earthquake records.

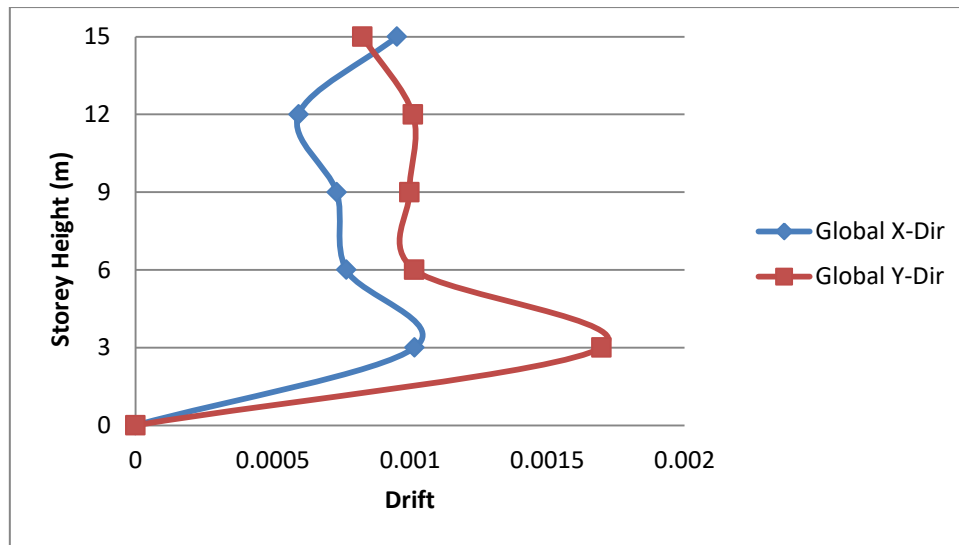


Figure 4.20: Storey response (maximum drift) of the CSC Frame with PSB Wall under the Lucerne earthquake

The maximum storey shears and the maximum overturning moments of the CSC Frame with PSB Wall under the Lucerne earthquake are shown in Figure 4.21 and 4.122 respectively. The maximum shear occurs at the base with a value of 1844.99 kN in global y-direction and the minimum on storey 1 with a value of -2023.36 kN in global y-direction. For the overturning moment the maximum moment occurs at the base with a value of 16680.37 kN-m in global x-direction and the minimum at storey 1 with a value of -11998 kN-m in global y-direction.

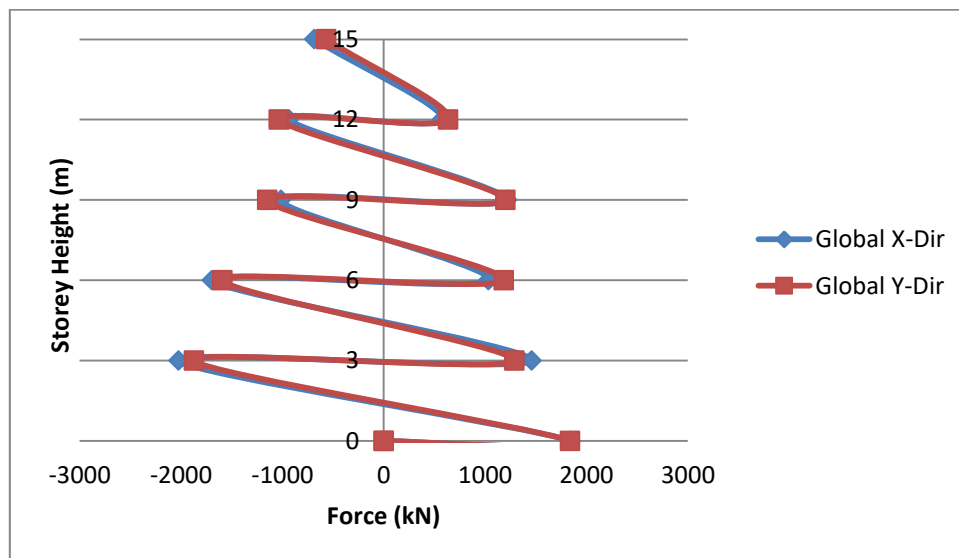


Figure 4.21: Storey response (shears) of the CSC Frame with PSB Wall under the Lucerne earthquake

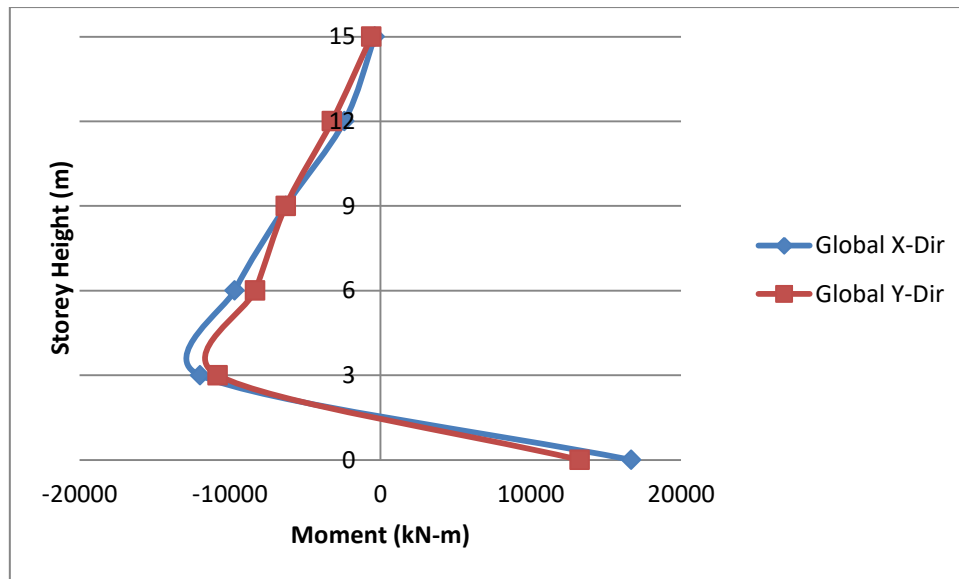


Figure 4.22: Storey response (overturning moments) of the CSC Frame with PSB Wall under the Lucerne earthquake

4.2.2.5 Slymarff Earthquake

The San Fernando earthquake (also known as the Sylmar earthquake) which occurred on February 9 1971 was also used for the time history analysis and its effects are recorded and discussed in this section. Figure 4.23 shows the maximum storey displacement of the CSC Frame with PSB Wall observed during the Slymarff earthquake. The building has a maximum displacement of 31.37 mm at the top in the global y-direction.

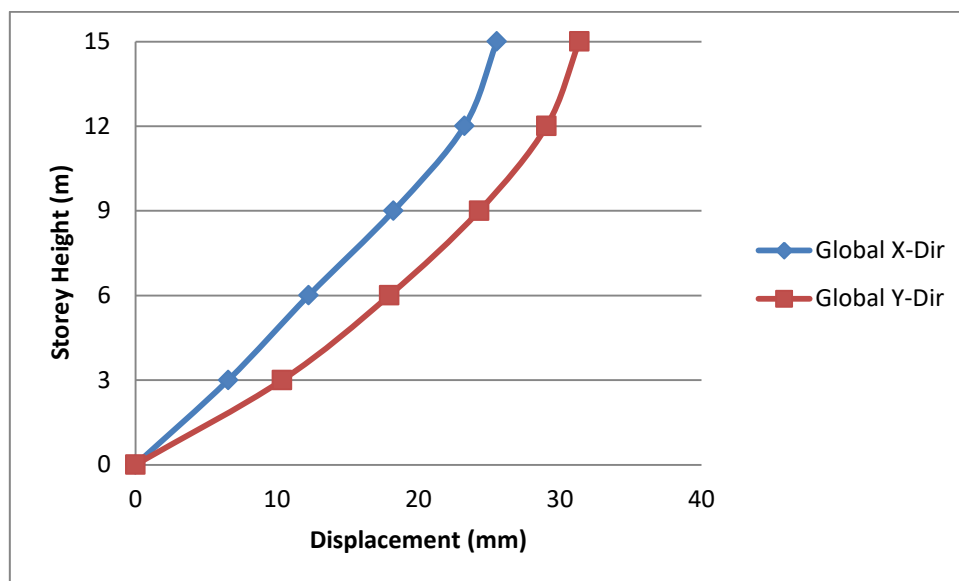


Figure 4.23: Storey response (maximum displacement) of the CSC Frame with PSB Wall under the Slymarff earthquake

The maximum drift of the CSC Frame with PSB Wall that was observed under the Slymarff earthquake occurs at the top of storey one with a value of 0.0035 in global y-direction as shown in Figure 4.24. After reaching the maximum at the 1st storey the sway value reduces as the storey height increases in the global y-direction. It does not follow the same pattern in the x-direction.

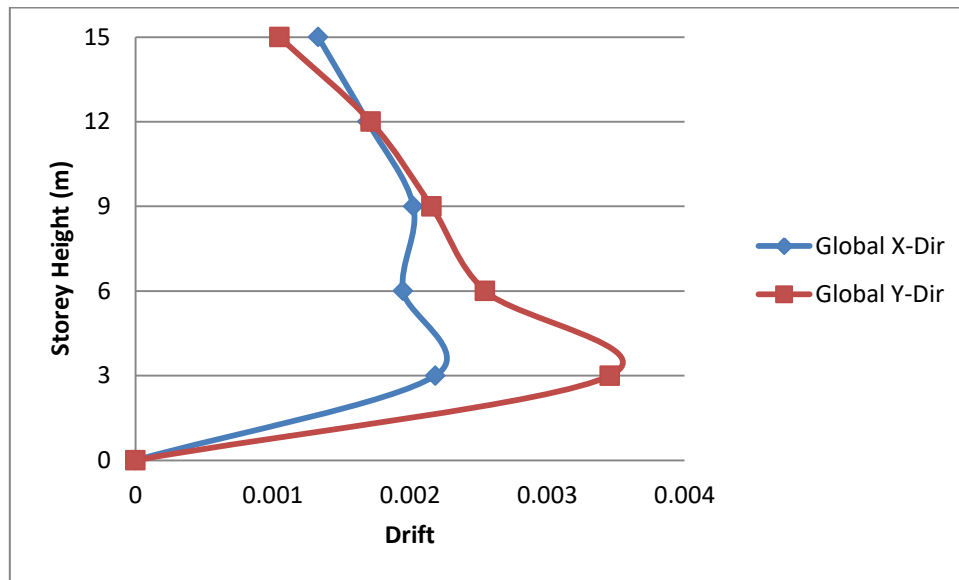


Figure 4.24: Storey response (maximum drift) of the CSC Frame with PSB Wall under the Slymarff earthquake

The maximum storey shears and the maximum overturning moments of the building in both x and y direction are shown in Figure 4.25 and 4.26 respectively.

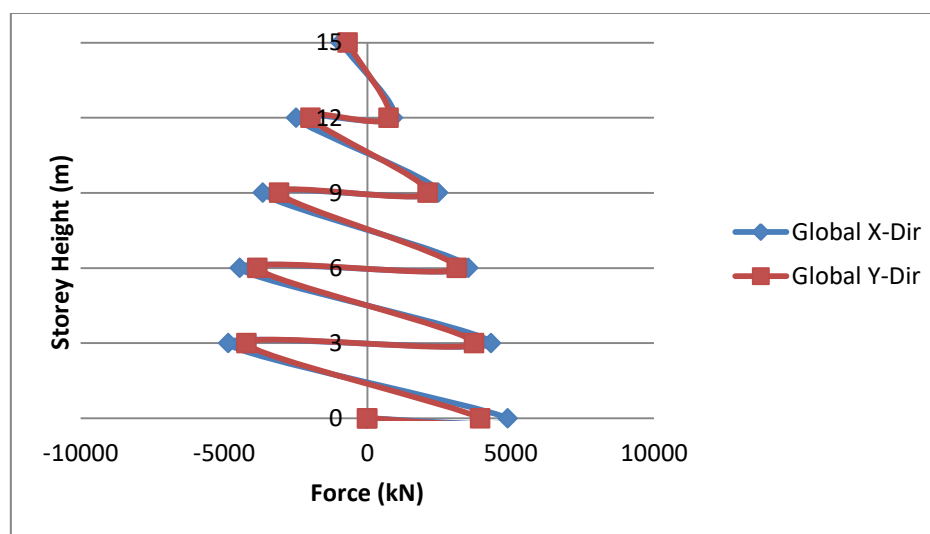


Figure 4.25: Storey response (shears) of the CSC Frame with PSB Wall under the Slymarff earthquake

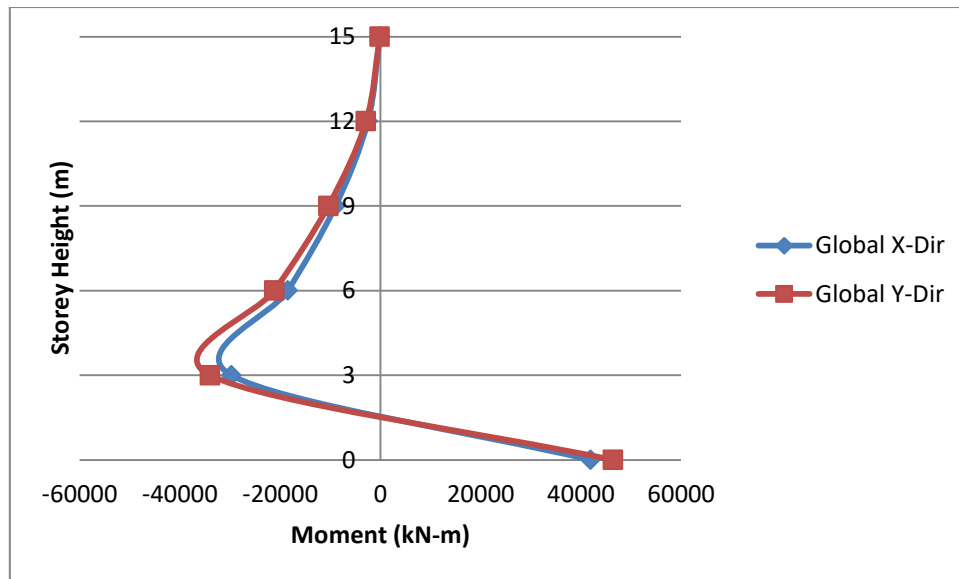


Figure 4.26: Storey response (overturning moments) of the CSC Frame with PSB Wall under the Slymarff earthquake

The maximum shear occurs at the base with a value of 4893.27 kN in global x-direction and the minimum on storey 1 with a value of -4855.38 kN in global x-direction. For the overturning moment the maximum moment occurs at the base with a value of 46411.34 kN-m in global y-direction and the minimum at storey 1 with a value of -33994 kN-m in global x-direction.

4.2.3 Comparison of the Building Response to Different Historical Earthquakes

The response of the CSC Frame with PSB Wall under seismic analysis was studied. Figure 4.27 and 4.28 shows the deflection of the building under various earthquake loads in the x and y direction respectively.

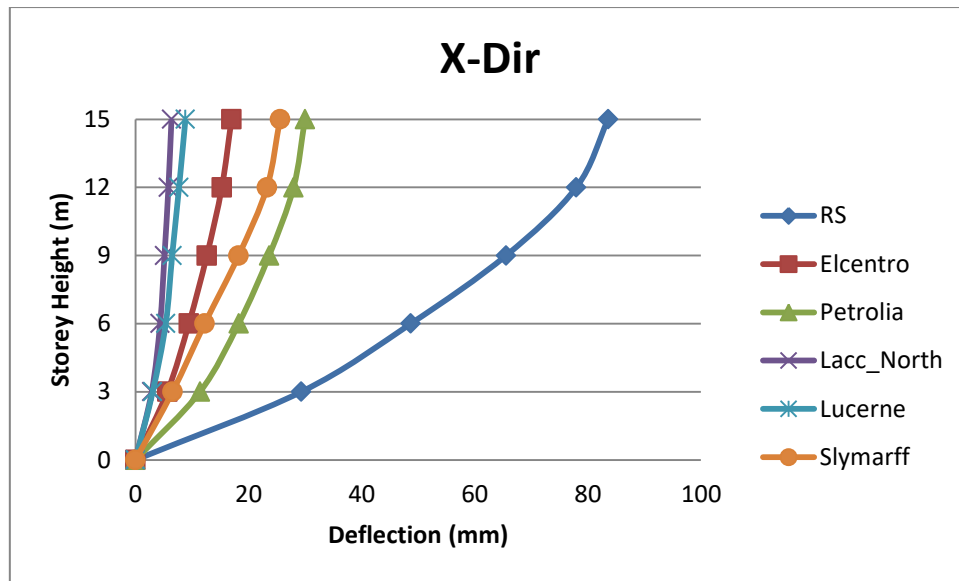


Figure 4.27: Deflection of the CSC Frame with PSB Wall in the x direction under various earthquake loads

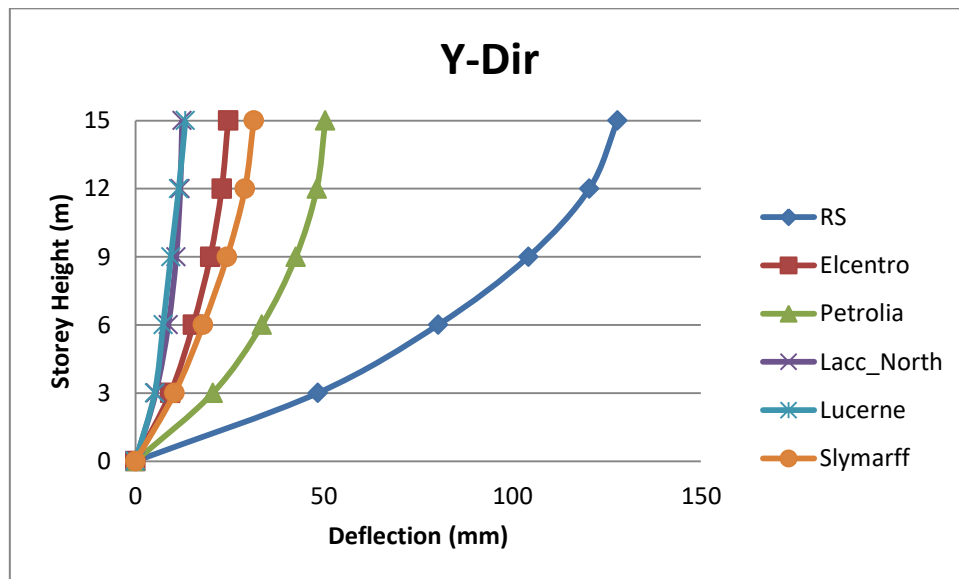


Figure 4.28: Deflection of the CSC Frame with PSB Wall in the y direction under various earthquake loads

From the analysis it was observed that the building deflects at a higher rate in the global y-direction than the global x-direction. The Haiti earthquake which was simulated using the response spectrum analysis produces the highest deflection as compared to other historical earthquakes. It gives the highest deflection of 83.59 mm in the global x-direction and 127.85 mm in the global y-direction. The highest deflections of the building occur at the y-direction for all the different earthquake records. This shows that the shape of buildings contribute to its resistance behaviour in the presence of earthquakes and thus the lower deflection in the x-

direction. Figure 4.29 and 4.30 show the storey drift of the building in both global x and y direction respectively.

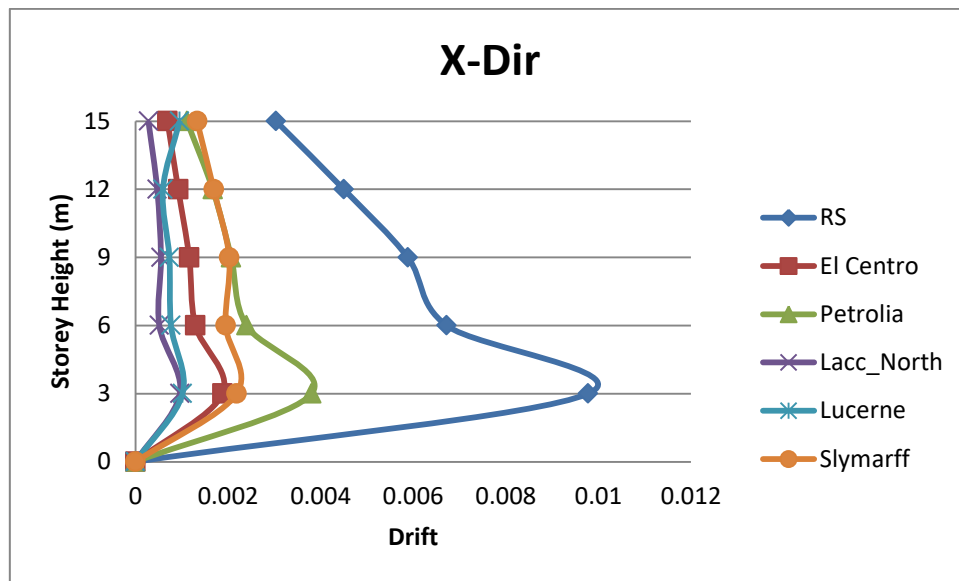


Figure 4.29: Drift of the CSC Frame with PSB Wall in the x direction under various earthquake loads

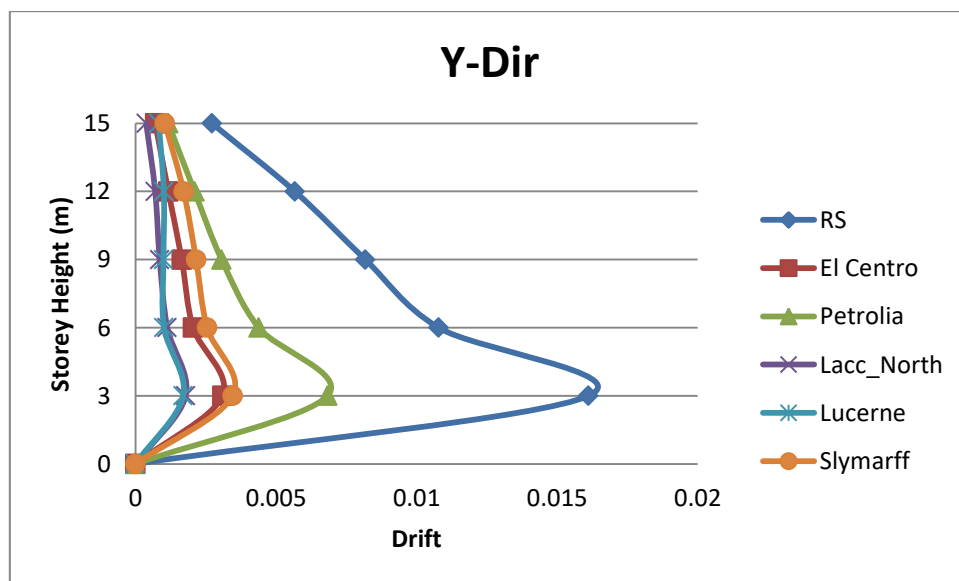


Figure 4.30: Drift of the CSC Frame with PSB Wall in the y direction under various earthquake loads

The Haiti earthquake (Response Spectrum) produced the highest storey drift when compared with the other earthquakes in both x and y directions as it uses the peak ground acceleration of an earthquake in an earthquake building simulation. It has its maximum drift at the first

floor with a value of 0.0098 in the global x-direction and 0.016 in the global y-direction. The highest drift patterns are observed in the y-direction for all the earthquakes as the building exhibits higher resistance in the global x-direction when compared to the y-direction.

4.3 Comparison between the Responses of CSC with CFB

The deflection and drift of the CFB and the CSC frame with PSB wall are compared in Figure 4.31 to Figure 4.34. In the x direction the CSC frame with PSB wall has a maximum deflection of 29.97 mm while the CFB has a maximum deflection of 33.03 mm at the top of the building as shown in Figure 4.31. In the y direction the CSC frame with PSB wall has a value of 50.35 mm while the CFB has a value of 53.64 mm at the top as shown in Figure 4.32.

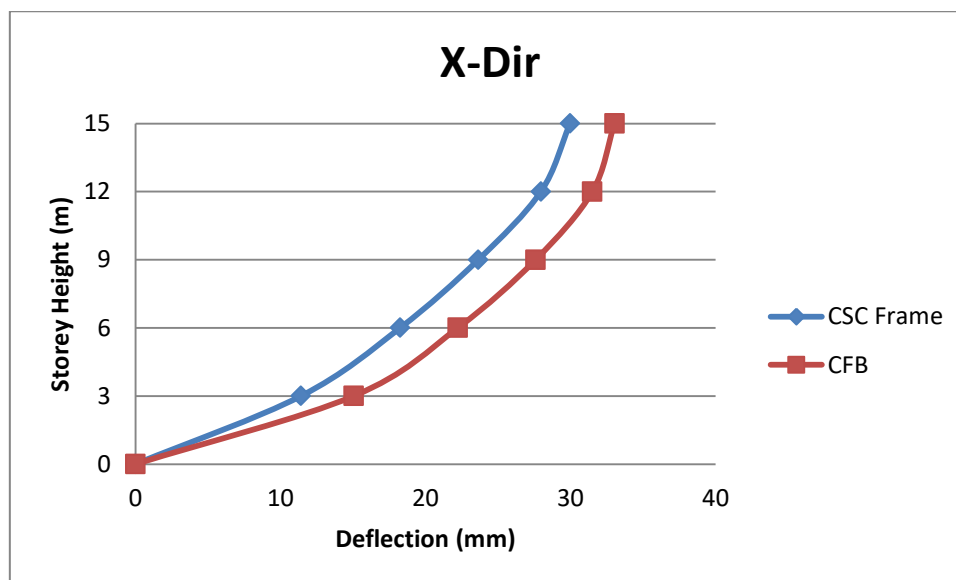


Figure 4.31: Deflection of the CSC Frame with PSB Wall and CFB in the x-dir

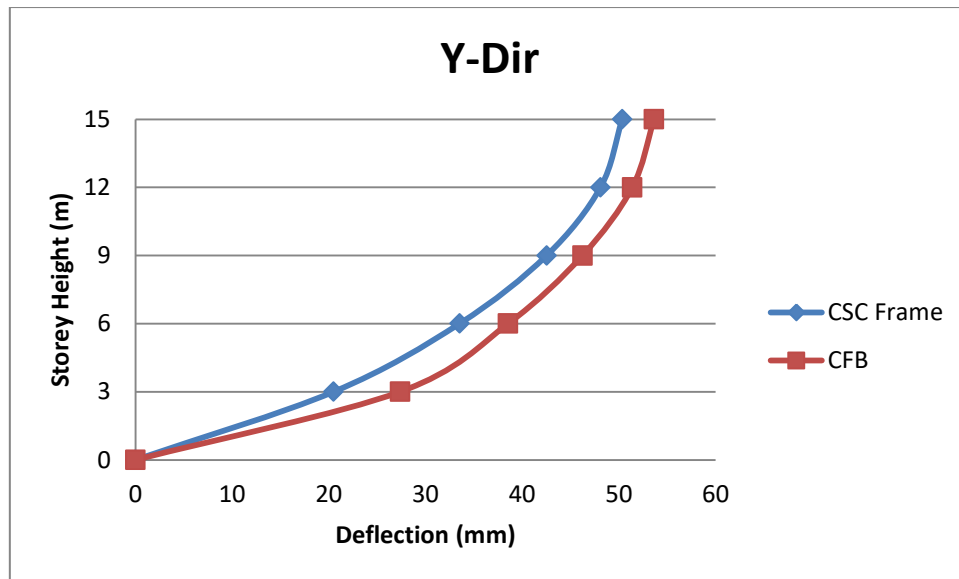


Figure 4.32: Deflection of the CSC Frame with PSB Wall and CFB in the y-dir

In the x direction the CSC frame with PSB wall has a maximum drift value of 0.0038 while the CFB has a maximum drift of 0.005 as shown in Figure 4.33. The maximum drift in the y direction shows CSC frame with PSB wall with a value of 0.0068 while CFB has a value of 0.0091 as shown in Figure 4.34. The CSC frame with PSB wall shows lower deflection and drift values when compared to the CFB and this is as a result of its high stiffness.

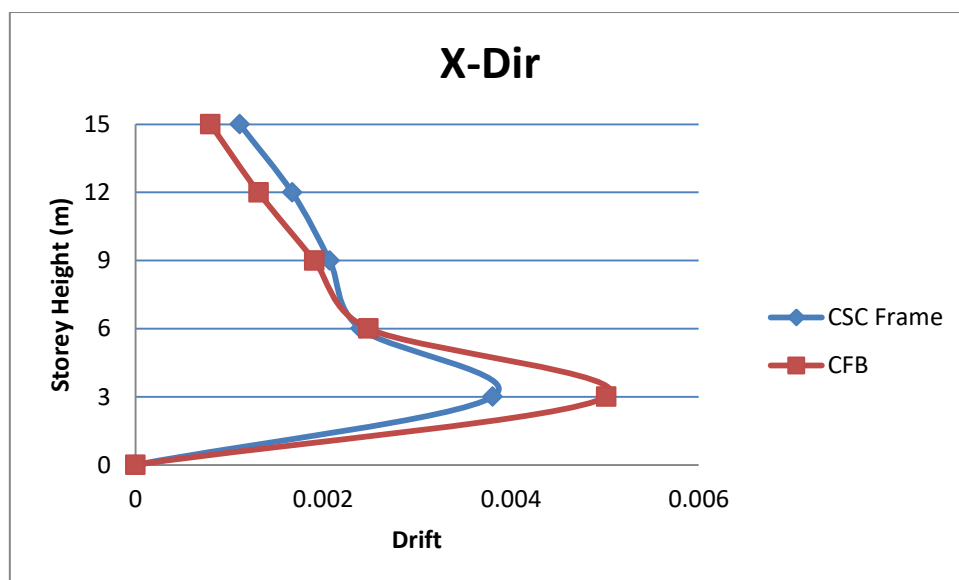


Figure 4.33: Drift of the CSC Frame with PSB Wall and CFB in the x-dir

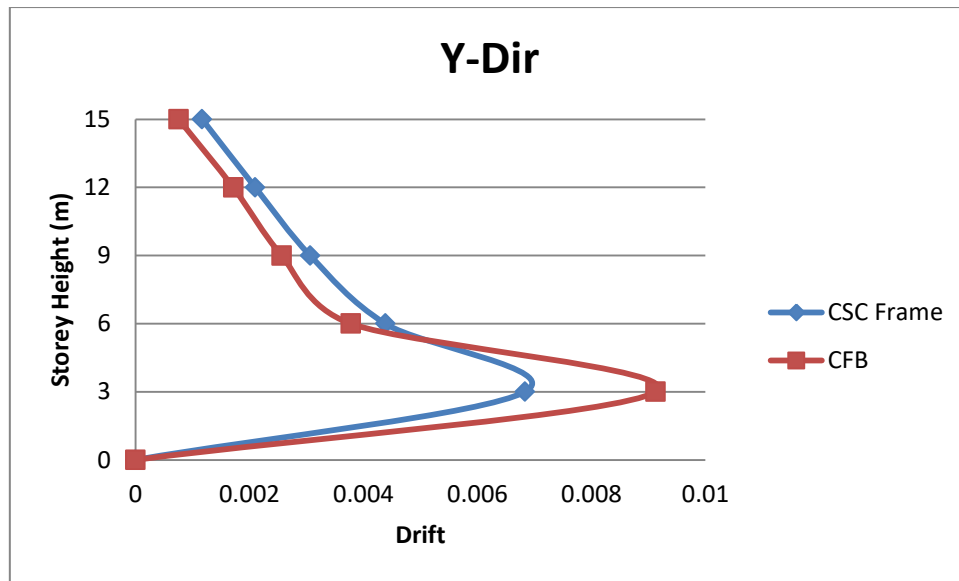


Figure 4.34: Drift of the CSC Frame with PSB Wall and CFB in the y-dir

The response of the CFSC building with PSB wall under seismic analysis has been studied. The response under various earthquake loads can be compared for further study. The building has its maximum drift on storey one in both global x and y-direction. The response spectrum has maximum drift values of 0.0098 in the global x-direction and 0.016 in the global y-direction.

In verification of the damage limitation according to Eurocode 8 (EN 1998-1/4.4.3.2) It can be seen that the most severe drift limit ($d_r v \leq 0.005h$ for a building having non-structural elements of brittle materials attached to the structure) is not exceeded in any storey. For the maximum storey drift in first storey the resultant value of $d_r v$ is 0.0064 which is less than 0.015. This shows that the maximum displacement and drift of the CFSC building with PSB wall is safe from damage under Haiti (response spectrum) earthquake.

Also the maximum storey drift of the Petrolia earthquake occurred at the first storey with a value of 0.0068 hence the resultant value of $d_r v$ is 0.0027 and is less than 0.015. This result verifies the safety of the cold-formed steel frame building with straw-bale panels under the El Centro Petrolia Lacc_North Lucerne and the Slymarff earthquake.

CHAPTER FIVE

5.0 CONCLUSIONS AND RECOMMENDATIONS

5.1 Conclusions

The building has its maximum displacement at the global y-direction of the building model which is 9.68m wide while the displacement was minimal at the global x-direction with 22.125m length. The CSC frame building with SIP offered 25.3% more of resistance to external forces and building model showed a total weight of 16067.11kN and modal analysis of first 6 modes ranged between 0.644 to 0.117 periods.

The maximum displacement occurred at the top of the building with a value of 127.85 mm for Haiti earthquake (Response Spectrum) while the maximum drift occurred in the global y-direction on the first Storey with a value of 0.016.

The Five Time History data showed a value of 24.57 mm for El Centro earthquake 50.35 mm for Petrolia 12.37 mm for the Lacc_North 13.25 mm for Lucerne and 31.37 mm for the Slymarff earthquake. The building has its maximum displacement under the Petrolia earthquake while the maximum drift is in the global y-direction on the first floor for all the earthquakes with a maximum value of 0.003091 0.006832 0.001785 0.001698 and 0.003456 for El Centro Petrolia Lacc_North Lucerne and Slymarff earthquake respectively.

With the response spectrum the shear decreases as the height of the building increase from 0 kN at the base to 10967.91 kN at first storey and to 2120.67 kN at the roof level in the global x-direction. The overturning moment also decreases as the height of the building increases from 109205.06 kNm at the base to 248.75 kNm at the roof level (i.e. the overturning moment is high at the base of the building).

The results also showed that CSC frame building with SIP under seismic load has a lower storey drift value of 0.0068 compared to conventional frame building (CFB) with drift value of 0.00912 this is due to CSC frame building having lower stiffness compared to the CFB.

Under all the historical earthquakes the shear of the building decreases as the height of the building increases.

The seismic responses i.e. the base shear storey displacement storey drift in both global x and y directions were found to vary in similar pattern for all the Time Histories studied. The maximum shear occurs at the base (Foundation) of the building since this is where the most

impact of the earthquake is felt before the resonance effect is spreads to the storey height this resonance effect from the base shear results into having the maximum displacement at the top of the building.

The result of the safety verification under Eurocode 8 verifies the safety of the cold-formed steel (CSC) frame building with straw-bale infill panels (SIP) under the Haiti (Response spectrum) and Time History data of El Centro Petrolia Lacc_North Lucerne and the Slymarff earthquake.

This project is further validated by comparing the response of the modelled building to the research work carried out by Abolarin and Adedeji (2016). In their work a maximum displacement of 163.63 mm was obtained at the top of a three storey reinforced concrete building under seismic loading as compared to 127.85 mm obtained at the top of the four storey cold formed steel concrete frame with plastered straw bale infill panel building.

5.2 Recommendations

Based on the result obtained from the seismic analysis of the building the maximum displacement and drift occurred on the global y-direction of the building. This shows that building geometry should also be considered in the construction of earthquake resistant structure. CSC building with SIP should be incorporated in building standards and codes as this composition was observed to be more stable under earthquake load than CFB.

After being subjected to seismic analysis the cold-formed steel concrete building with plastered strawbale walls is in accordance with the damage limitations of Eurocode 8 (EN 1998-1: 20004 /4.4.3.2). Hence the damage limitation requirement of the building is considered to have been satisfied and therefore the building is considered as an earthquake resistant structure.

To further understand the performance of the cold-formed steel concrete building it is recommended that further study on the building be carried out and subjected to various historical earthquakes data with more emphasis on the foundation in order to determine the response of the structure during the occurrence of the earthquakes.

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APPENDIX

APPENDIX A

General Procedure for Modelling a Building Structure in CSI ETABS

1. **Start:** Launch ETABS by clicking on the Start Menu then All Programs select the ETABS icon from the list of the programs. It is assumed that ETABS must have been installed on your computer system. Then from “File” menu select New Model and change the settings to your preferred choice. Set the grid and click ok. Figure A1 shows the ETABS version 15.0.0 interface.

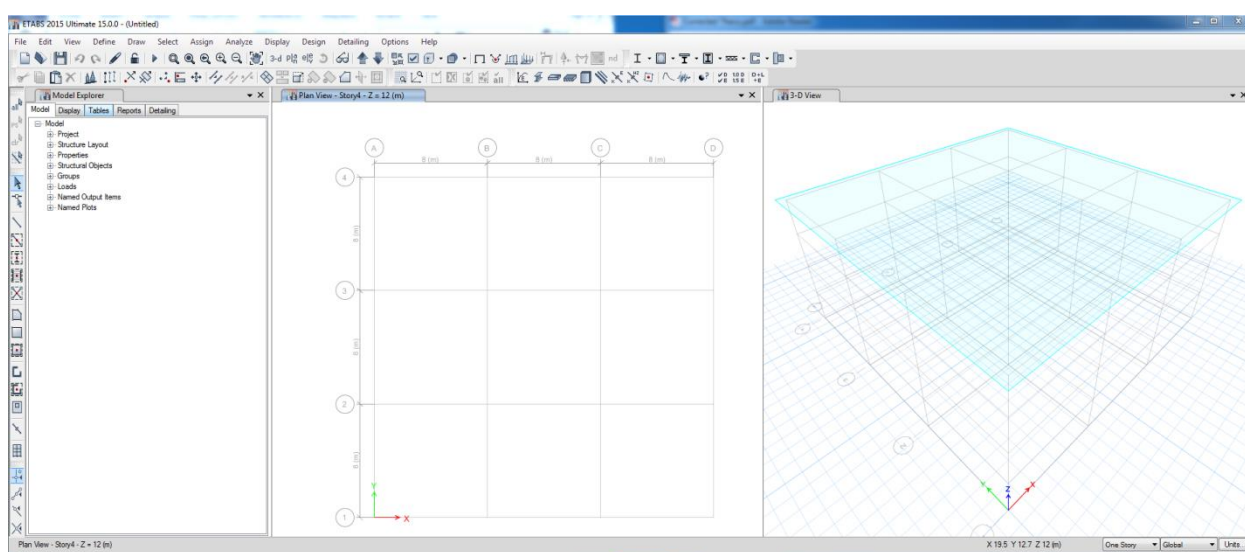


Figure A1: ETABS Version 15.0.0 Interface

2. **Defining Material Properties:** On the menu bar click the “define” menu and select “materials” item from the drop down list to display “define materials” form. The inbuilt material properties may be modified or Click “add new material” button to add new material property definition.
3. **Defining Member Sections:** click the “define” menu from the menu bar and choose ‘frame sections or slab sections’ submenu depending on the member that is to be modelled. The inbuilt material properties may be modified or click “add new property” button to add new material property definition. Sections may be imported from the inbuilt sections of the software.
4. **Defining Load and Load Combinations Cases:** This allows you to specify the loads that are acting on the bridge components. Click “define” menu select “load Pattern” item

and pick “Add new load pattern” to display the “Load pattern” form. Add the loads acting on the bridge. To define the load combinations Click “define” menu select “load combination” item and pick “Add new Combo” to display the “Load combination data” form. Create combinations for the loads based on the design codes.

5. **Assigning Loads:** This is adding the specified load to the structural members. Click “assign” menu select the ‘frame loads’ submenu and pick ‘distributed’ to assign distributed load to the beams. Select ‘shell loads’ and pick ‘uniform’ to add load to the slabs.
6. **Analyze for the Already Defined Cases:** This is the processing stage. Click the “analyze” menu and select “set load cases to run” item. On the “set load cases to run” form select the cases to run and select the “run/do not run” button. Click “run now” button to run the analysis.
7. **Check Moment Forces and Reactions:** click on the “display” menu and select the “deformed shape” and then “force/stress diagram” item. Place cursor on each of the members to display the moment forces and reactions.
8. **Design Member:** Click “design” menu and select “view/revise preference” item to choose the preferred code for design. Click the “design” menu select “concrete frame design” item and then choose “start design/ check structure”.
9. **End:** This is Exiting the program after getting all the required results. Click the “file” menu select “Exit” item.

APPENDIX B

Modes of the Building during the Seismic Analysis

During the seismic event (analysis) the first six modes of the building were documented in this section. The deflection of the building in both x and y direction for each modes are also recorded.

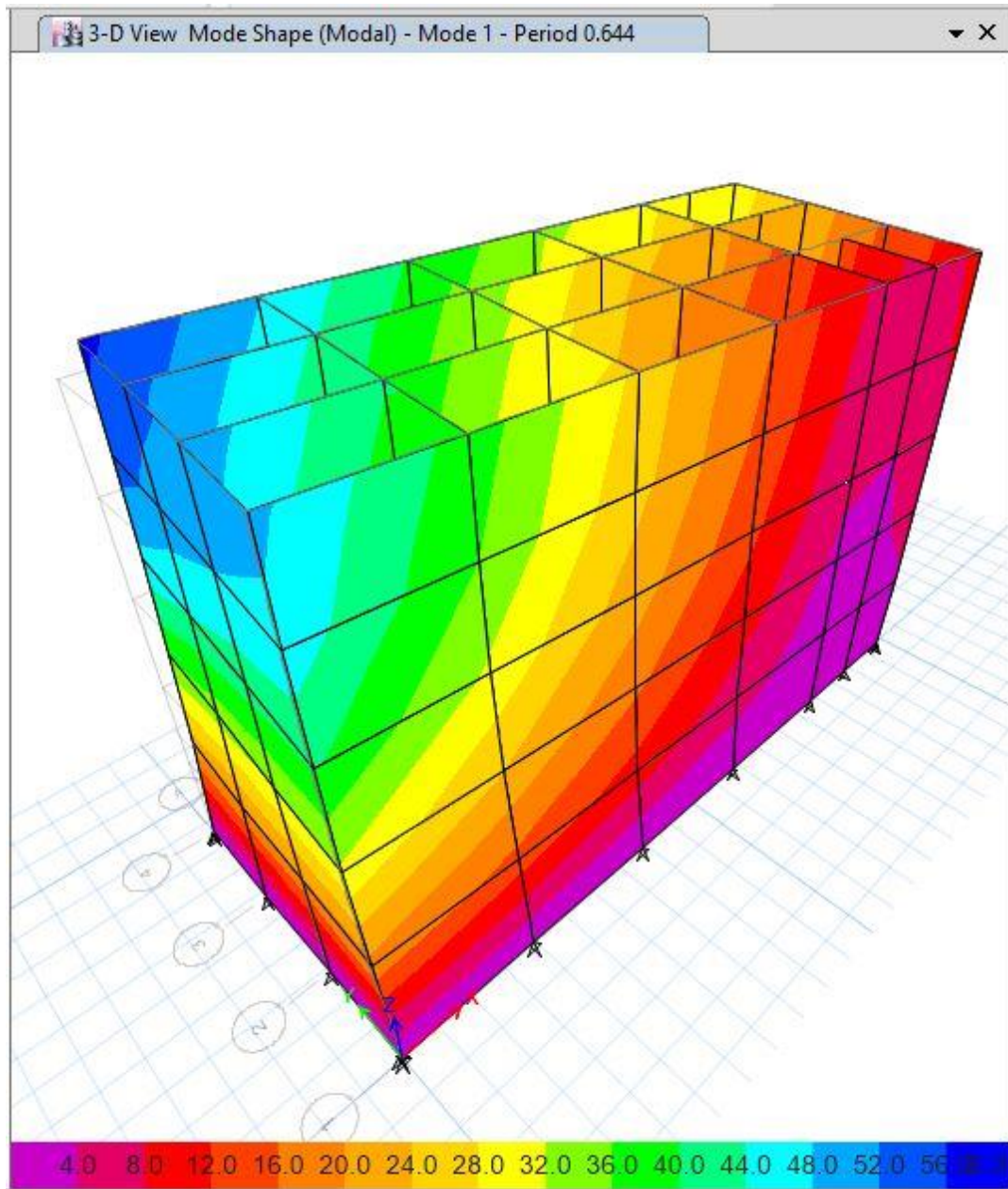


Figure B1: Mode 1 Building Reaction

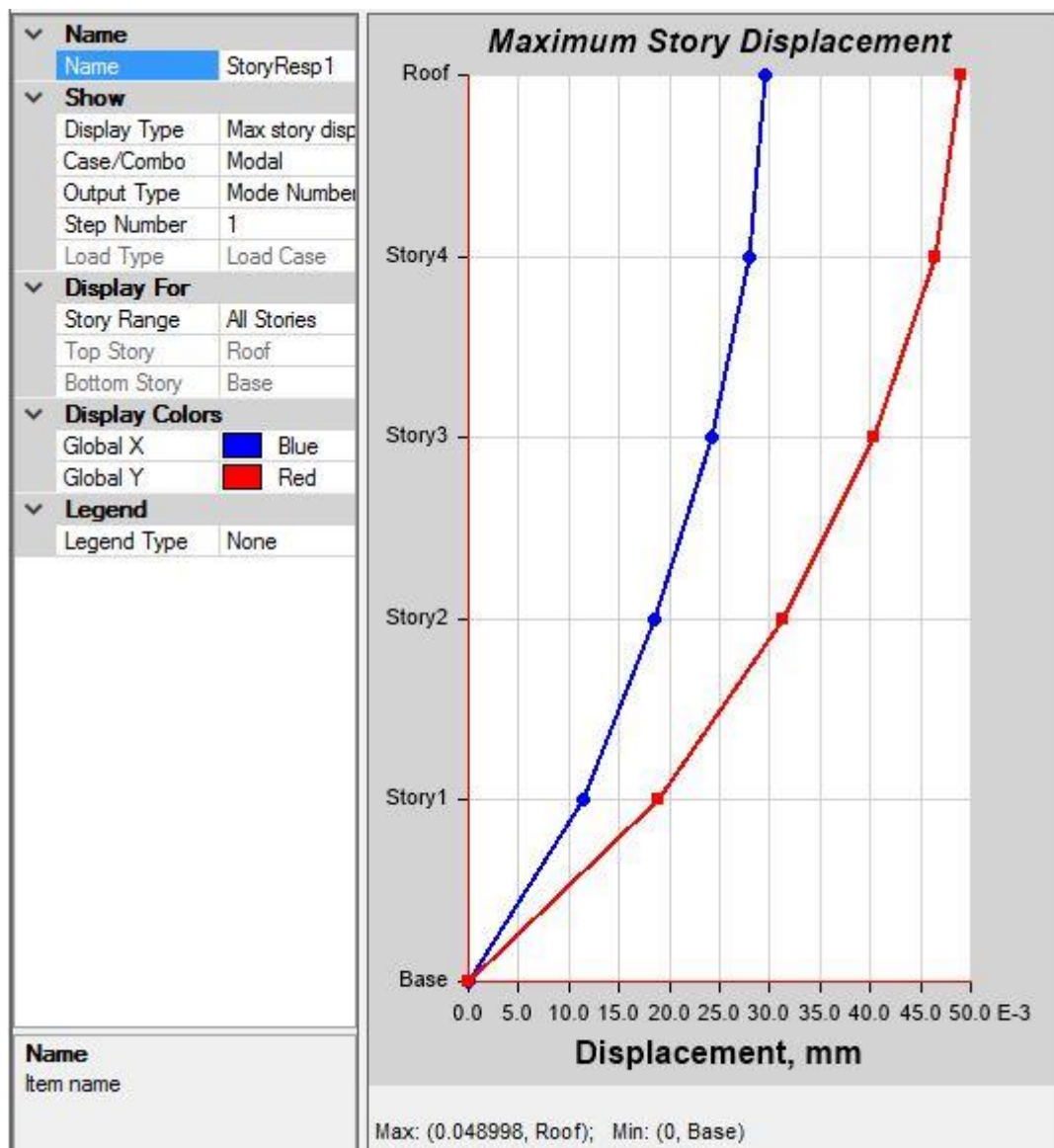


Figure B2: Displacement Plot of the Building for Mode 1

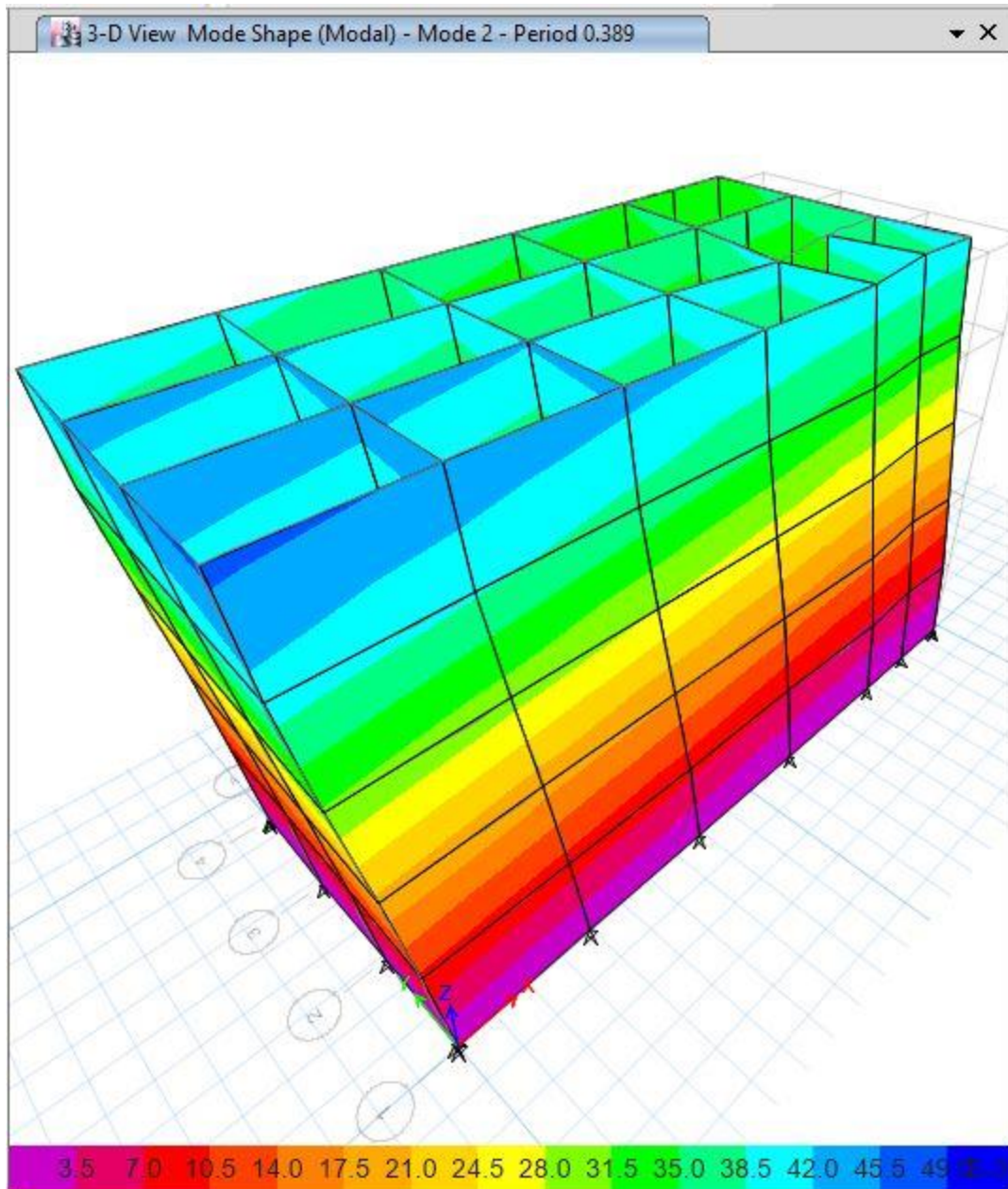


Figure B3: Mode 2 Building Reaction

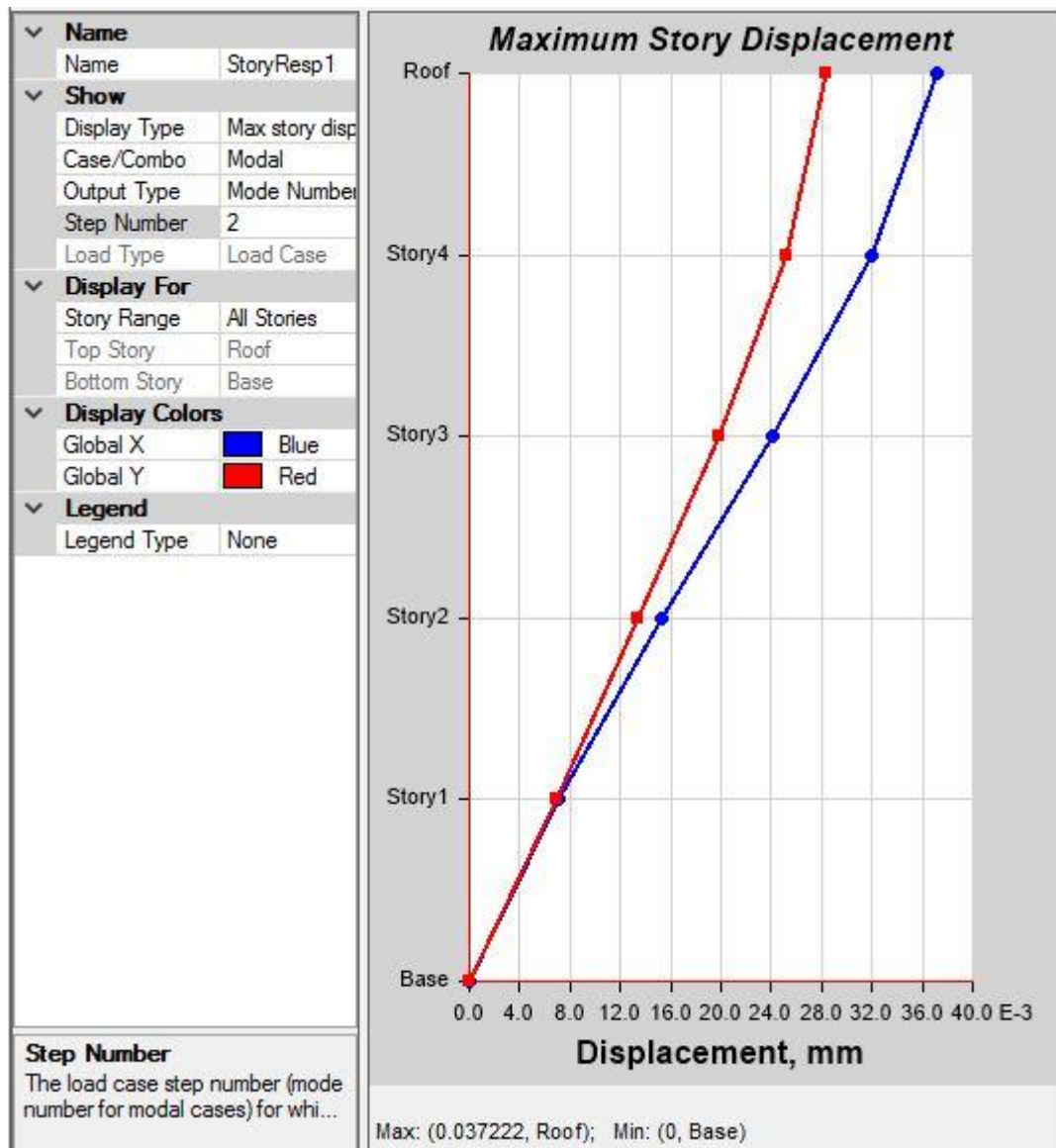


Figure B4: Displacement Plot of the Building for Mode 2

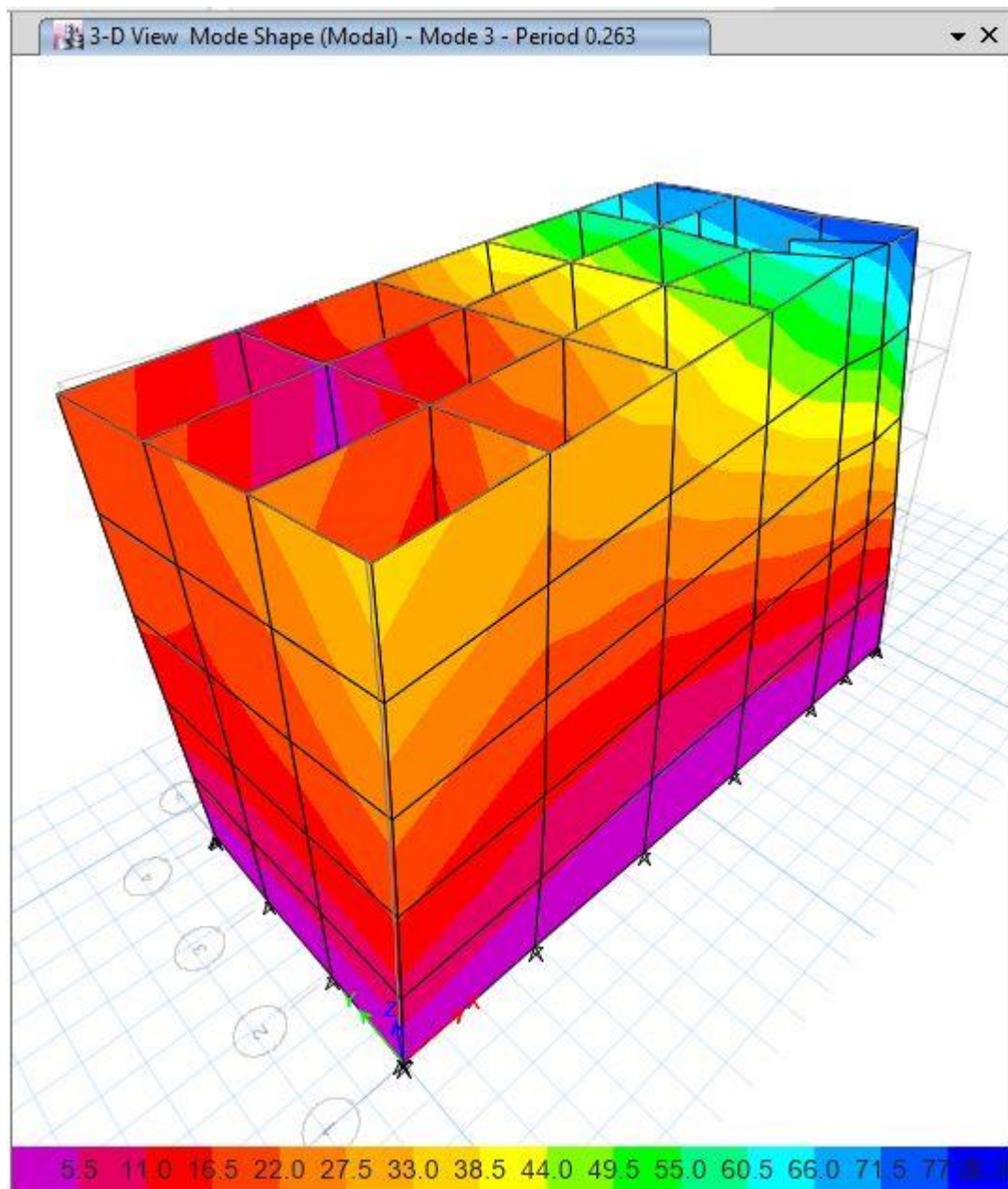


Figure B5: Mode 3 Building Reaction

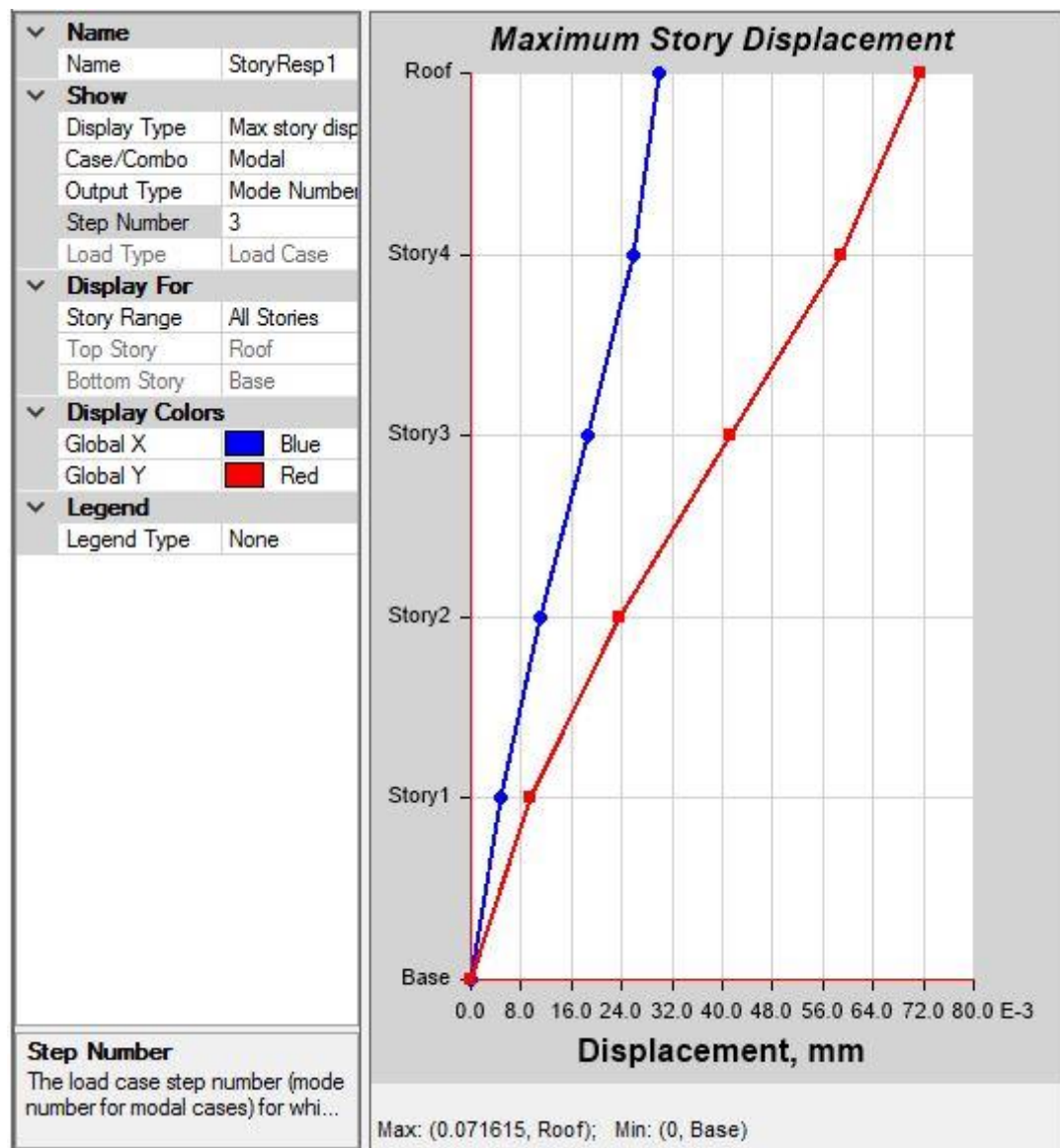


Figure B6: Displacement Plot of the Building for Mode 3

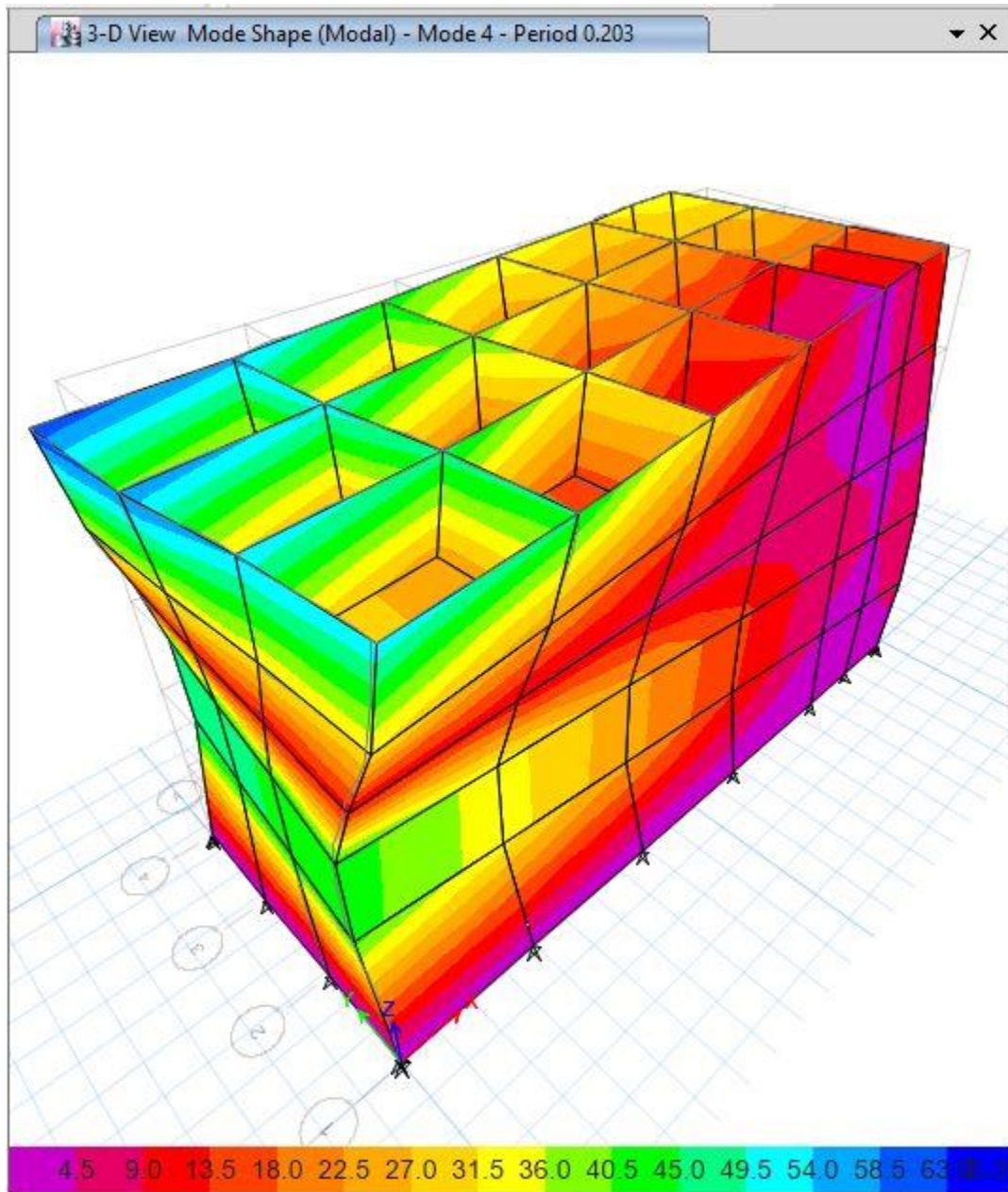


Figure B7: Mode 4 Building Reaction

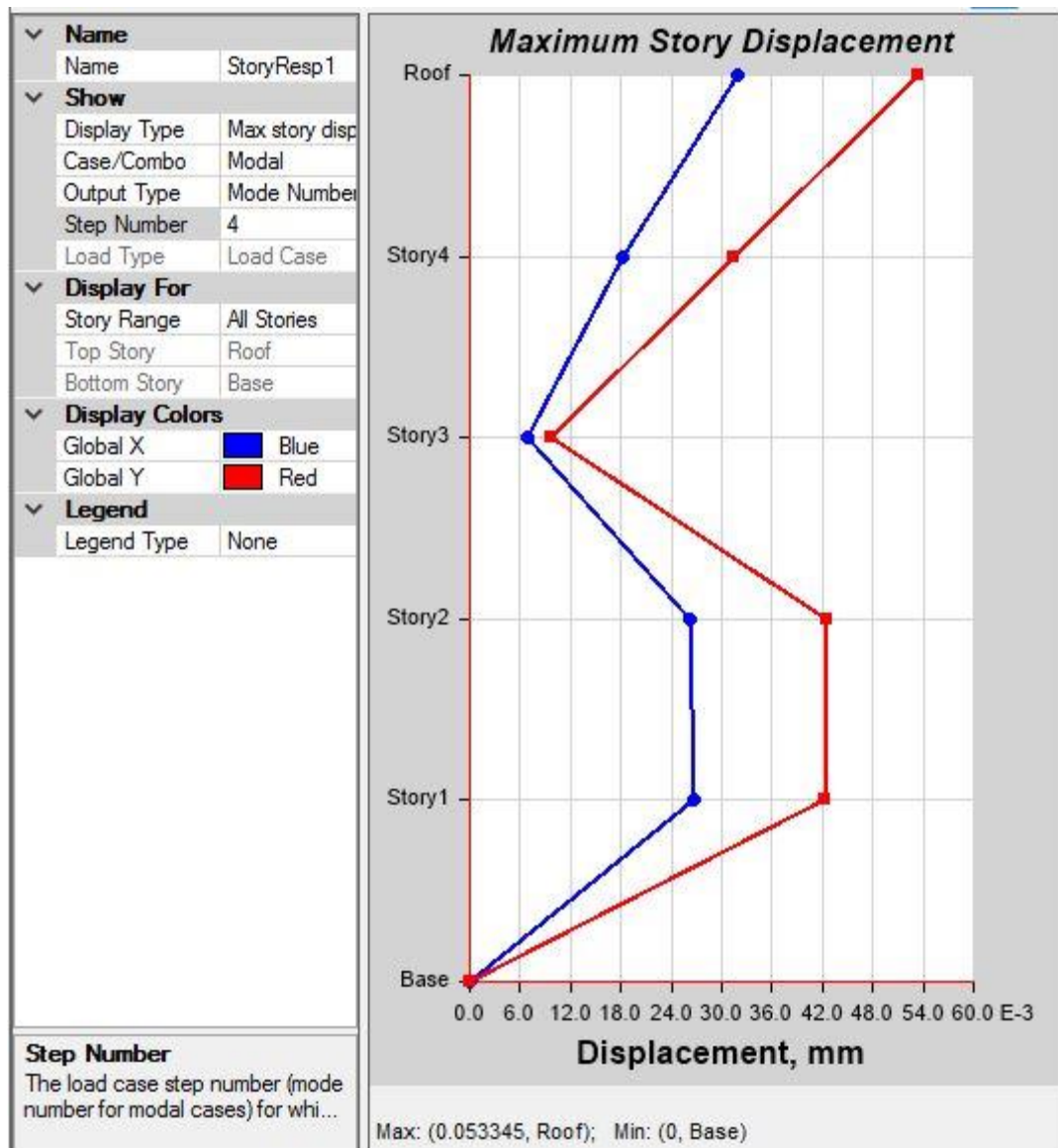


Figure B8: Displacement Plot of the Building for Mode 4

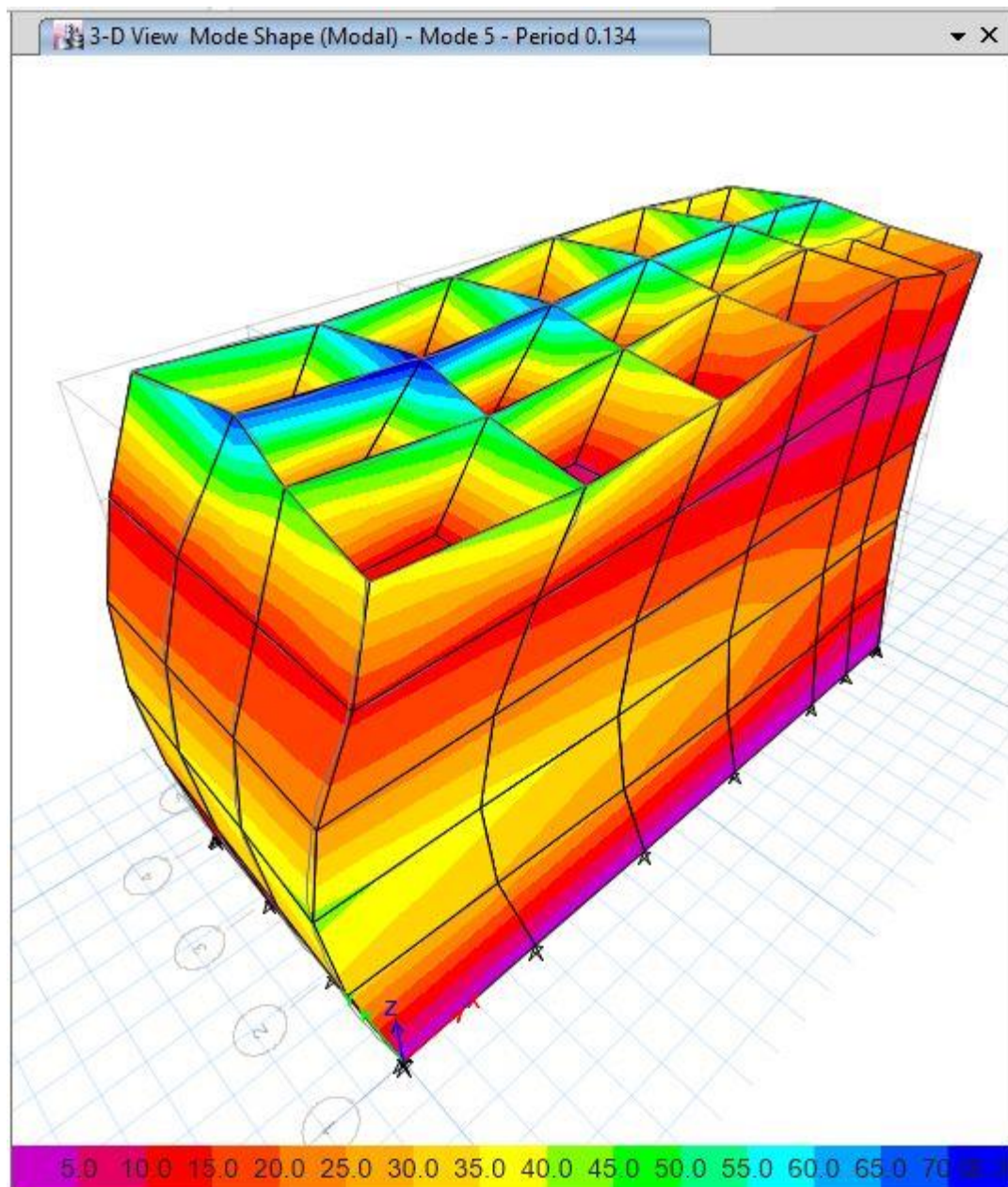


Figure B9: Mode 5 Building Reaction

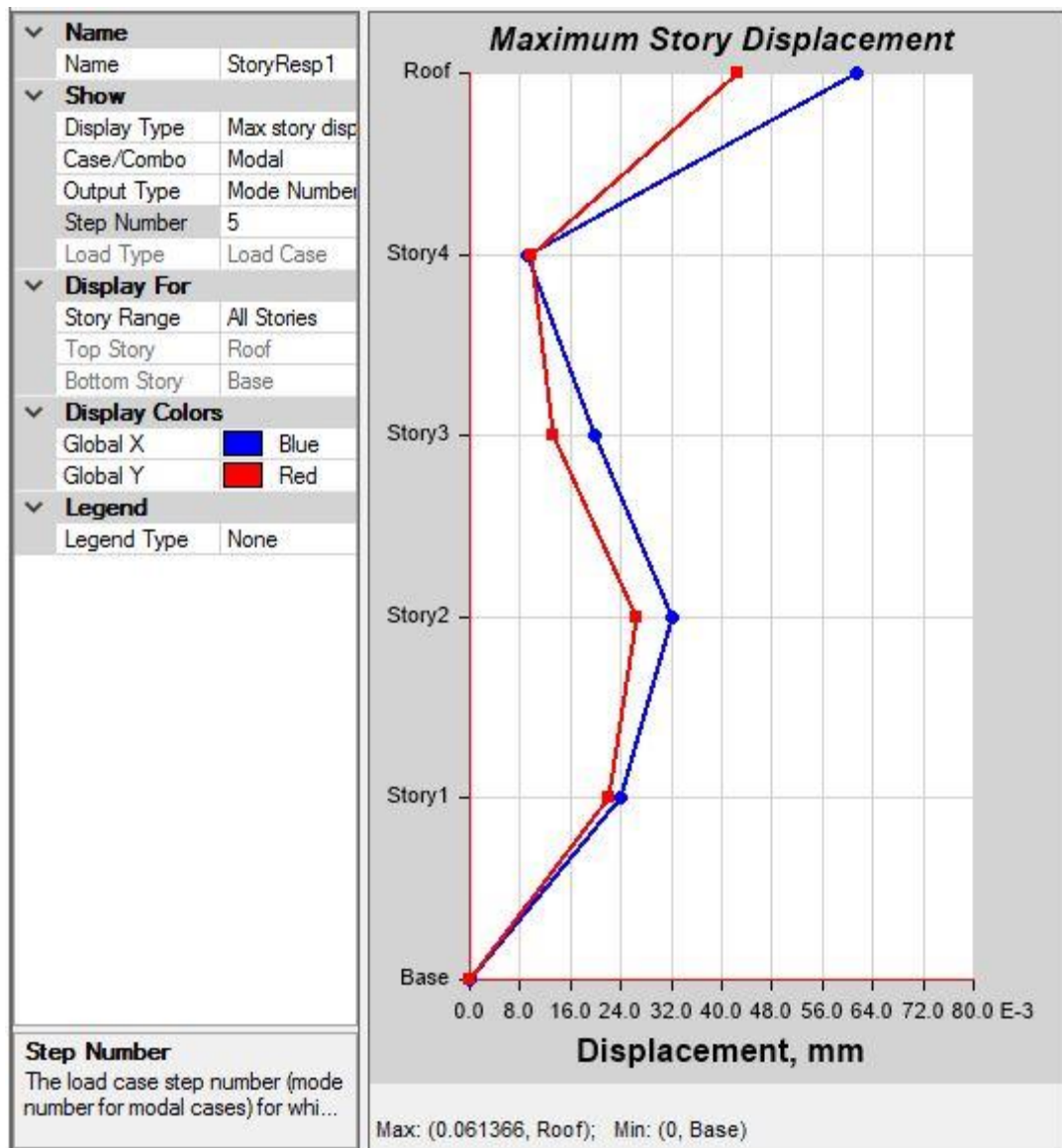


Figure B10: Displacement Plot of the Building for Mode 5

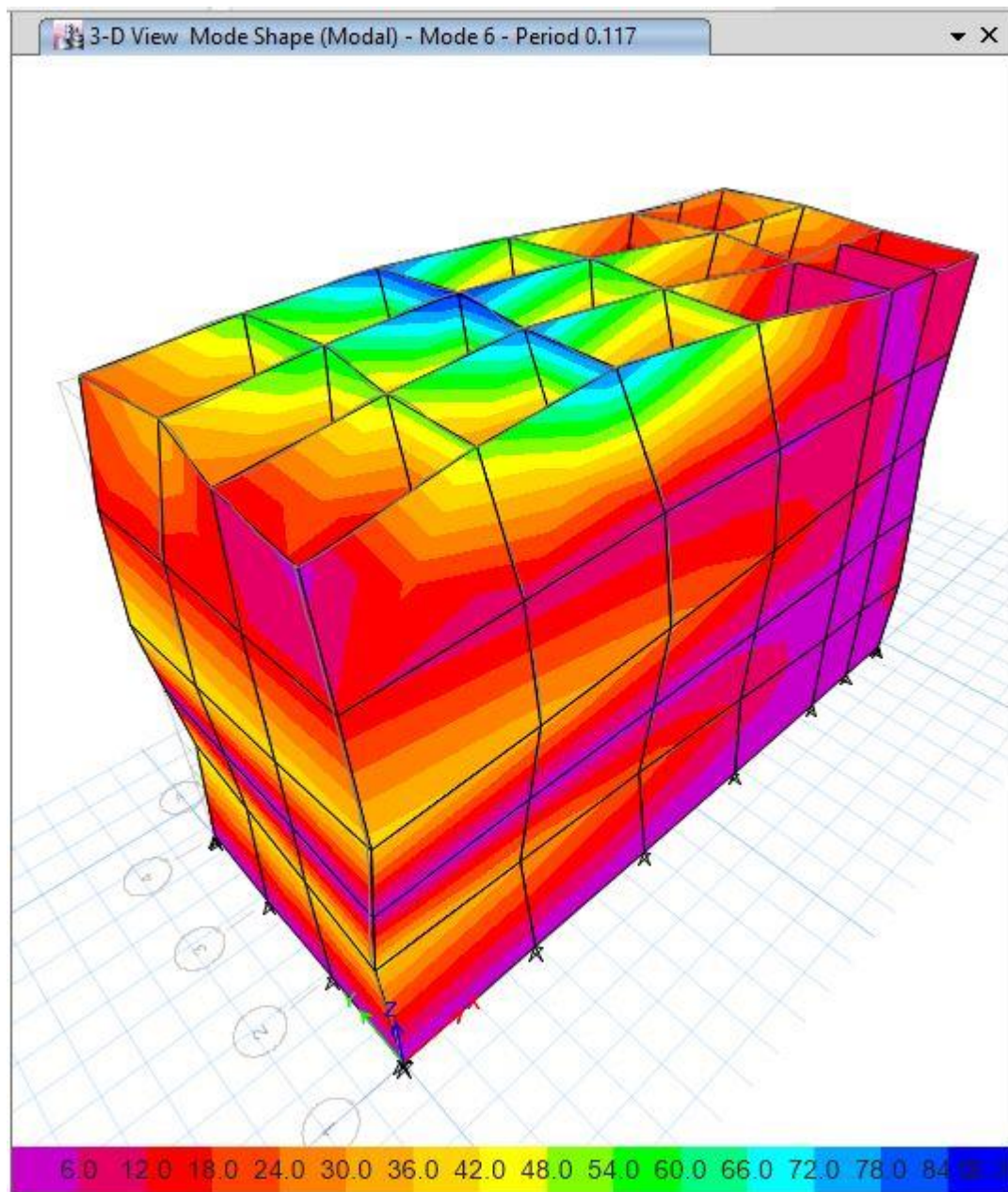


Figure B11: Mode 6 Building Reaction

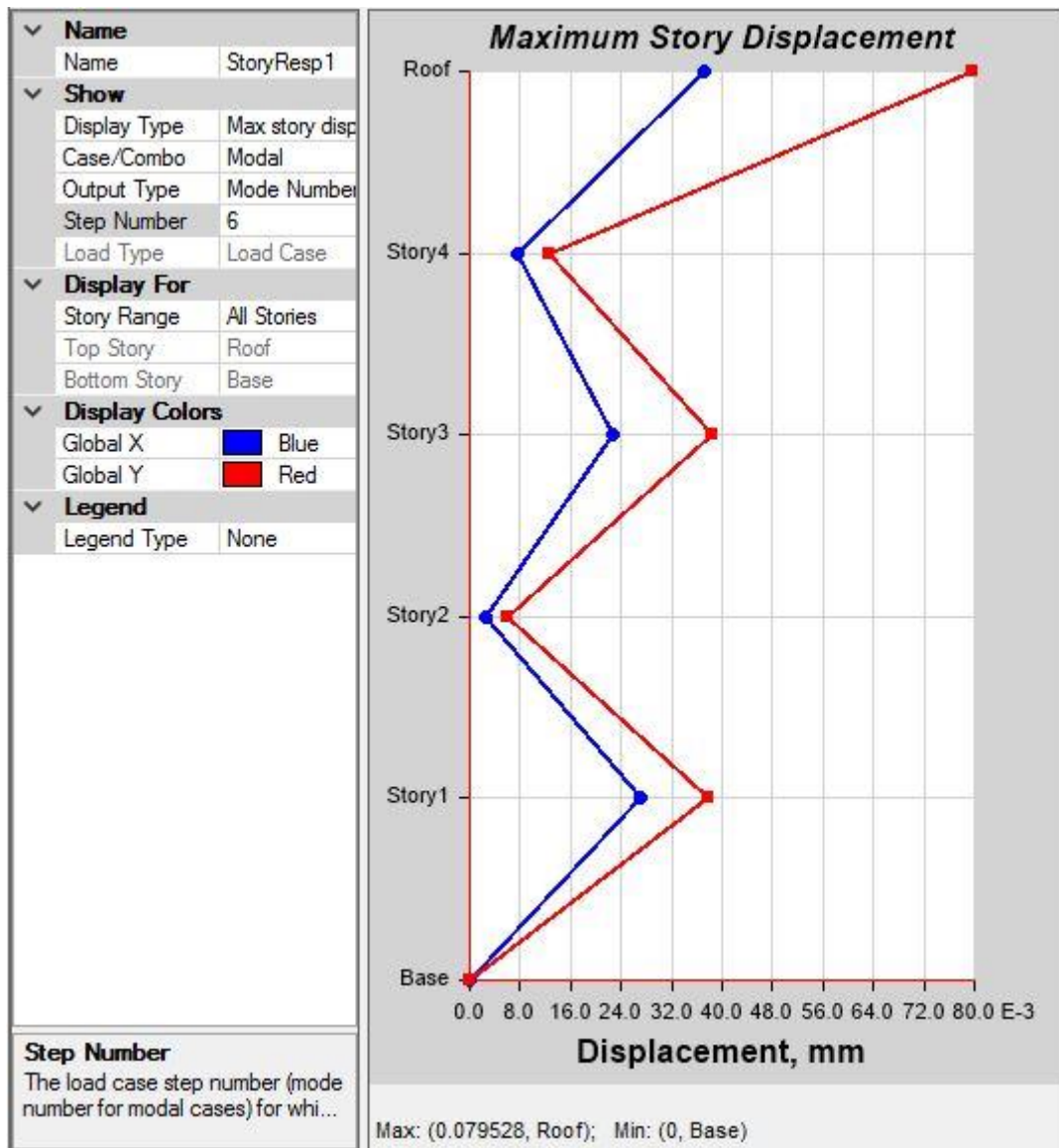


Figure B12: Displacement Plot of the Building for Mode 6

APPENDIX C

Storey Response Plots

1. Response Spectrum Analysis

Table C1: Storey Response (Displacement)

Story	Elevation	Location	X-Dir	Y-Dir
	M		mm	mm
Roof	15	Top	83.593	127.854
Story4	12	Top	77.946	120.394
Story3	9	Top	65.532	104.204
Story2	6	Top	48.69	80.259
Story1	3	Top	29.333	48.346
Base	0	Top	0	0

Table C2: Storey Response (Drift)

Story	Elevation	Location	X-Dir	Y-Dir
	M			
Roof	15	Top	0.003037	0.002718
Story4	12	Top	0.004501	0.005672
Story3	9	Top	0.005884	0.008177
Story2	6	Top	0.006723	0.010781
Story1	3	Top	0.009778	0.016115
Base	0	Top	0	0

Storey C3: Storey Response (shear)

Story	Elevation	Location	X-Dir	Y-Dir
	m		kN	kN
Roof	15	Top	2120.6696	1713.7084
		Bottom	2120.6696	1713.7084
Story4	12	Top	5666.8419	4673.2638
		Bottom	5666.8419	4673.2638

Story3	9	Top	8284.9754	6883.542
		Bottom	8284.9754	6883.542
Story2	6	Top	10030.8815	8372.219
		Bottom	10030.8815	8372.219
Story1	3	Top	10967.9076	9193.1722
		Bottom	10967.9076	9193.1722
Base	0	Top	0	0
		Bottom	0	0

Table C4: Storey Response (Stiffness)

Story	Elevation	Location	X-Dir	Y-Dir
	m		kN/m	kN/m
Roof	15	Top	268312.867	244650.542
Story4	12	Top	470564.856	383368.164
Story3	9	Top	563631.005	433125.518
Story2	6	Top	645117.151	439542.044
Story1	3	Top	586869.821	355844.924
Base	0	Top	0	0

Table C5: Storey Response (Overturing Moment)

Story	Elevation	Location	X-Dir	Y-Dir
	m		kN-m	kN-m
Roof	15	Top	213.6573	248.7465
Story4	12	Top	5749.6601	6319.5315
Story3	9	Top	20047.1806	23057.0277
Story2	6	Top	40792.6868	47487.1783
Story1	3	Top	65724.2558	76959.233
Base	0	Top	92852.9554	109205.0639