VULNERABILITY OF CYCLONE SHELTERS IN BANGLADESH DUE TO TSUNAMI

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ABSTRACT

Bangladesh is struck by cyclone and cyclonic surge almost every year. However, after the devastation caused by tsunami due to the great Sumatra Earthquake in 2004, tsunami is also considered as a probable hazard in the Asia Pacific region. Though tsunami did not cause any disaster in this country in the past but it can cause in future. So studies related to tsunami are now necessary mainly for the coastal region of Bangladesh.

The study is conducted considering tsunami loading only. Very few studies have been performed on tsunami in Bangladesh. The present study does not investigate whether tsunami may hit Bangladesh or not rather the effect of tsunami on structures is examined here. If tsunami strikes Bangladesh then people can survive only by taking shelter on places which is moderately higher than sea level. For this purpose, cyclone shelters in the coastal areas of Bangladesh may be suitable. The cyclone shelters, however, were not constructed considering tsunami load. So their vulnerability should be checked before using them as a safe shelter for tsunami.

Considering this fact, the study assessed the vulnerability of the existing cyclone shelters for tsunami. A number of three dimensional finite element models of the cyclone shelters were created for this study. Different heights of tsunami were assumed to identify the effect on different condition. Structure type, loading pattern for tsunami, loading combinations and the result of the analysis are provided in this project report.

The analysis was performed on the information available. Concrete strength of the existing structures was assumed 3000 psi as actual concrete strength was not available. Strength of reinforcing steel was assumed 40 ksi as design strength and actual strength data of steel was not available.

Structural details of these structures were not available so checking the adequacy of the members of the structures was difficult. To solve this problem it was assumed that 3% longitudinal reinforcement was provided in the columns of the structures. When the required steel reinforcement exceeded 3% the members were assumed vulnerable. A time history analysis was performed also but the time history data used was taken from a laboratory test result. It was performed as an experimental dynamic analysis. Foundations of the structures were not checked as these structures might have different types of foundations and foundation design related data were not available. Moreover different structures will need different types of foundation design depending on the soil condition of different sites.

From the study it is found that most of the structures are vulnerable in the analysis for tsunami, but this vulnerability is found depending on some assumptions due lacking of actual data. For tsunami, load was applied considering 1m, 2m and 3m tsunami height. Different types of cyclone shelters were found vulnerable for different heights. Only JICA type cyclone shelter survived up to 3m tsunami loading. For some structures column can survive 3m loading but beams are vulnerable due to torsion. Recommendations of further study are also discussed in this report.

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Abbreviations

ASCE	American Society of Civil Engineers
BDRCS	Bangladesh Red Crescent Society
BIDS	Bangladesh Institute of Development Studies
BNBC	Bangladesh National Building Code
BUET	Bangladesh University of Engineering Technologies
CDMP	Comprehensive Disaster Management Programme
CEGIS	Center for Environmental and Geographic Information services
FEMA	Federal Emergency Management Agency
GoB	Government of Bangladesh
IBC	International Building Code
LGED	Local Government Engineering Department
MCSP	Multipurpose Cyclone Shelter Program
NGO	Non-Government Organization
PWD	Public Works Departments
UBC	Uniform Building Code

Notations

E_m	Modulus of Elasticity of the masonry unit
E_c	Modulus of Elasticity of concrete
h_w	Clear ht of Column member
I_{col}	Moment of Inertia of the Column
Θ	Angle Produced by the strut with the horizontal
F_d	Hydrodynamic force (drag force)
ρ	Water density
C_d	Drag coefficient
d_s	Surge/Tsunami height
u	Flood velocity
g	Gravitational acceleration.

CHAPTER 1

INTRODUCTION

1.1 General

The coastal line of the northern part of the Bay of Bengal in Bangladesh is about 800 km (Khan, 2007). This region is always vulnerable to natural hazards. Its flat deltaic topography with very low elevation makes it more vulnerable to tsunami and surge.

The cyclone shelters are needed during a severe tropical cyclone for people who do not have proper accommodation to resist wind load and wind or water borne debris or who are evacuated from areas which may be inundated by the sea water due to tidal surge or flooding. To protect the coastal people from these havocs and to reduce the after effects, cyclone shelters are built. There are around 1800 cyclone shelters along the coast of Bay of Bengal. The construction of cyclone shelter started mainly after the independence since 1972. During the last 36 years these cyclone shelters are built by different government and non-government organizations. Some cyclone shelters are still being constructed in the coastal region. Generally these shelters are used as school, mosque, maktab or family welfare centers.

Hazards in the Bangladesh coastal zone include unavoidable risks to life and property generated by: coastal flooding, high winds, tidal waves, short-term and long-term shoreline erosion and storm surges. Each of these natural hazards creates a series of associated risks to coastal communities from hydrostatic and hydrodynamic forces on structures generated by coastal floodwaters and breaking waves, debris impacts, undermining of structures by scour and erosion and damage from high winds.

Recently a new threat is identified in the coastal region of Bangladesh which is tsunami. This is called new threat as this was not considered as a threat because it did not hit Bangladesh severely in the past and so research work on tsunami is infrequent here. Moreover the structures in the coastal region are not constructed considering the tsunami occurrence. So a study and research work in this topic will be helpful for designing safe structures in the coastal region.

Tsunami is described as a series of very long wavelength ocean waves caused by the sudden displacement of water by dynamic processes of the Earth such as plate tectonics, earthquakes, landslides, or submarine slumps and can also be triggered by anthropogenic activities such as dam breaks, avalanches, glacier calving or explosions (BUET, 2008).

There are significant differences in physical conditions between tsunami and other floods. Yeh et al., 2008 described that for a typical tsunami, the water surface fluctuates near the shore with amplitude of several meters during a period of a few to tens of minutes. This timescale is intermediate between the hours to days typical of riverine floods, and the tens of seconds or less associated with cyclic loading of storm waves. This intermediate timescale makes tsunami behaviors and characteristics quite distinct from other coastal hazards.

Tsunami differs from riverine flooding. In the case of tsunami inundation fluctuates faster, hence there is a higher potential to cause greater buoyant forces to be exerted on buildings; i.e. the water level outside may increase rapidly while the inside is still dry and empty.

On December 26, 2004 a magnitude 9.3 earthquake of Sumatra, Indonesia generated terrible tsunami (BUET, 2008). That tsunami crashed into the coast of India, Indonesia, Sri Lanka and some other countries. Bangladesh was relatively less affected by this tsunami. But this incident has raised the question whether and to what extent the country and its huge coastal population are vulnerable to tsunami hazard.

In this study the effect of tsunami on cyclone shelters will be considered. In the absence of reliable data on past events analysis will be performed assuming different heights of tsunami.

The analysis will be performed on the information available. Concrete strength of the existing structures is assumed 3000 psi and strength of reinforcing steel is assumed 40 ksi as actual strength data is not available.

Structural details of these structures were not available so checking the adequacy of the members of the structures is difficult. To solve this problem it is assumed that 3% longitudinal reinforcement was provided in the columns of the structures. When the required steel reinforcement exceeds 3% the members will be assumed vulnerable.

1.2 Objectives

The major objectives of this research are as follows:

- To create three dimensional finite element models of existing cyclone shelters and other structures situated in the coastal region for analysis.
- To study the vulnerability of cyclone shelters considering tsunami.
- To perform a time history analysis for tsunami.
- To identify effects of these loads on the cyclone shelters and check the vulnerability.

1.3 Methodology of work

All information related to particular building/cyclone shelter such as type of structures, adopted design criteria, construction etc. are collected. Data related to tsunami are collected. Tsunami load are computed using various available empirical formulas. Three dimensional finite element models of reinforced concrete frame buildings are developed from the collected information. The frame structure comprising columns and beams are modeled with beam column element and the slabs are modeled with shell elements. Lateral load from tsunami is applied to the structure. Static and dynamic analyses for tsunami load are performed. Results are analyzed and finally the adequacies of the structures are checked.

1.4 Organization of Thesis

The research work conducted for the achievement of the stated objectives is presented in this dissertation in several chapters organized in a way so that the steps implied in the study may properly delineate the methodology. This document is organized in five chapters and with some appendices and a list of references. This discourse addresses the more general and conceptual aspects of the methodology. A brief description of the contents of each chapter follows:

Chapter 1, **Introduction**, provides a statement of the purpose and scope of this thesis, followed by a brief description of the content of each of the chapter and a summary of supporting appendices.

Chapter 2, **Literature Review**, provides an abridged discussion on tsunami, soft story buildings, stiffness of infill walls and dynamic analysis.

Chapter 3, **Modeling and Analysis**, provides a detail description of finite element models and procedures of analysis.

Chapter 4, **Results and Discussions**, represents the results of the analysis.

Chapter 5, **Conclusions and Recommendations**, summarizes the outcome and recommends regarding future direction of this study.

Appendix A, entitled as Sample Calculation of Tsunami Load.

Appendix B, entitled as Sample Calculation of Strut Width.

Appendix-C: entitled as Pictures of Cyclone Shelters

Appendix-D: entitled as Plans of the Cyclone Shelters

Appendix-E: entitled as Beam and Column Dimension

CHAPTER 2

LITERATURE REVIEW

2.1 General

A tsunami is a special type of water wave generated by a disturbance in the ocean. The term "tidal wave" is sometimes used to describe a tsunami, but is misleading because these waves are independent of tides. Such disturbances in the ocean may be caused by earthquakes, submarine landslides, volcanic eruptions and explosions. Compared to other water waves, tsunamis are taller, faster, have long wavelengths, and have long periods. These characteristics give tsunamis greater energy than other water waves, and with it, the power to wreak havoc along the coastlines encountered.

2.2 Cause and Nature of Tsunami

According to Gulick et al., 2005, earthquakes are the most common cause of tsunamis. When an earthquake occurs at the seafloor, sections of the Earth's crust at the seafloor are displaced causing a sudden rise or fall in the sea surface level above. If the sea surface rises, then gravity pulls the elevated water back down to an equilibrium surface level. If, on the other hand, a depression is produced at the seafloor because a portion of the Earth's crust moves downward, gravity causes the surrounding water to flow into it. Both cases can generate waves with extremely long wavelengths (100 to 200 km) and long periods (10–20 minutes). Since the average depth of the ocean is about four kilometers (4000 meters), this depth is less than one twentieth the wavelengths of these waves. Tsunamis, therefore, behave like shallow water waves. Their speed is dependent on water depth because tsunamis 'feel' the seafloor, even in the open ocean. Tsunamis typically travel at speeds of 500km/hr or more across the ocean. Generally multiple waves are generated from one tsunami causing event and these waves may be several hours apart by the time they reach a coastline across the ocean from the point of origin.

Gulick et al., 2005 explained that the destructive nature of a tsunami wave on shore depends on the initial wave height in the ocean, the bathymetry (topography of the ocean bottom), the degree of slope to the continental shelf, wave velocity, and the shape of the shoreline. A channel may produce larger waves than a straight shoreline because less energy is dispersed. Likewise, densely varying bathymetry helps dissipate energy. In deep water the tsunami height may only be a few meters—hardly noticeable in the open ocean. As a tsunami approaches the shore its wave height increases dramatically. The length of the tsunami and its velocity decrease as the water shallows and the bottom of wave is slowed by friction with the seafloor. Wave energy, however, remains nearly the same. The front of the wave slows first while the back of the wave piles into it increasing the wave's height. Mathematically, wave energy is proportional to both the length of the wave and the height squared. Therefore, if the energy remains constant and the wavelength decreases, then the height must increase. As a rule-of-thumb, the height of a tsunami wave is about three times its height in deep water, but in some locations, such as parts of Hawaii this ratio of shallow water height to deep water height, known as the "run-up factor", may reach up to 40. That means a wave in deep water can increase to a 20-meter wave when it hits shore.

2.3 Multi-Hazard Cyclone Shelters and Tsunami Vulnerability in Bangladesh

The devastating cyclone of 29th April of 1991 tormented the life, economy and infrastructure of coastal areas of Bangladesh. That single cyclone made it clear like daylight that our country lacked disaster management. So, initiatives were taken to ameliorate the dire situation, a massive project was undertaken jointly by Government of Bangladesh, UNDP, World Bank. BUET was involved in the whole process and submitted the final report "Multi hazard cyclone shelter programme" on July 1993. The performance of the then existing shelters were reviewed. To overcome the shortcomings (structural and non- structural) of those shelters, some new designs were proposed. Now these buildings are standing along the coast of Bay of Bengal and saving lives of thousands of people. The most recent hurricane SIDR in 2007 struck the coastal belt with greater force than 1991, but the death toll was greatly reduced.

This is possibly due to a large increase of cyclone shelters, although it is still insufficient. During the review and design of the above mentioned cyclone shelters, the tsunami vulnerability was not assessed. So, the cyclone shelters may not be safe for tsunami.

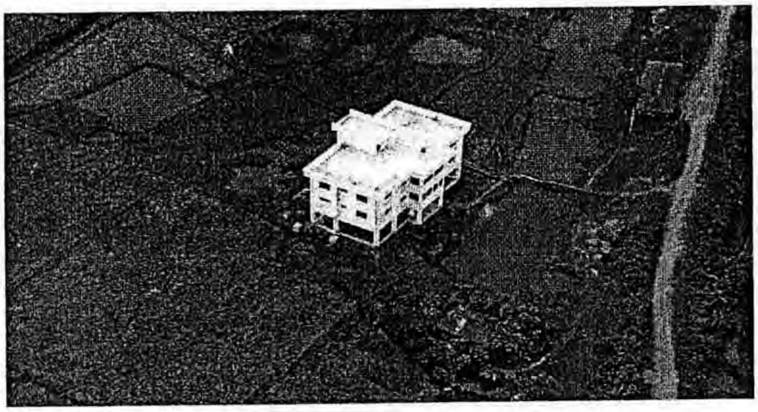

Figure 2.1 Aerial view of a typical cyclone shelter

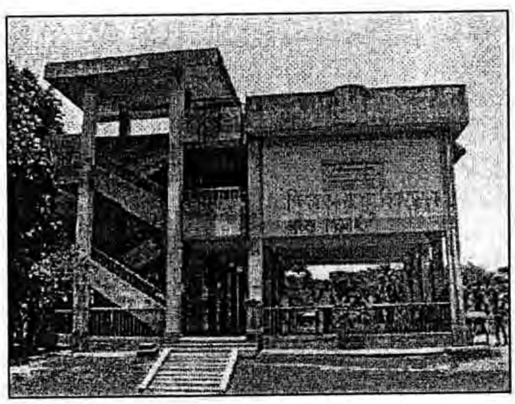

Figure 2.2 A typical cyclone shelter (European Union)

Cyclone shelters are supposed to withstand, in addition to its own weight and other imposed gravity loads, forces caused by cyclones which include forces induced by the strong wind and storm surge. Although cyclone shelters are built to resist forces caused by cyclones, they may as well be subjected to other forces of nature like tsunami and earthquake. The basic capacity of a structure depends on its planar geometric configuration, material properties, sectional properties of members, detailing, vertical irregularity etc. The basic capacity of the structure can be assessed by modeling the structure considering these factors and subjecting the model with appropriate forces which virtually simulate forces of nature.

Regarding loading aspect, detail information on how to consider regular gravity loads (dead load and live load) and lateral loads (wind load and earthquake load) are available in the BNBC, 1993. However, there is only a brief description of loads due to flood and surge and there is no mention of tsunami loading in the BNBC, 1993. BNBC suggests depth of inundation for calculation of hydrostatic forces due to flood and surge loads on structures at coastal areas.

Apart from BNBC, the 1997 Uniform Building Code (UBC 97) covers special construction topics in flood resistant construction. The 2000 International Building Code (IBC 2000) provides information of flood design and flood resistant construction. The ASCE 7-98 describes the different forces involved with flood and wave loads. The Federal Emergency Management Agency Coastal Construction Manual (FEMA CCM-2000) contains expressions for flood loads which include wave loads. In a report of Washington State Department of Natural Resources on design guidelines for tsunami vertical evacuation sites by Yeh et al., 2005 step by step procedure on how to consider hydrostatic forces, buoyant forces, hydrodynamic forces, surge forces, impact forces and breaking wave forces for tsunami loading is described.

Reviewing these literature appropriate loading has been selected for cyclone shelters. Linear analyses have then been conducted and appropriate clauses of Bangladesh National Building Code (BNBC, 1993) have been used to check the adequacy of the structural members.

2.4 Tsunami Load

During tsunami, different types of forces act on structures. Yeh et al. (2005) described these forces. These loads are discussed below :

Hydrostatic Forces

Hydrostatic forces occur when standing or slowly moving water encounters a building or building component. Hydrostatic loads can act laterally on an object. This load always acts perpendicular to the surface to which it is applied. It is caused by an imbalance of pressure due to a differential water depth on opposite sides of a structure or structural member.

Buoyant Forces

The buoyant or vertical hydrostatic forces on a structure or structural member subject to partial or total submergence will act vertically through the center of mass of the displaced volume. Buoyant forces are a concern for basements, empty above-ground and belowground tanks, and for swimming pools. Any buoyant force on an object must be resisted by the weight of the object and any opposing force resisting flotation.

Hydrodynamic Forces

When water flows around a building (or structural element or other object) hydrodynamic loads are applied to the building. These loads are a function of flow velocity and structure geometry, and include frontal impact on the upstream face, drag along the sides, and suction on the downstream side. These loads are induced by the flow of water moving at moderate to high velocity. They are usually called the drag forces, which are combination of the lateral loads caused by the impact of the moving mass of water and the friction forces as the water flows around the obstruction.

Surge Forces

Surge forces are caused by the leading edge of a surge of water impinging on a structure. The surge force is computed as a force per unit width on a vertical wall subjected to a surge from the leading edge of a tsunami.

Impact Forces

Impact loads are those that result from debris such as driftwood, small boats, portions of houses, etc., or any object transported by floodwaters, striking against buildings and structures or parts thereof. The magnitude of these loads is very difficult to predict, yet some reasonable allowance must be made for them. The velocity of waterborne objects is assumed to be the same as the flood velocity. The object is assumed to be at or near the water surface level when it strikes the building.

Therefore, the object is assumed to strike the building at the water level. Uncertainty about the duration of the impact time is the most likely cause of error in the calculation of debris impact loads. The duration of impact is influenced primarily by the natural frequency of the building, which is a function of the building's "stiffness." This stiffness is determined by the properties of the material being struck by the object, the number of supporting members (columns or piles), the height of the building above the ground, and the height at which the building is struck.

Breaking Wave Forces

Two breaking wave load conditions are of interest in construction; waves breaking on small-diameter vertical elements (e.g., piles, columns in the foundation of a building in V zones) and waves breaking against walls (e.g., breakaway walls in V zones). Breaking wave forces are modified in instances where the walls or surfaces upon which the breaking waves act are non-vertical. Breaking waves that incident obliquely and not perpendicular to the wall result in a lower force. The net force resulting from breaking wave acting on a rigid vertical pile or column is assumed to act at the still water elevation. A wave breaking against a vertical wall causes a reflected or standing wave to form against the seaward side of the wall. The crest of the wave is some height above the still water elevation. Two cases are considered: (1) where a wave breaks against a vertical wall of an enclosed dry space, and (2) where the still water level on both sides of the wall is equal. Case 1 is equivalent to a wave breaking against an enclosure in which there is no floodwater below the still water level. Case 2 is equivalent to a wave breaking against a wall with openings that allow floodwaters to equalize on both sides of the wall.

2.5 Soft Storied Structures

In this article recognition of soft story in different building codes are investigated. Then the mechanism of soft story and their failure patterns are discussed in the light published materials.

2.5.1 Provision of Soft Story in Building Codes

When a sudden change occurs in stiffness along the building height, the story at which this drastic change of stiffness occurs is called a soft story. According to BNBC, 1993, a soft story is the one on which the lateral stiffness is less than 70% of that in the story above or less than 80% of the average stiffness of the three stories above. In IBC 2000, an extreme soft story is one in which the lateral stiffness is less than 60% of that in the story above or less than 70% of the average stiffness of the three stories above. The vertical geometric irregularity shall be considered to exist where the horizontal dimensions of the lateral-force-resisting systems in any story is more than 130% of that in an adjacent story.

BNBC 93 does not provide any rigorous procedure to account for soft story phenomenon. According UBC 1997 the allowable story drift is .005. If this limitation exceed for any floor then that particular floor forms a soft story. Researchers are working to find out suitable measure to limit the story drift within acceptable value.

2.5.2 Failure Pattern of RC Buildings with Soft Story

Masonry infill (MI) walls confined by reinforced concrete RC frames on all four sides play a vital role in resisting the lateral seismic loads on buildings. It has been shown experimentally that MI walls have a very high initial lateral stiffness and low deformability (Moghaddam and Dowling 1987). Thus introduction of MI in RC frames changes the lateral-load transfer mechanism of the structure from predominant frame action to predominant truss action (Murty and Jain 2000), which is responsible for reduction in bending moments and increase in axial forces in the frame members. In addition, construction of MI is cheaper because it uses locally available material and labor skills. Moreover, it has good sound and heat insulation and waterproofing properties, resulting in greater occupant comforts and economy. Buildings can become irregular in plan and elevation because of uncertain position of MI walls and openings in them. Often MI walls are rearranged to suit the changing functional needs of the occupants, the changes being carried out without considering their adverse effects on the overall structural behavior because MI walls are generally regarded as nonstructural elements of buildings. MI can be distributed in RC frames in several patterns, for example, as shown in Figure 2.3 Thus it is not only difficult to construct a regular MI-RC frame building, but also it cannot be taken for granted that it will remain regular after it is constructed.

Depending on relative properties of frame and infill, failure modes of masonry infilled frame show variety. In other words, failure can occur in the frame elements or in the infill. In estimating the lateral strength and lateral stiffness of masonry infilled frame, it is necessary to find the most critical of the various modes of failure of the frame and infill. The usual modes for frame failure are tension failure of surrounding column

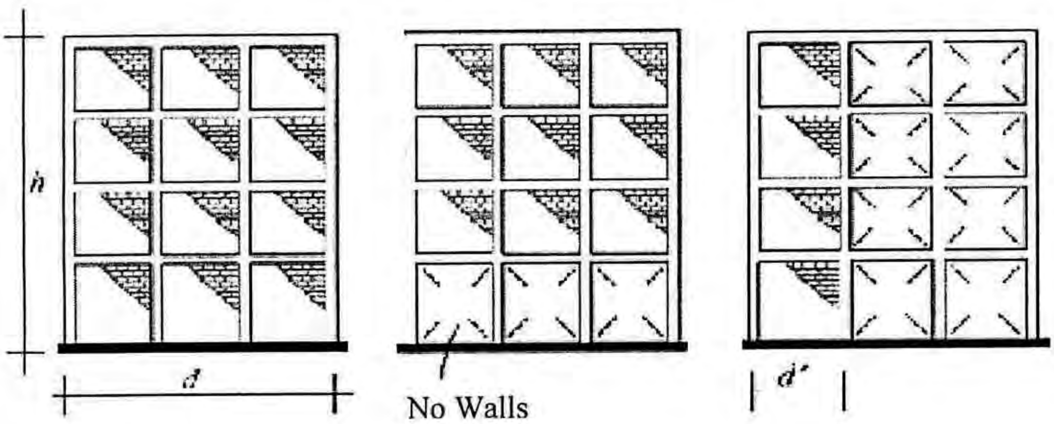

Figure 2.3 Different arrangements of masonry infill walls in RC frame. (Kaushik et al., 2006)

elements or shear failure of the columns or beams. These modes are given in Figure 2.4 (Smith and Coull, 1991). Tension failure of the column results from applied overturning moments. Such mode may be critical one in infilled frames with high aspect ratio and with very rigid frame elements.

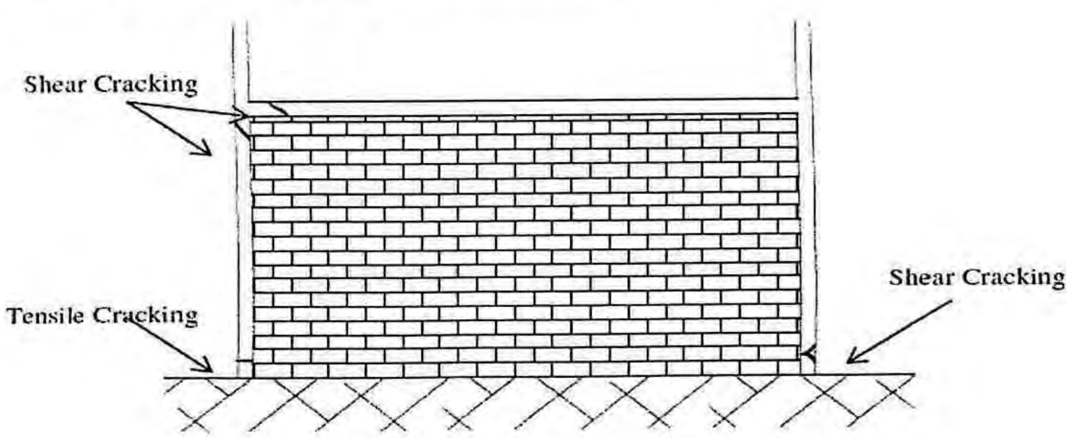

Figure 2.4 Failure mode of masonry infill wall

The tension steel acts as a flange of the composite wall. However, in case of weak frame element, dominant modes of failures are flexural or shear failure of column or beams at plastic hinge locations. However, if the frame strength is enough to withstand, increasing lateral load results in failure of infill. In addition to that, the failure may be a sequential combination of the failure modes of frame and infill. For example, flexural or shear failure of the columns will generally follow a failure of infill. In both case, failure modes of infill show variety depending on geometric and material properties. Failure of the infill occurs by one of the following modes;

a) Shear cracking along the interface between the bricks and mortar

b) Tension cracking through the mortar joints and masonry

c) Local crushing of the masonry or mortar in compression corner of the infill.

Shear failure of infill is directly related with the horizontal shear induced in the infill panel by applied load. In addition to applied load, shear resistance of masonry plays an important role. The resistance of masonry to shearing stress is usually considered to be provided by the combined action of the bond shear strength and the friction between the masonry and mortar. Also, vertical compressive stress level induced in infill panel by applied load is important. When a vertical compressive stress is applied to masonry the shear resistance is increased with the increase of friction between the masonry and the mortar. However, friction effect is less effective for the case of perforated brick. Test results (Polyakov, 1956) showed that for perforated brick the coefficient of internal friction about 0.15, while it varies between 0.6 to 1.7 for solid brick.

Diagonal tension cracking is the result of the diagonal force which produces a principle tensile stress in the infill equal to tensile strength of the infill material (Smith and Carter, 1969) derived the lateral force cause diagonal crack on infill in terms of contact length between frame and infill under the light of their experimental results. This relation showed that greater value of the length to height ratio of infill or smaller value of λh (stiffer column relative to the infill) result in greater diagonal strength of infill. Compressive failure of infill is accompanied by a rapidly increasing rate of deflection. Therefore, it can be said that compressive failure is a plastic type of infill failure. As done for diagonal tension failure, compressive failure load is related with the contact length between frame and infill (Smith & Carter, 1969) according to experimental results. The result of this relation can be concluded as follows; smaller value of λh results in greater compressive strength of infill. This can be explained with that stiffening of column leads to the reduction in lateral deflection. And stiffer column means smaller value of λh. However, because of the weakness of the shearing and tensile modes relative to the compressive failure mode, it is thought that a compressive failure would be unlikely to occur in brickwork.

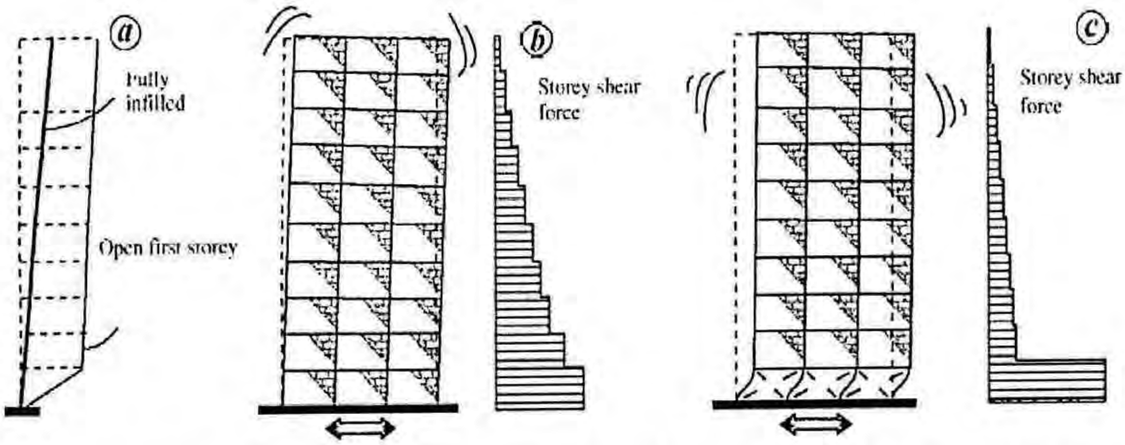

Figure 2.5 Effects of masonry infills on the first mode shape of a typical frame of a ten-storey RC building. a, Displacement profile;b, Fully infilled frame; c, Open first storey frame.(Kaushik et al., 2003)

In the case of a fully infilled frame, lateral displacements are uniformly distributed throughout the height as shown in Figure 2.5 *a* and *b*. On the other hand, in the case of open first storey buildings, most of the lateral displacement is accumulated at the first storey level itself because the first storey is the most flexible due to absence of infills (Figure 2.5 *c*). Similarly, the seismic storey shear forces and subsequently the bending moments concentrate in the open first storey, instead of gradually varying as in fully infilled frame (Figure 2.5 *c* and *b*).

(a) Shear failure of columns (b) Excessive drift (c) Reversed axial force due to overturning moment

Figure 2.6 Cause of collapse of RC buildings with soft story (Fukuyama et al., 2004)

Figure 2.6 shows typical failure patterns of RC buildings with soft story. This investigation was carried out by Fukuyama et al. (2004) in Japan after the Kobe earthquake. The Figure 2.6 (a) shows shear failure of a column, which was observed frequently at the previous earthquakes in the RC buildings with soft story designed according to the previous code.

Fukuyama et al. (2004) stated that this type of failure could be prevented in the buildings designed according to the current code in Japan. However, other types of failure of the soft story were observed in the disaster caused by the 1995 Kobe Earthquake. Those are collapse due to excessive drift of the soft story, mainly caused by the lack of story shear capacity, as shown in Figure 2.6 (b), and collapse caused by the reversed axial force due to large overturning moment as shown in Figure 2.6 (c). In case of Figure 2.6 (c), buckling of the longitudinal steel bars under the compressive axial force, which have yielded by the tensile axial force firstly, was observed. The bars may rupture if the large tension forces act after buckling. Thus not only shear failure of the columns but excessive story drift and/or excessive axial force of the columns should be prevented for meeting the structural safety requirement.

2.5.3 Characteristics of Infilled Frame

A number of researchers throughout the world have been conducting investigations in determining the behavior of masonry infilled frame for a long time in attempts to develop an appropriate design method. The use of a masonry infill to brace a frame combines some of the desirable structural characteristics of each, while overcoming some of their deficiencies. The high in-plane rigidity of the masonry wall significantly stiffens the otherwise relatively flexible frame, while the ductile frame contains the brittle masonry, after cracking, up to loads and displacements much larger than it could achieve without the frame. The result is, therefore, a relatively stiff and tough bracing system.

The wall braces the frame partly by its in-plane shear resistance and partly by its behavior as a diagonal bracing strut in the frame. This behavior is shown in Fig. 2.7.

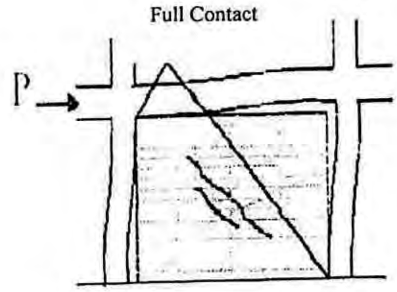

Figure 2.7 Behavior of masonry infill panel

When the frame is subjected to horizontal loading, it deforms with double-curvature bending of the columns and beams. The translation of the upper part of the column in each storey and the shortening of the leading diagonal of the frame cause the column to lean against the wall as well as to compress the wall along its diagonal. It is roughly analogous to a diagonally braced frame, shown in Fig. 2.8.

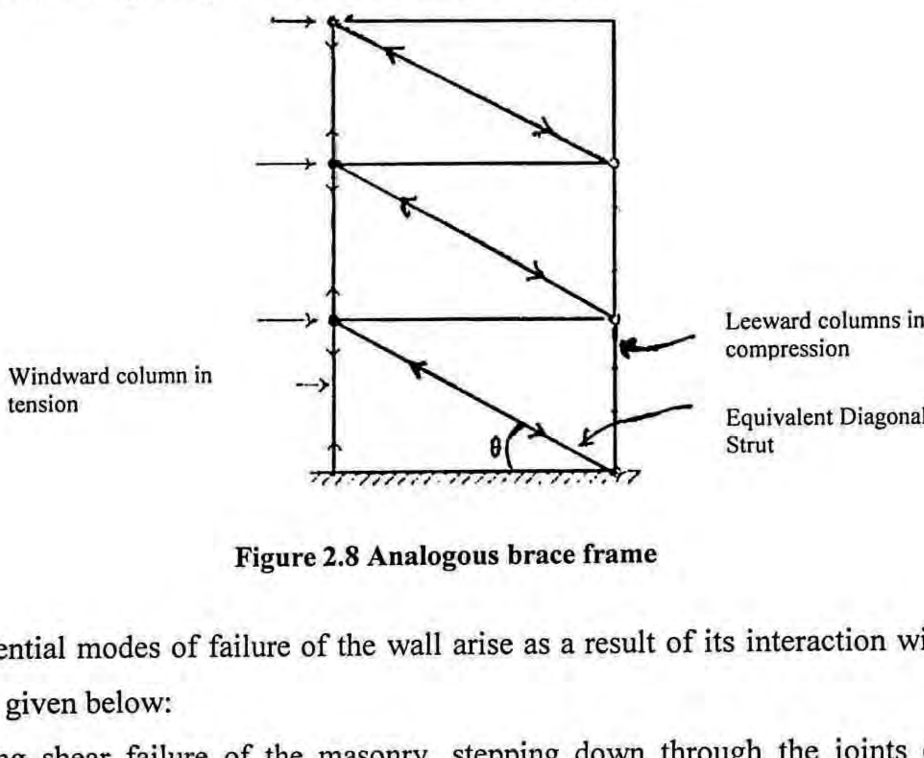

Figure 2.8 Analogous brace frame

The potential modes of failure of the wall arise as a result of its interaction with the frame is given below:

1. Sliding shear failure of the masonry, stepping down through the joints of the masonry along horizontal mortar beds.
2. Diagonal tension cracking of the panel.
3. Compression failure of the diagonal strut.
4. Flexure shear failure of the columns and
5. Tension failure of the tension column due to overturning moments.

2.6 Analysis of Infilled Frame

A number of researchers attempts at the analysis and design of infilled frames under lateral load since the mid-1950s have lead to several methods. Holmes (1961),Stafford Smith (1962, 1966, 1969), Mainstone and Weeks (1970), Me Bride (1984), Yong (1984), Amos (1986), and Richardson (1986) conducted experimental and analytical investigations of the lateral stiffhess and strength of steel frames infilled with mortar and concrete panels. Dawe and Seah (1989), Flanagan et al. (1999), and Mander et al. (1993) studied the behavior of masonry infill steel frames under in-plane and out-of-plane loads. Dhanasekaif and Page (1986) developed finite element models of masonry infilled steel frames. Saneinejad and Hobbs (1995) developed equivalent diagonal strut method to predict the strength and stiffness of infilled steel frames as well as infill diagonal cracking load.

Klinger and Bertero 1978; Bertero and Brokken 1983; Zarnic 1990; Mander and Nair 1994 focused on evaluating the experimental behavior masonry infilled frames to obtain the formulations to limit strength and equivalent stiffness. The behavior of masonry infilled R.C frames, which is generally more complicated than that of infilled steel frames, has also been examined by Fiorato et al. (1970), Kahn and Hanson(1979), Zarnic and Tomazevic (1990), Murty et al. (2000) and recently by Ghosh and Amde (2002), Al- Chaar (2002). Both experimental and analytical studies have been carried out by Mehrabi et al. (1994, 1996, and 1997) to investigate the performance of masonry infilled R.C frames under in- plane lateral loadings under different design conditions. Manos et al. (2000) experimentally investigated the influence of masonry infills on the earthquake response of multi-storey R.C frames and Haque (2002) investigated the effect of randomly distributed infills on the vibration characteristics of reinforced concrete frames. Dey (2000) and Waset (2002) worked on sway characteristics on masonry infilled R.C building under lateral load and A. Shahriar (2004) worked on numerical study of R.C building frame-infill interaction.

2.6.1 The Equivalent Diagonal Strut Method

Saneinejad and Hobbs (1995) developed a method based on the equivalent diagonal strut approach for the analysis and design of steel or concrete frames with concrete or masonry infill walls subjected to in-plane forces. The method takes into account the elastoplastic behavior of infilled frames considering limited ductility of infill materials. Various governing factors such as the infill aspect ratio, the shear stresses at the infill frame interface, and relative beam and column strengths are accounted for this development. The proposed analytical development assumes that the contribution of the masonry infill panel shown in Fig. 2.9 to the response of the infilled frame can be modeled by "replacing the panel" by a system of two diagonal masonry compression struts shown in Fig. 2.10.

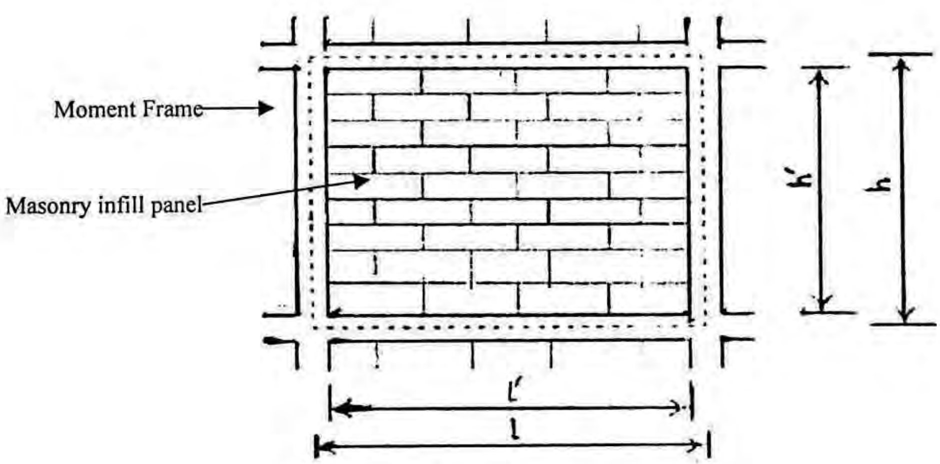

Figure 2.9. Masonry infill panel.

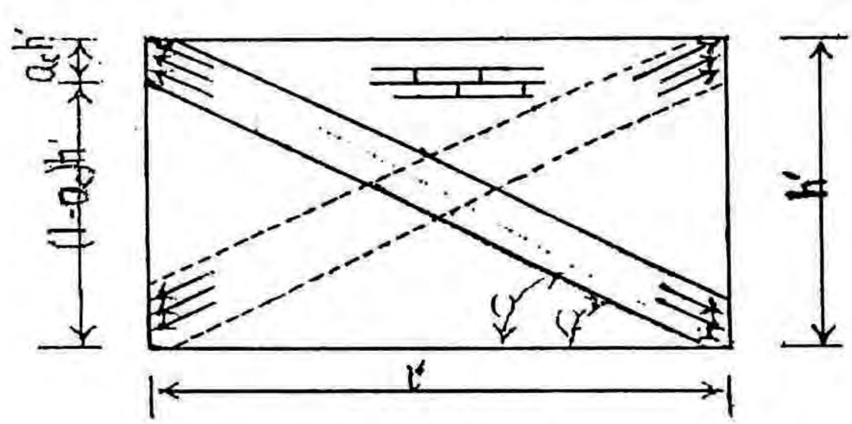

Figure 2.10 Equivalent strut model.

2.6.2 Calculation of Equivalent Strut Area

The expressions used in this chapter have been adopted from Mainstone (1971) and Stafford-Smith and Carter (1969) for their consistently accurate predictions of infilled frame in-plane behaviour when compared with experimental results. The masonry infill panel will be represented by an equivalent diagonal strut width, a, and net thickness, t, as shown in Fig. 2.12.

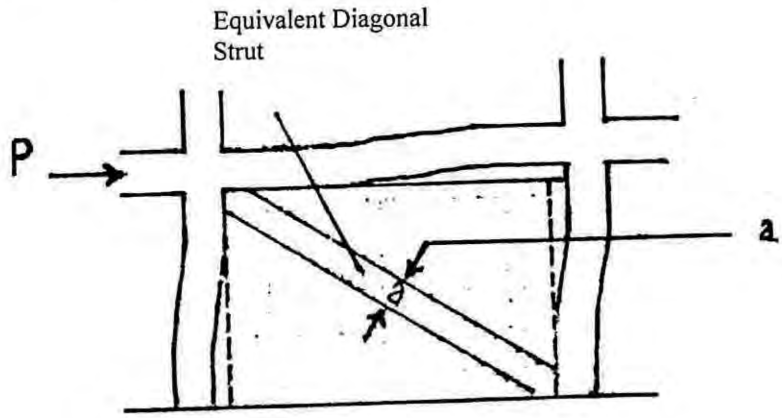

Figure 2.11 Equivalent Diagonal Strut

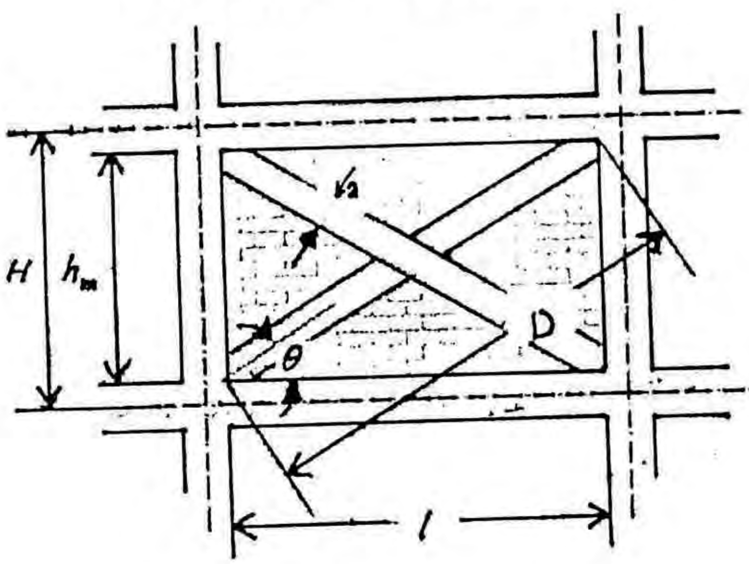

Figure 2.12 Strut Geometry

The equivalent strut width "a" depends on the relative flexural stiffness of the infill to that of the columns of the confining frame. The relative infill-to-frame stiffness shall be evaluated using Eq.2.1 (Stafford-Smith and Carter 1969):

$$\lambda_1 H = H\left(\frac{E_m t \sin 2\theta}{4E_c I_{col} h_w}\right)^{1/4} \quad \text{Eq.2.1}$$

Using this expression, Mainstone (1971) considered the relative infill-to-frame flexibility in the evaluation of the equivalent strut width of the panel as shown in below:

$a = 0.175D(\lambda_1 H)^{-0.4}$

Where

A = Equivalent Strut width
t = Thickness of the masonry infill
E_m = Modulus of Elasticity of the masonry unit
E_c = Modulus of Elasticity of concrete
h_w = Clear ht of Column member
I_{col} = Moment of Inertia of the Column
Θ = Angle Produced by the strut with the horizontal
D = Diagonal

2.7 Linear Dynamic Analysis

Linear dynamic analysis means Response spectrum analysis. In this case the load is dynamic and the material properties are linear. Response spectrum analysis is the procedures to compute the peak response of a structure during an earthquake directly from the earthquake response (or design) spectrum without the need for response history analysis. This method is not an exact predictor of peak response, but it provides an estimate that is sufficiently accurate for structural design application.

John R. Freeman (1855-1932) played a key role in the early development of earthquake engineering. He was principally responsible for the development and installation of the first strong motion accelerographs, originally called the Montana Accelerograph, in 1932 (Reitherman, 1997).

The concept of "response spectra" was developed in the 1930s, but it wasn't until 1952 that a joint committee of the San Francisco Section of the ASCE and the Structural Engineers Association of Northern California (SEAONC) proposed using the building period (the inverse of the frequency) to determine lateral forces.

The University of California, Berkeley was an early base for computer-based seismic analysis of structures, led by Professor Ray Clough (who coined the term finite element). Students included Ed Wilson, who went on to write the program SAP in 1970, an early "Finite Element Analysis" program.

Since the mid seventies researchers are working relentlessly to improve the accuracy of earthquake analysis with increasing availability and ability of personal computers. A few of those efforts are discussed here.

Wilson et al. (1982) implied direct superposition of Ritz vectors for dynamic analysis. It diminished the computational effort and led to enhancement analysis procedure. (Wilson et al., 1982)

Burdisso and Singh (1986) developed a response spectrum procedure for seismic analysis of multiply supported secondary systems. The formulation was based on the random vibration analysis of structural systems subjected to correlated inputs applied at several supports. For a proper response spectrum analysis of a multiple support system, the support inputs are required to be defined in terms of the auto and cross pseudo-acceleration and relative velocity floor response spectra.

Kiureghian and Neuenhofer (1992) developed a new response spectrum method for seismic analysis of linear multi-degree-of-freedom, multiply supported structures subjected to spatially varying ground motions. Variations of the ground motion due to wave passage, loss of coherency with distance and variation of local soil conditions were included. The method was based on fundamental principles of random vibration theory and properly accounts for the effects of correlation between the support motions as well as between the modes of vibration of the structure.

2.7.1 Modal Analysis

Modal analysis calculates vibration modes for the structure. These can be used to investigate the behavior of a structure, and are required as a basis for subsequent response-spectrum and/or time-history analyses. Two types of modal analysis are available: eigenvector analysis and Ritz-vector analysis. Only one of these can be used in a single analysis run.

Eigenvector analysis determines the undamped free-vibration mode shapes and frequencies of the system. These natural Modes provide an excellent insight into the behavior of the structure.

Wilson et al. (1982) explained that they can also be used as the basis for response-spectrum or time-history analyses, although Ritz vectors are strongly recommended for this purpose.

2.7.2 Time History Analysis

Time history analysis is the procedures to compute the peak response of a structure using a loading or acceleration curve or data. This method provides an estimate that is sufficiently accurate for structural design application.

2.8 Conclusion

After reviewing available literature it is found that very few studies have been performed to assess the tsunami vulnerability of cyclone shelters. This study will be very significant in this field.

CHAPTER 3

MODELING AND ANALYSIS

3.1 General

The purpose of this chapter is to represent the modeling and analysis procedure that has been employed to carry out this investigation. First, a brief description of the finite element package which has been used in this work is presented. Then the plans and elevations of the cyclone shelters are presented at first. This is followed by the explanation of modeling procedure. The last portion is deals with the finite element analysis procedure.

3.2 The FEM Software

Linear 3D FEM analyses of the selected buildings have been conducted in order to assess the capacity of the structures to withstand loading caused by tsunami. A wide spectrum of FEM software and tools are available for such analyses. In the present study, ETABS (Version 9.0.4) has been used for this purpose. ETABS has been chosen for its user friendly features in analyzing building structures. The special features of ETABS are mentioned below:

- User friendly graphical user interface for generating story-wise building models
- Availability of necessary elements for developing FEM model of a building
- Provision for changing orientation of frame elements
- Automatic consideration of rigid end zones of frame elements
- Automatic calculation of member self-weight
- Automatic calculation of member sectional properties
- Integrated design features

3.3 Multipurpose cyclone shelters

Studying the report "Multipurpose Cyclone Shelters Programme" prepared by BUET and BIDS (1993), it is found that the following are various types of cyclone shelters now existing in our country.

- PWD type
- BDRCS shelters.
- Facilities department type cyclone shelters.
- Type A, Type B, Type C
- Type D, Type E, Type F
- Type G, Type H, Type J, Type K, Type L, Type L

Some other types of cyclone shelters are also available which are not mentioned in the report. Those are–

- European union
- JICA
- LGED
- Saudi
- CDSP-2
- LGED-2
- Grameen Bank
- German

Among the above mentioned shelters, some of the shelters are so old and poor that they are not worthy of assessment. Some of the shelters lack sufficient data needed for satisfactory modeling. These types of numerous shelters are standing on the coast of Bay of Bengal. Following types are analyzed in this study.

- European union
- JICA
- LGED
- Saudi
- CDSP-2
- LGED-2

- Grameen Bank
- German
- BDRCS
- Type D
- Type E
- Type J
- PWD
- College Building

3.4 Finite Element Modeling

Frame elements have been used to model the columns and plate/shell elements have been used to model the slabs. The frame elements are typical two-noded elements in space having six degrees of freedom per node – three translations and three rotations in three mutually perpendicular axes system. The plate/shell elements are of rectangular (or quadrilateral) and triangular shape. The quadrilateral element has four nodes at its four corners. Each node has six degrees of freedom – three translations and three rotations in a 3D space configuration.

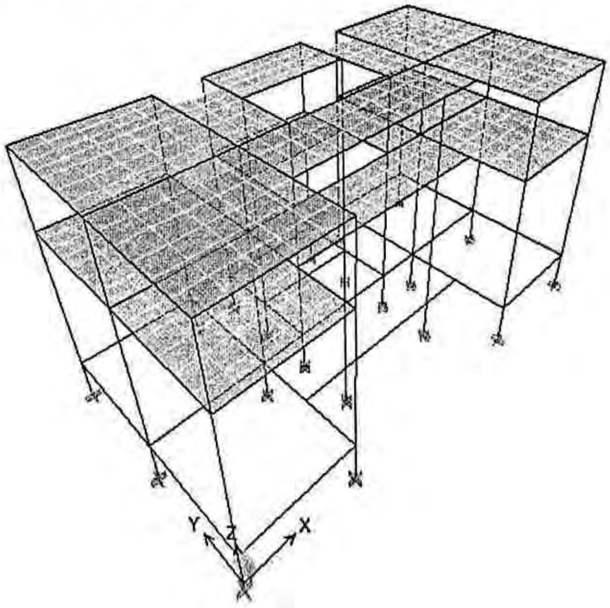

Figure 3.1: Type E bare frame model

Each regular slab has been meshed with 4 by 4 grid of plate elements. For irregular shapes triangular elements have been used when necessary. At the base level, the columns are assumed to be held fixed. Some models are made from the drawings of the report "Multipurpose Cyclone Shelters Programme" prepared by BUET and BIDS (1993). Other models are made from collected drawings from different departments. 3D views of the Finite Element Models of a structure are shown in **Figures 3.1 and 3.2.**

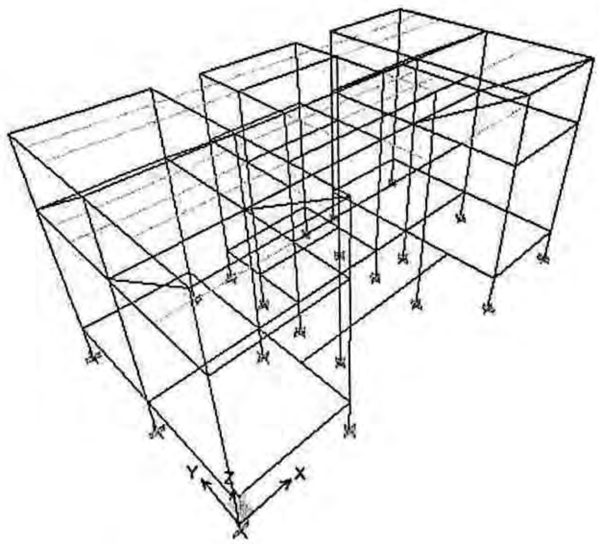

Figure 3.2: Type E story model with bracing

3.4.1 Materials

The material properties of the structure and structural components are linearly elastic. This assumption follows the superposition of forces and deflections and, hence the use of their linear methods of analysis. The development of linear methods and their solution by computer have made it possible to analyze large complex statically indeterminate structures. Material properties of concrete and masonry are fixed for all members. . Material properties of concrete are assigned to column, beams and slabs. Material properties of masonry is assigned to the struts. For analysis compressive strength of concrete is taken 3000 psi, modulus of elasticity of masonry is taken 1200 psi and the yield strength grade of steel is taken 40 ksi.

3.4.2 Loads for Analysis

Prior to structural analysis it is essential that the loads that may act upon a building during its lifetime be duly considered and incorporated in the analysis. The loads that may act upon the cyclone shelters are considered as follows:

Dead Loads (D): Dead loads (D) are those gravity loads which remain acting on the structure permanently without any change during the structure's normal service life. These are basically the loads coming from the weight of the different components of the structure. For the sake of convenience in the analysis, sometimes this kind of load is divided into two types, namely a) self weight of the structure (SW) and b) the weight coming from the non-structural permanent components of the building (SDEAD). In concrete buildings, the weight of slabs, beams, columns etc., which form the main structural system, is considered as the self weight (SW). The weights of floor finish, water proofing layer, partition walls and other non-structural permanent components generally constitute the rest of the total dead load, i.e. (SDEAD). For the analysis and design checking of the cyclone shelters, following values of dead loads have been used:

Unit weight of reinforced concrete =	150 pcf
Unit weight of brickwork =	120 pcf
Floor finish =	30 psf

Partition wall load = 0 psf to 70 psf (calculated on the basis of the usage of the floors)

Live load (L): Live load is the gravity load due to non-permanent objects like furniture, human occupancy etc. Since during cyclones a large mass of people occupy almost all the spaces of a cyclone shelter, Live Load for a cyclone shelter may be taken as 100 psf as suggested in the MCSP report (BUET & BIDS, 1993a). Figure 3.3 shows application of live load on a floor of the CDSP-II type cyclone shelter.

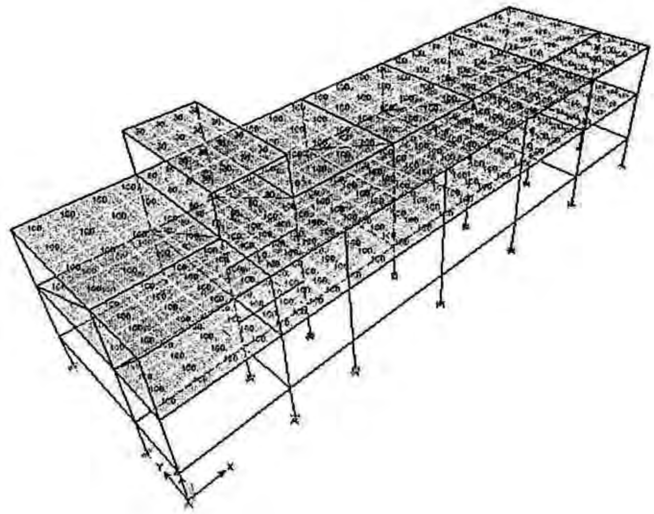

Figure 3.3: Live load (psf) applied on the CDSP-II type cyclone shelter

Wind load (W): BNBC(1993) recommends a basic wind speed of 260 km/h for cyclone of 50-year return period at coastal areas. In order to examine the performance of the structures for the worst case 260 km/h wind speed is used for analysis. Specifications on wind loading on buildings are obtained from BNBC (1993). For the present study the following basic parameters are used in wind load calculation,

Exposure category = C (Flat and unobstructed open terrain, coastal areas, riversides)

Structure Importance coefficient $C_I = 1.25$ (Essential facilities)

Tsunami load (T): Tsunami load is associated with hydrostatic, hydrodynamic and impact loads. When a structure is submerged, buoyant forces are also exerted on the structure. Local scouring caused by tsunami also influences the load carrying capacity of the structure.

Tsunami load on columns: Water usually remains on all sides of columns and thus hydrostatic forces are insignificant for columns. Columns are basically subjected to hydrodynamic and impact forces.

Hydrodynamic Forces: When water flows around a building (or structural element or other object) hydrodynamic loads are applied to the building. These loads are a function of flow velocity and structure geometry, and include frontal impact on the upstream face and drag along the sides. These loads are induced by the flow of water moving at moderate to high velocity. They are usually called the drag forces, which are combination of the lateral loads caused by the impact of the moving mass of water and the friction forces as the water flows around the obstruction.

FEMA CCM (2000) provides the following expression for hydrodynamic force (drag force) F_d:

$$F_d = \frac{1}{2}\rho C_d A u_p^2$$

where ρ is the water density, C_d is the drag coefficient, and A is the projected area of the body on the plane normal to the flow direction. The FEMA CCM recommends Cd = 2.0 for square or rectangular columns and 1.2 for round columns.

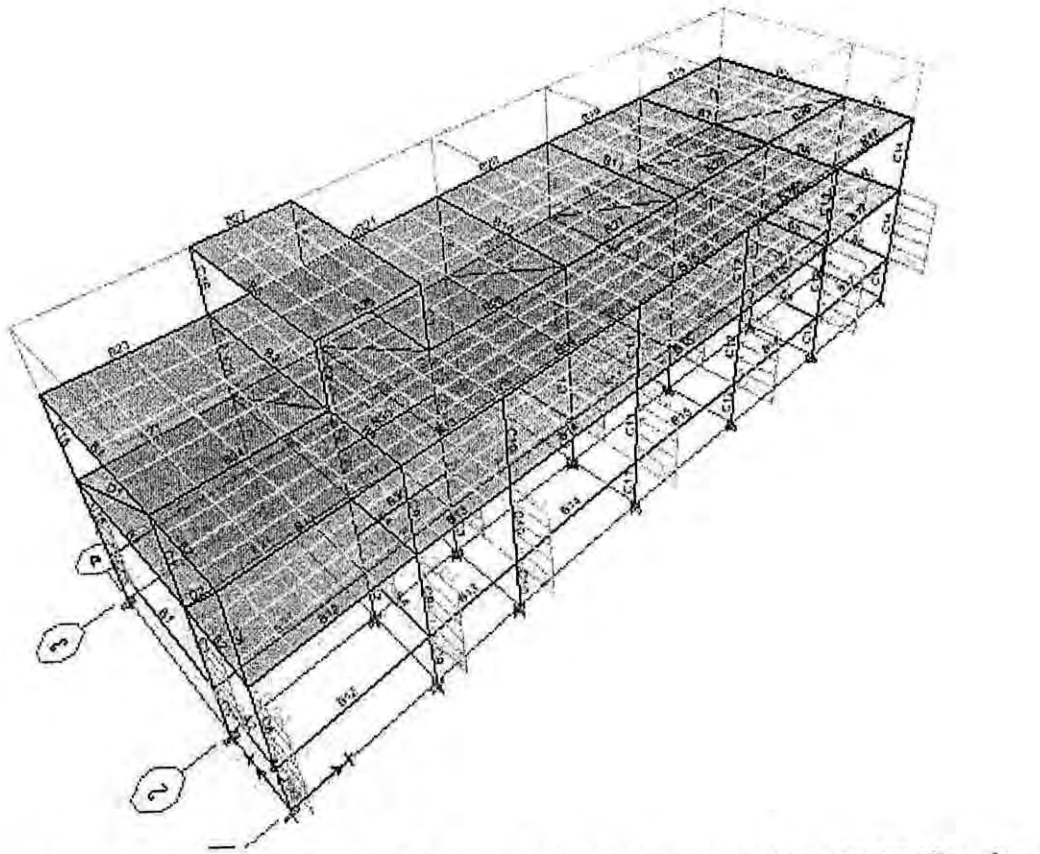

Figure 3.4: Hydrodynamic forces (lb/ft) applied on the columns of an CDSP-II cyclone shelter for 3m inundation due to tsunami

FEMA CCM provides the following estimate of the flood velocity u in the surge depth d_s:

$$u = 2\sqrt{gd_s}$$

Here g is the gravitational acceleration.

Figure 3.4 shows hydrodynamic forces applied on the columns of a CDSP-II type cyclone shelter for 6ft inundation due to tsunami.

Impact Forces: Impact loads are those that result from debris such as driftwood, small boats, portions of houses, etc., or any object transported by floodwaters, striking against buildings and structures or parts thereof. The magnitude of these loads is very difficult to predict, yet some reasonable allowance must be made for them. The velocity of waterborne objects is assumed to be the same as the flood velocity. The object is assumed to be at or near the water surface level when it strikes the building. Therefore, the object is assumed to strike the building at the water level. According to Chopra (1995), the duration of impact is influenced primarily by the natural frequency of the building, which is a function of the building's stiffness.

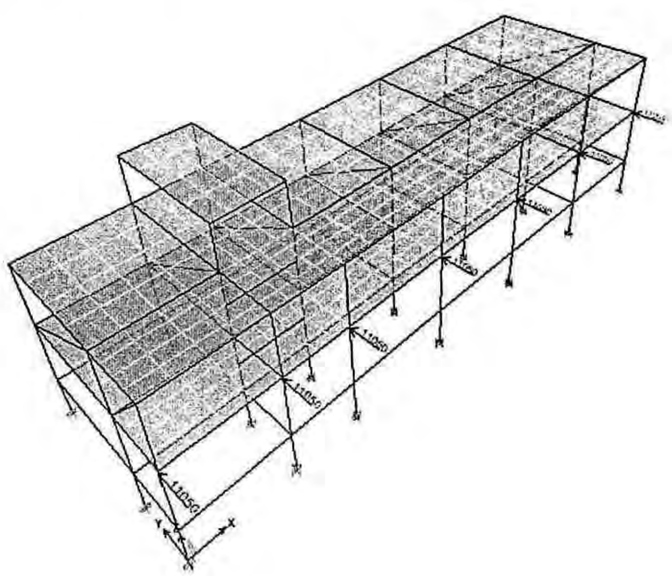

Figure 3.5: Impact forces applied on the columns of an CDSP-II type cyclone shelter for 3m inundation due to tsunami

The CCH (2000), FEMA CCM (2000) and ASCE 7 (1998) contain similar equations that result in the following generalized expression for impact force, F_I acting at the still water level:

$$F_I = m\frac{du_b}{dt} = m\frac{u_I}{\Delta t}$$

where u_b is the velocity of the impacting body, u_I is its approach velocity that is assumed equal to the flow velocity, *m* is the mass of the body, Δt is the impact duration that is equal to the time between the initial contact of the body with the building and the maximum impact force. The CCH recommends Δt values for reinforced concrete as 0.1 second. Figure 3.5 shows impact forces applied on the columns of an CDSP-II type cyclone shelter for 3m inundation due to tsunami. A sample calculation of the tsunami load is given in **Appendix-A**.

Loads on walls: Hydrostatic forces, in addition to hydrodynamic and impact forces, can be significant in case of walls. However, BNBC (1993) allows walls to sustain a maximum uniformly distributed load of 20 psf (1.0 kN/m^2) but not less than 10 psf (0.5 kN/m^2). A masonry wall usually can sustain an out of plane distributed load of about 10 psf (Hendry, 1981). In the present study the grounds floor of the cyclone shelters were open. So distributed load on wall was not considered.

Load Combination:

The basic sources of loads are described in earlier section. These loads are applied on the model in four basic categories. These are as follows,

Load Case 1: Self-weight of structure (SW).

Load Case 2: Floor finish and partition wall (SDEAD).

Load Case 3: Live load on floor/roof (LL)

Load Case 4: Tsunami load (T)

Load Case 5: Wind load (W)

These five basic load cases are analyzed in ETABS-V9.0.4. The results are then combined in accordance with the specifications set forth by BNBC (1993). BNBC (1993) specifies a number of combination options. The combinations used for tsunami are as follows:

1.4 D

1.4 D + 1.7 L

0.9 D + 1.3 (W or 1.1 E)

0.75 (1.4 D + 1.7 L ± 1.7 (W or 1.1 E))

Where D stands for total dead load i.e. D = DL + SDEAD, L stands for live load i.e. L=LL, W stands for wind load, E stands for earthquake load. Yeh et al. (2005) suggests that loads and stresses due to tsunamis are to be treated in the same fashion as for earthquake loading. When these basic load cases are combined accordingly, we obtain the following combination cases,

Combination Case 1: 1.4 D

Combination Case 2: 1.4 D + 1.7 L

Combination Case 3: 0.9 D + 1.4 T

Combination Case 4: 0.9 D - 1.4 T

Combination Case 5: 0.9 D + 1.3 W

Combination Case 6: 0.9 D - 1.3 W

Combination Case 7: 1.05 D + 1.275 L + 1.4 T

Combination Case 8: 1.05 D + 1.275 L - 1.4 T

Combination Case 9: 1.05 D + 1.275 L + 1.275 W

Combination Case 10: 1.05 D + 1.275 L - 1.275 W

The above combinations are factored combinations, i.e. factors like 1.4 or 1.7 are used to multiply the basic load cases before they are added. In this study combination cases 1 - 10 are used.

3.4.3 Masonry Infill Model

Equivalent strut model has been used to consider the effect of infill masonry walls. The masonry infill panel has been represented by an equivalent diagonal strut of width, a and thickness, t. The equivalent strut width, a depends on the relative flexural stiffness of the infill to that of the columns of the confining frame. The relative infill-to-frame stiffness is evaluated using equation (Stafford-Smith and Carter, 1969):

$$\lambda_1 H = H[(E_m t \sin\theta)/(4E_c I_{col} h_w)]^{1/4}$$

Where, H is the height of the story, E_m is modulus of elasticity of the masonry work, t is the thickness of the masonry wall, θ is the angle of the diagonal with the horizontal, E_c is modulus of elasticity of concrete, I_{col} is the moment of inertia of the column section and h_w is the height of the masonry work. The equivalent strut width is given by,

$$a = 0.175D(\lambda_1 H)^{-0.4}$$

Where, D is the diagonal length of the wall. In the FEM model strut is modeled using frame elements. A sample calculation of strut width is provided in **Appendix- B.**

3.5 Finite Element Analysis

The finite element analysis includes the static analysis and the dynamic analysis. As mentioned earlier for calculating equivalent static load BNBC 93 was followed. ETABS has some options to generate Time History load from given data. The data used as time history function is found from an experimental result by Kato *et al.*, 2005. In this experiment maximum height of the water level was 1.18 m. Load from water pressure for time history loading is found from this experiment. No actual time history data for tsunami was available. So this data was used as an experimental study by taking the height of water level for tsunami as 1.18m. The loading graph is presented in the following figure.

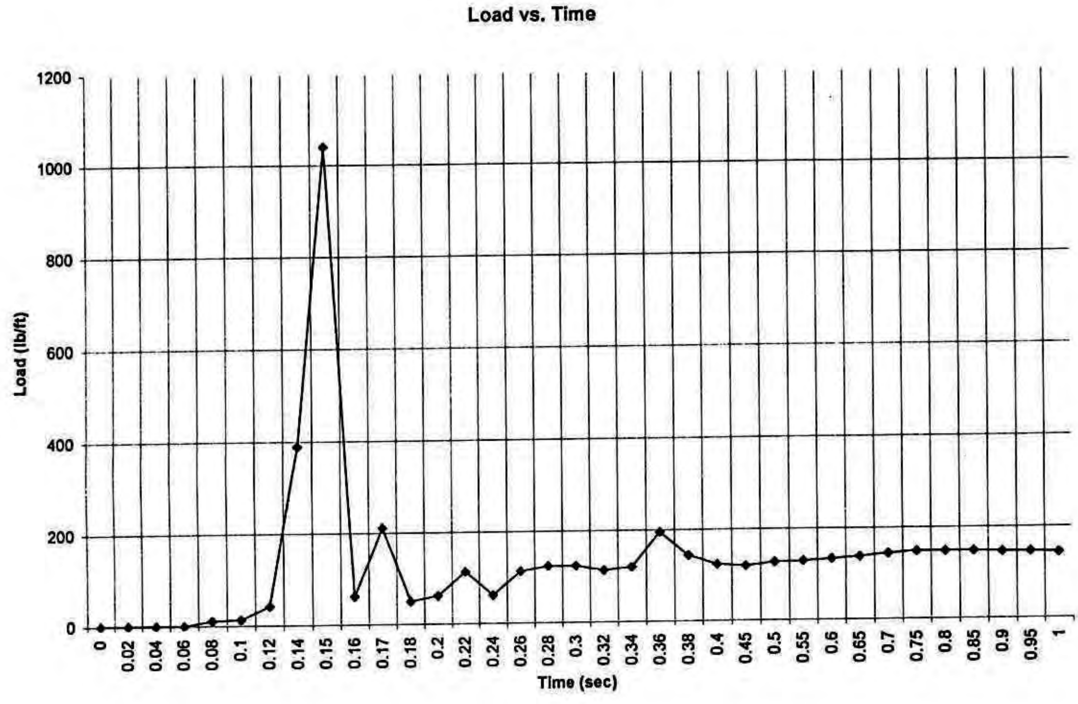

Figure 3.6: Time History Graph

CHAPTER 4

RESULTS AND DISCUSSIONS

4.1 General

The structural design of most of the cyclone shelters is not available. The analysis is performed only to find out the required reinforcement and check the adequacy of the structures. From analysis, it was found that the beams of these structures are in general adequate in flexure. But in some structures a number of beams are inadequate to resist torsion. Since previously, structures were used to be analyzed commonly with 2D models, torsion was often ignored. In the present 3D analysis, thus it is found that beams are often not designed to resist torsion. But due to presence of masonry walls these beams can resist torsion. Each type of cyclone shelter has been analyzed for different loading conditions. As the designs of almost all of the structures are not available, it is assumed in the present study that there is maximum 3% steel reinforcement in columns of the cyclone shelters. When the required steel reinforcement in column is found less than 3%, it is assumed that the column will survive and expressed as "Ok".

In this study fourteen types of cyclone shelters have been analyzed. As the height of the tsunami can not be predicted, analysis of these structures has been performed for three different heights (1m, 2m and 3m). Time History Analysis has been performed as an experimental study as no actual data for the Time History Analysis was available. The results of the analysis of these structures are given below.

4.2 Cyclone shelter funded by the European Union

This is a two-storied building with open ground story. The ground story height is about 12 ft. Figures 4.1 and 4.2 show, respectively, the 3D view without bracing and 3D view with bracing. Plan of the FE model of the building is shown in Figure 4.3.

Figure 4.4 shows the required rebar percentage of the frame member of the same structure. Moment diagram and deflected shape are shown in Figure 4.5 and 4.6 respectively.

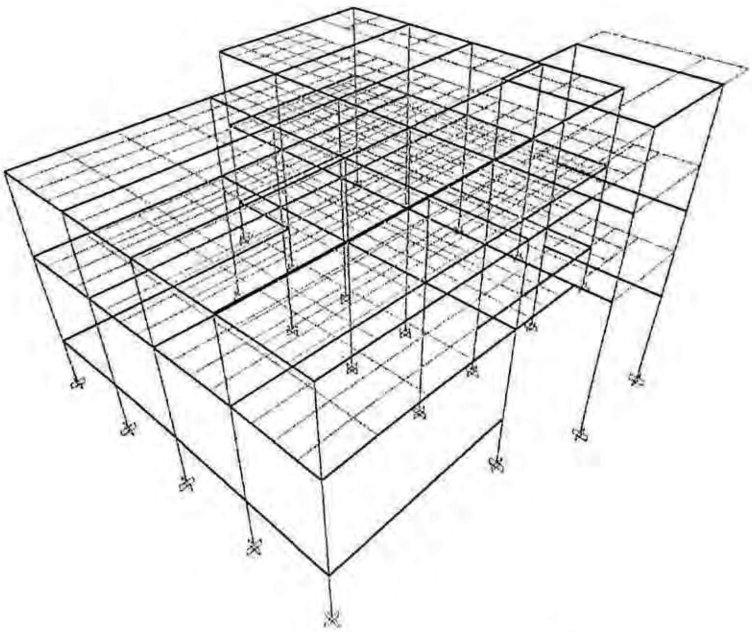

Figure 4.1: 3D view of the FE model of a typical cyclone shelter funded by the European Union (without bracing)

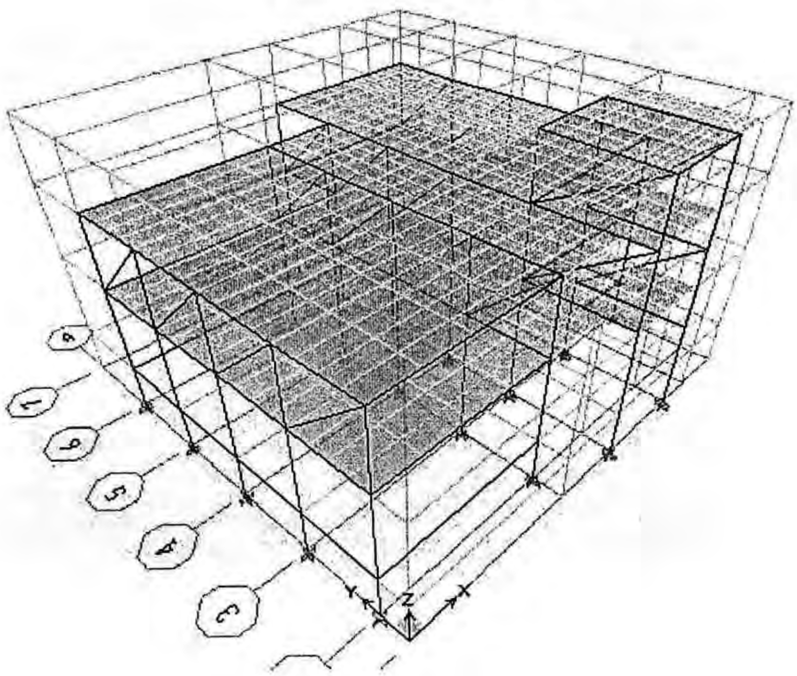

Figure 4.2: 3D view of the FE model of a typical cyclone shelter funded by the European Union (with bracing)

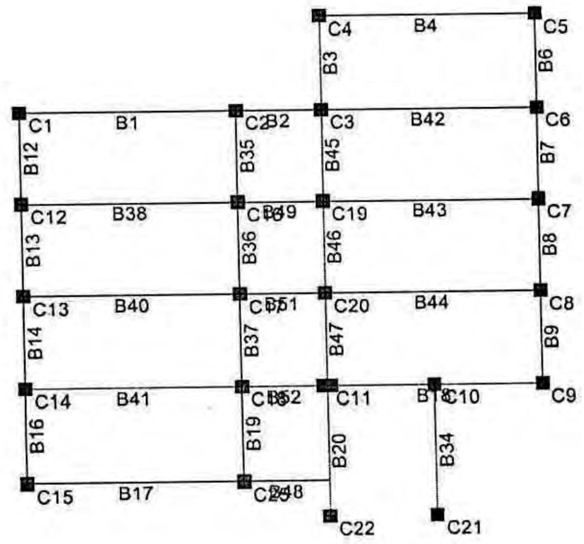

Figure 4.3: Plan of a typical cyclone shelter funded by the European Union

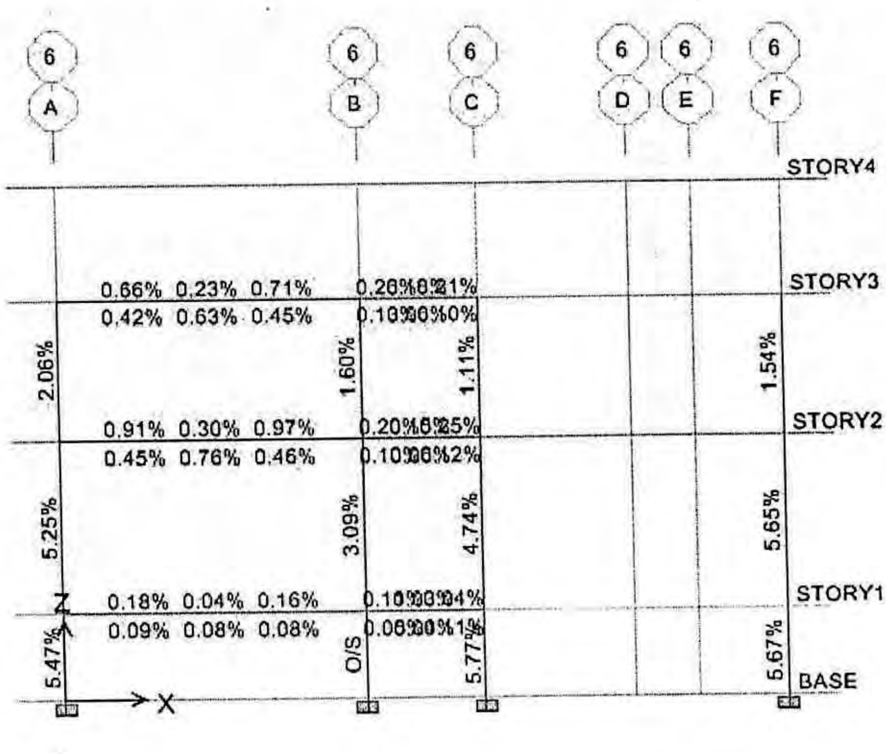

Figure 4.4: Required rebar percentage of frame members of a typical cyclone shelter funded by the European Union

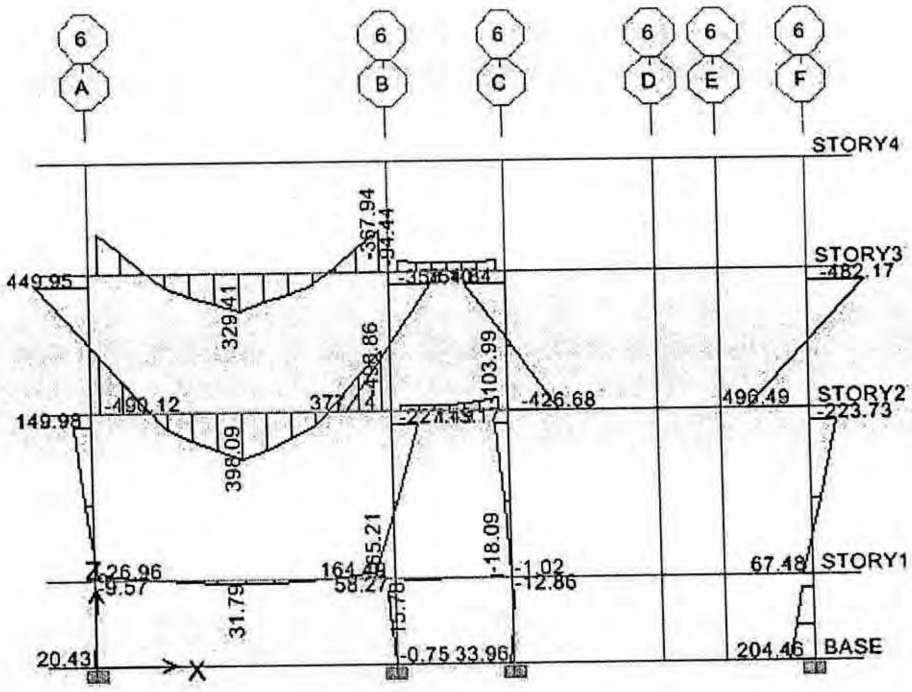

Figure 4.5: Moment Diagram of a typical cyclone shelter funded by the European Union

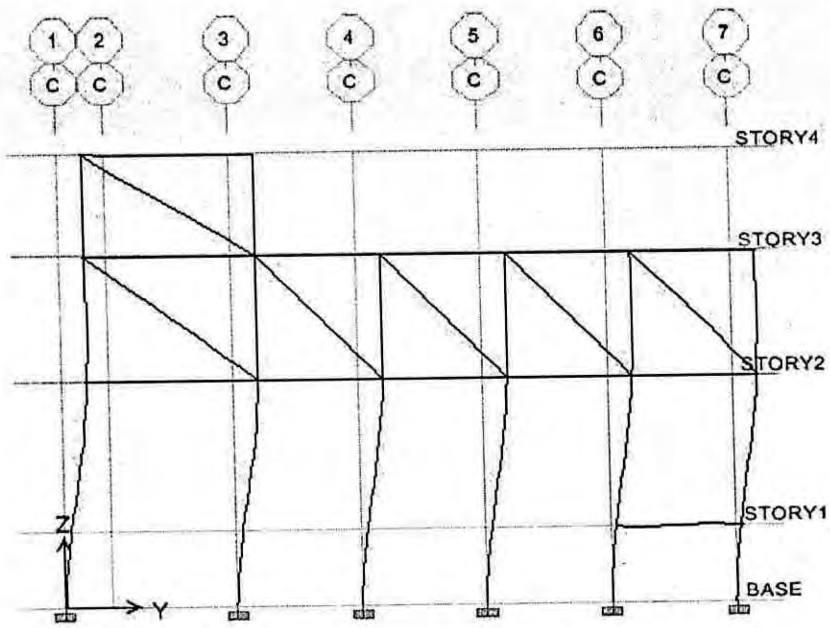

Figure 4.6: Deflected Shape of a typical cyclone shelter funded by the European Union

Tables 4.1 and 4.2 show the reinforcement requirement of columns for different loading conditions for two types of model-(1) without bracing and (2) with bracing. From the results it is found that for 3m high tsunami some columns will fail. If strut is considered instead of wall the required steel reduces but it is not significant. Time History loading effect is not significant for columns. But it is seen that a number beams are failing due to torsion effect on beams for all categories of loading. However, in reality, beam torsion is largely balanced due to the underlying masonry infill.

Table 4.1: Beam and column condition of a typical cyclone shelter (Model without bracing) funded by the European Union due to Tsunami Load

Height of Tsunami	Inadequate Beams	Beam Vulnerability Condition	Inadequate Columns	Maximum Required Reinforcement in Column
1m	B1,3, 4,6-9, 12-14, 16-19,35-37,	Torsion	–	Ok
2m	B1,3, 4,6-9, 12-14, 16-19,35-37,	Torsion	–	Ok
3m	B1-3, 4,6-9, 12-14, 16-19,35-37,	Torsion	C1-5,9-11, 15-19,25	C15= 5.68%
			C6-8, 12-14	>8%
Time History loading for 1.18m tsunami	B1,3, 4,6-9, 12-14, 16-19,35-37,	Torsion	–	Ok

Table 4.2: Beam and column condition of a typical cyclone shelter (Model with bracing) funded by the European Union due to Tsunami Load

Height of Tsunami	Inadequate Beams	Beam Vulnerability Condition	Inadequate Columns	Maximum Required Reinforcement in Column
1m	B1,3, 4,6-9, 12-14, 16-19,35-37	Torsion	–	Ok
2m	B1,3, 4,6-9, 12-14, 16-19,35-37	Torsion	–	Ok
3m	B1,3, 4,6-9, 12-14, 16-19,35-37	Torsion	C10, 12-14	>8%
			C1-9,15, 21,25	C6= 5.65%
Time History loading for 1.18m tsunami	B1,3, 4,6-9, 12-14, 16-19,35-37	Torsion	–	Ok

4.3 JICA type cyclone shelter

This is a two-storied building with open ground story. The ground story height is more than 13 ft. The columns are arranged in a regular grid. Figures 4.7 and 4.8 show, respectively, the 3D view without bracing and 3D view with bracing. Plan of the FE model of the building is shown in Figure 4.9. Figure 4.10 shows the required rebar percentage of the frame member of the same structure. Moment diagram and deflected shape are shown in Figure 4.11 and 4.12 respectively.

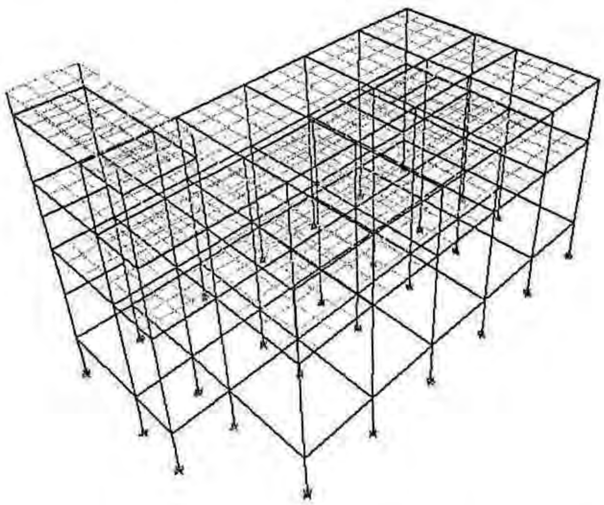

Figure 4.7: 3D view of the FE model of a JICA type cyclone shelter (without bracing)

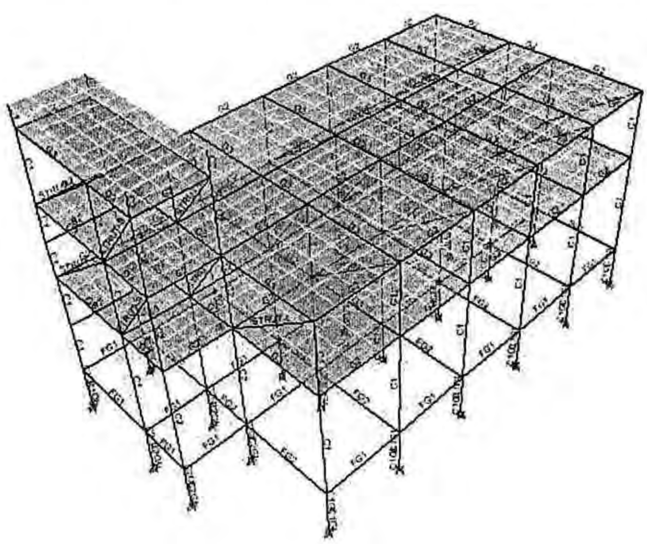

Figure 4.8: 3D view of the FE model of a JICA type cyclone shelter (with bracing)

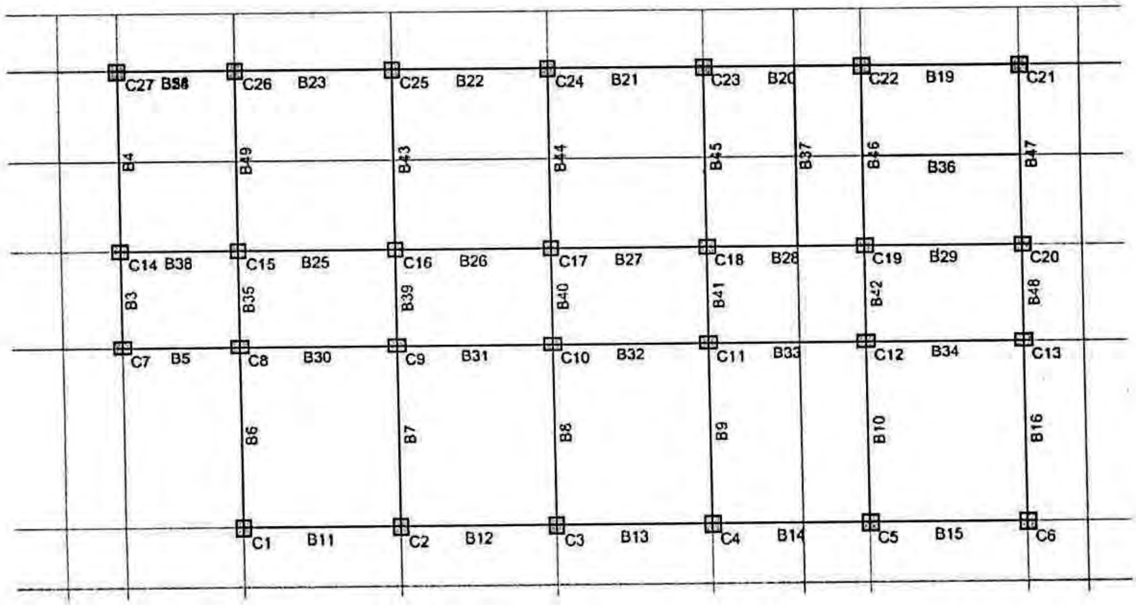

Figure 4.9: Plan of a JICA type cyclone shelter

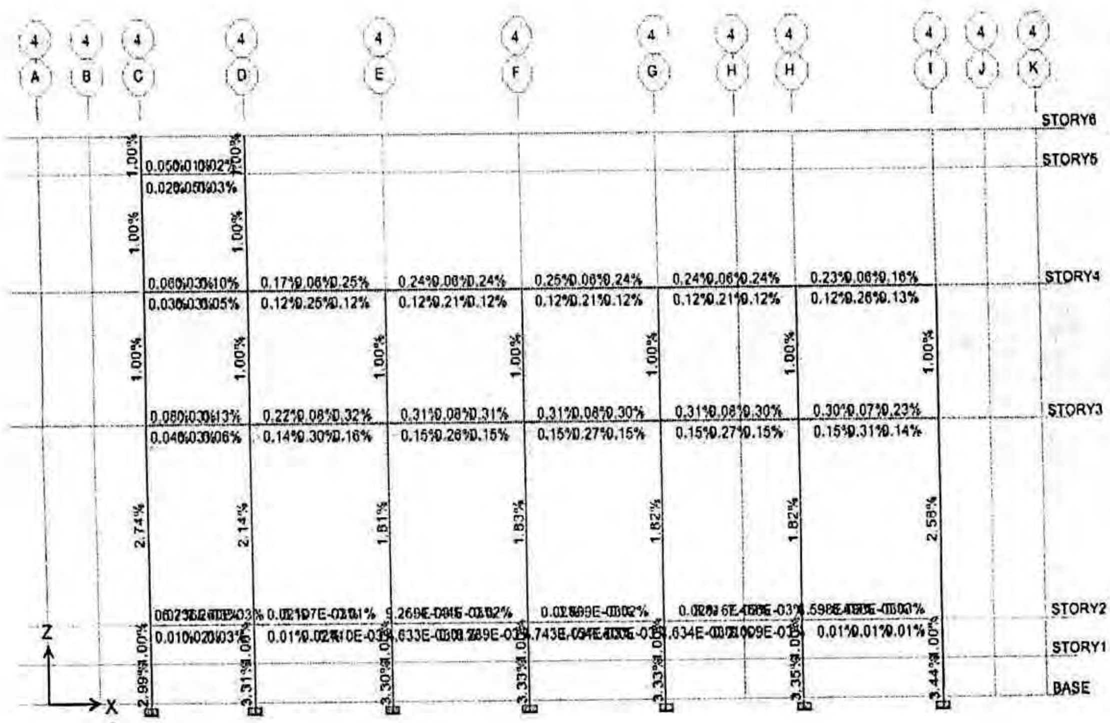

Figure 4.10: Required rebar percentage of frame members of a JICA type cyclone shelter

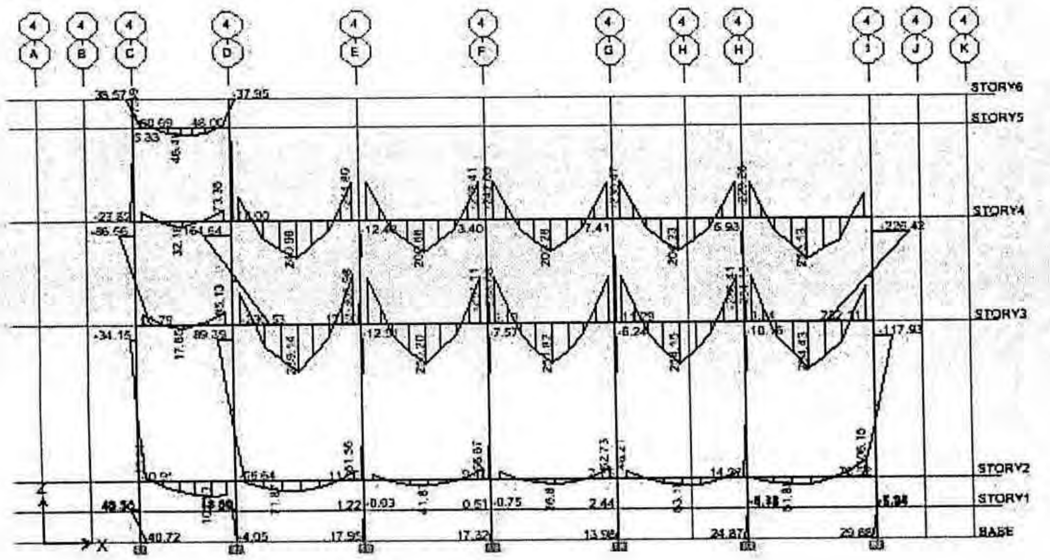

Figure 4.11: Moment Diagram of a JICA type cyclone shelter

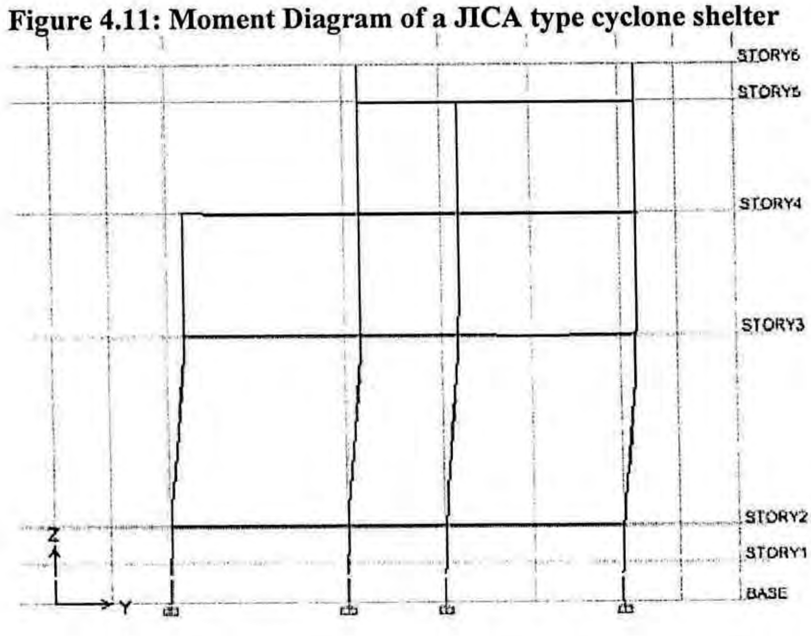

Figure 4.12: Deflected Shape of a JICA type cyclone shelter

Tables 4.3 and 4.4 show the reinforcement requirement of columns for different loading conditions for two types of model-(1) without bracing and (2) with bracing. From the results it is found that up to 3m high tsunami the structure survives. If strut is considered instead of wall the structure survives also. Time History loading effect is not significant. This structure seems to be safe in different loading categories.

Table 4.3: Beam and column condition of a typical Jica Type cyclone shelter (Model without bracing) due to Tsunami Load

Height of Tsunami	Inadequate Beams	Beam Vulnerability Condition	Inadequate Columns	Maximum Required Reinforcement in Column
1m	–	–	–	ok
2m	–	–	–	ok
3m	–	–	–	ok
Time History loading for 1.18m tsunami	–	–	–	ok

Table 4.4: Beam and column condition of a typical Jica Type cyclone shelter (Model with bracing) due to Tsunami Load

Height of Tsunami	Inadequate Beams	Beam Vulnerability Condition	Inadequate Columns	Maximum Required Reinforcement in Column
1m	–	–	–	ok
2m	–	–	–	ok
3m	–	–	–	ok
Time History loading for 1.18m tsunami	–	–	–	ok

4.4 LGED type cyclone shelter

This is a three-storied building with open ground story. The ground story height is about 10 ft. Figures 4.13 and 4.14 show, respectively, the 3D view without bracing and 3D view with bracing. Plan of the FE model of the building is shown in Figure 4.15. Figure 4.16 shows the required rebar percentage of the frame member of the

same structure. Moment diagram and deflected shape are shown in Figure 4.17 and 4.18 respectively.

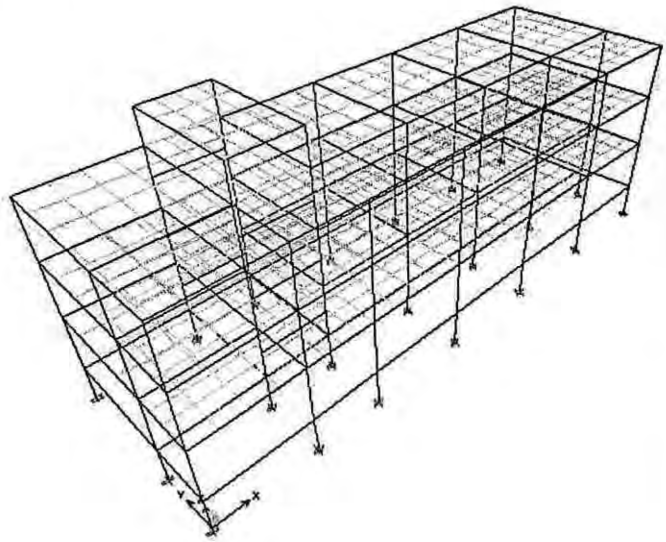

Figure 4.13: 3D view of the FE model of an LGED type cyclone shelter (without bracing)

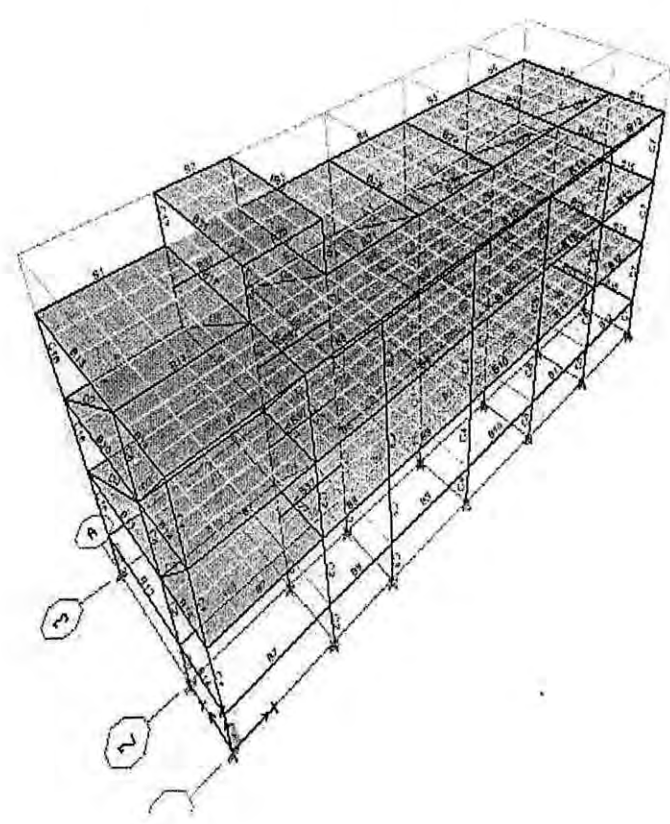

Figure 4.14: 3D view of the FE model of an LGED type cyclone shelter (with bracing)

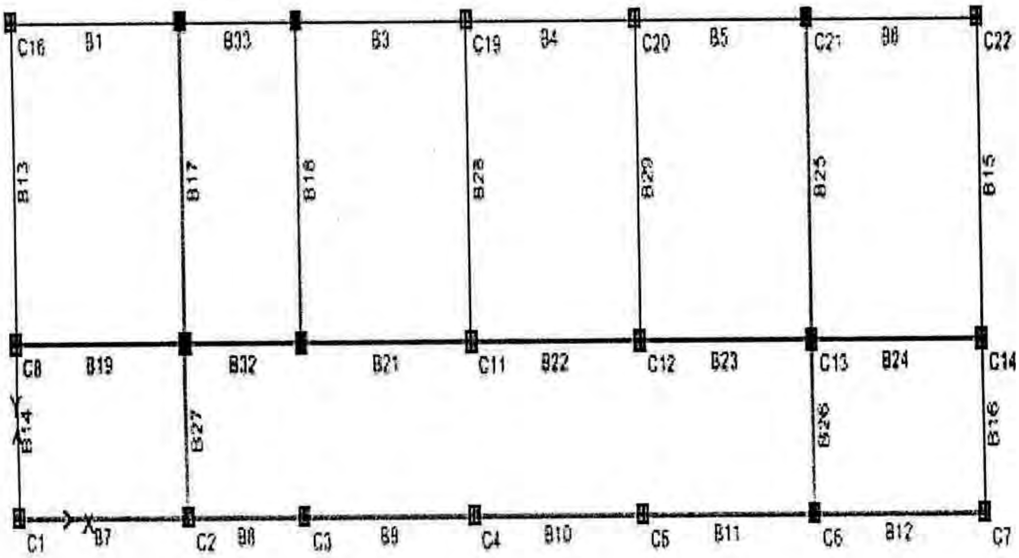

Figure 4.15: Plan of an LGED type cyclone shelter

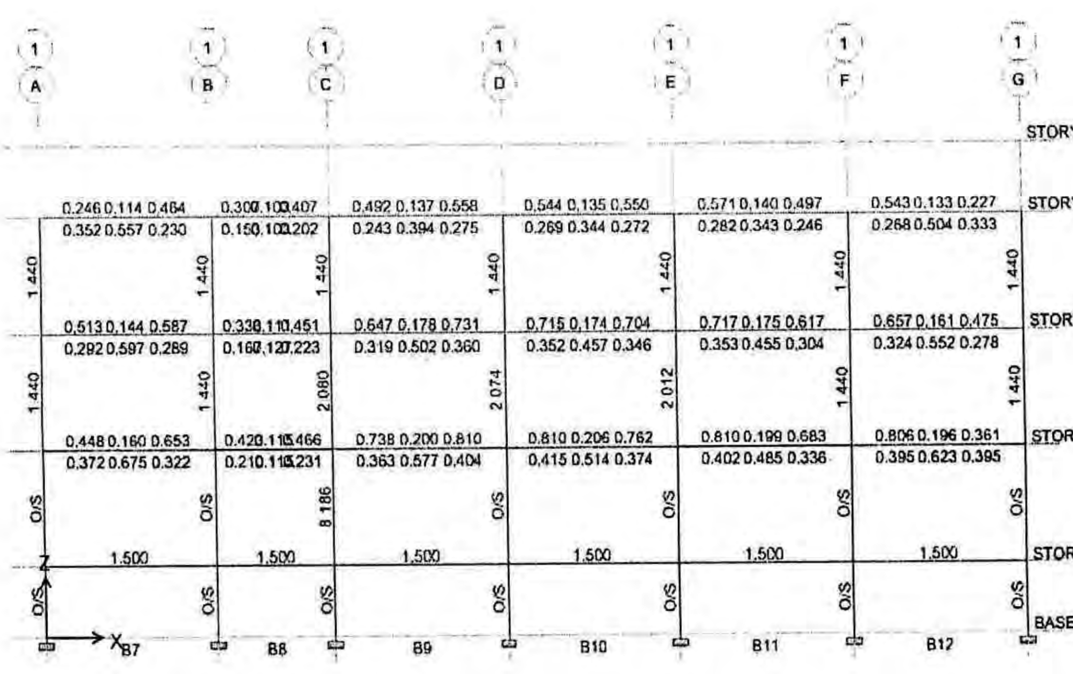

Figure 4.16: Required rebar percentage of frame members of an LGED type cyclone shelter

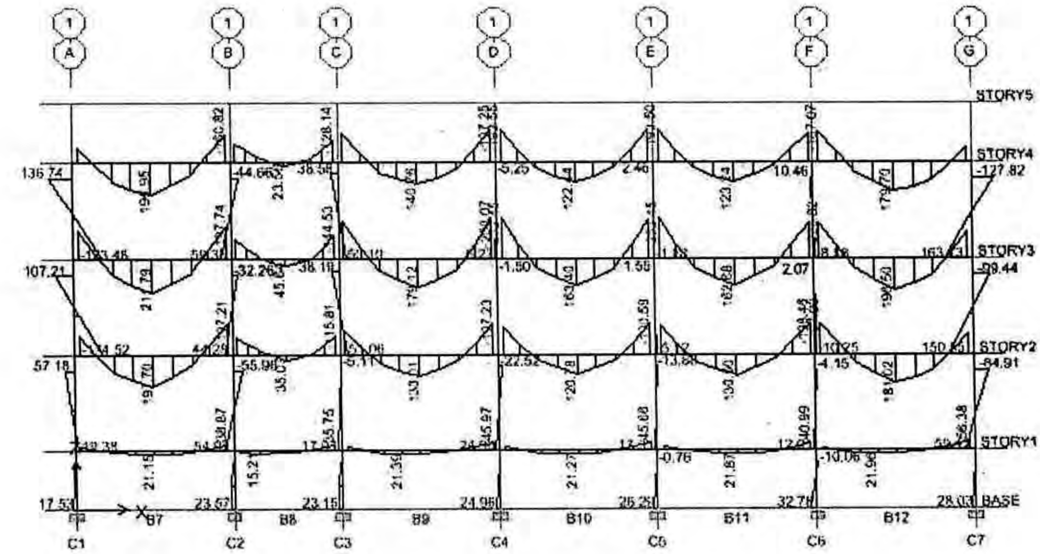

Figure 4.17: Moment Diagram of an LGED type cyclone shelter

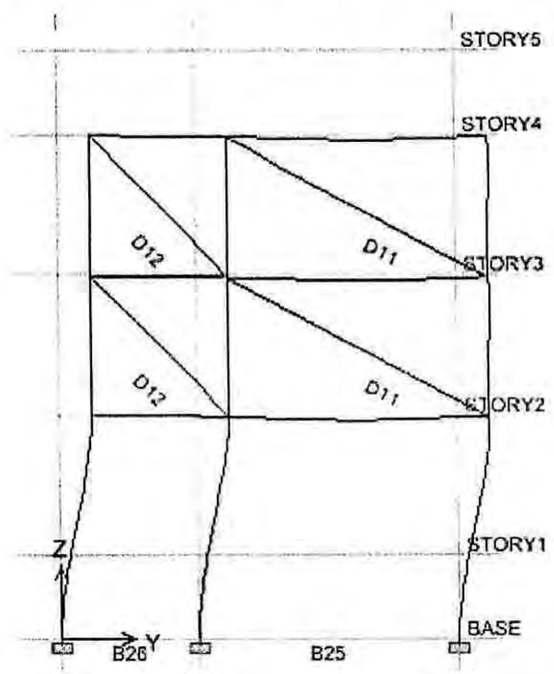

Figure 4.18: Deflected Shape of an LGED type cyclone shelter

Tables 4.5 and 4.6 show the reinforcement requirement of columns for different loading conditions for two types of model-(1) without bracing and (2) with bracing.

From the results it is found that for 2m and above high tsunami some columns will fail. If strut is considered instead of wall the result is almost same. Time History loading effect is not significant. But it is seen that some beams are failing due to torsion effect on beams for the model without strut.

Table 4.5: Beam and column condition of a typical LGED Type cyclone shelter (Model without bracing) due to Tsunami Load

Height of Tsunami	Inadequate Beams	Beam Vulnerability Condition	Inadequate Columns	Maximum Required Reinforcement in Column
1m	–	–	–	Ok
2m	–	–	C1,7,11-14, 17-22	C13= 5.17%
3m	B8,11	Torsion	C3-5	C4= 5.66%
			Rest of the columns	>8%
Time History loading for 1.18m tsunami	–	–	–	Ok

Table 4.6: Beam and column condition of a typical LGED Type cyclone (Model with bracing) shelter due to Tsunami Load

Height of Tsunami	Inadequate Beams	Beam Vulnerability Condition	Inadequate Columns	Maximum Required Reinforcement in Column
1m	–	–	–	Ok
2m	–	–	C1,7,11-14, 17-22	C19= 5.16%
3m	–	–	C3-5	C4= 5.63%
			Rest of the columns	>8%
Time History loading for 1.18m tsunami	–	–	–	Ok

4.5 Cyclone shelter funded by Saudi Arabia

This type of cyclone shelter is three storied building with open ground story. The ground story height is 12 ft. Figures 4.19 and 4.20 show, respectively, the 3D view without bracing and 3D view with bracing. Plan of the FE model of the building is shown in Figure 4.21. Figure 4.22 shows the required rebar percentage of the frame member of the same structure. Moment diagram and deflected shape are shown in Figure 4.23 and 4.24 respectively.

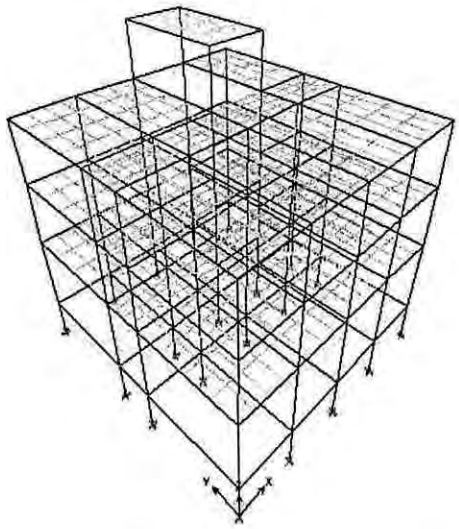

Figure 4.19: 3D view of the FE model of a typical cyclone shelter funded by Saudi Arabia (without bracing)

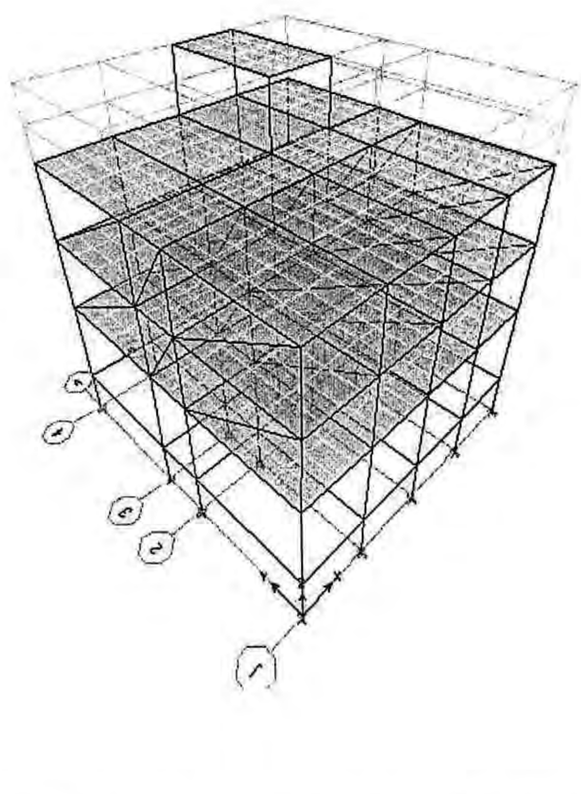

Figure 4.20: 3D view of the FE model of a typical cyclone shelter funded by Saudi Arabia (with bracing)

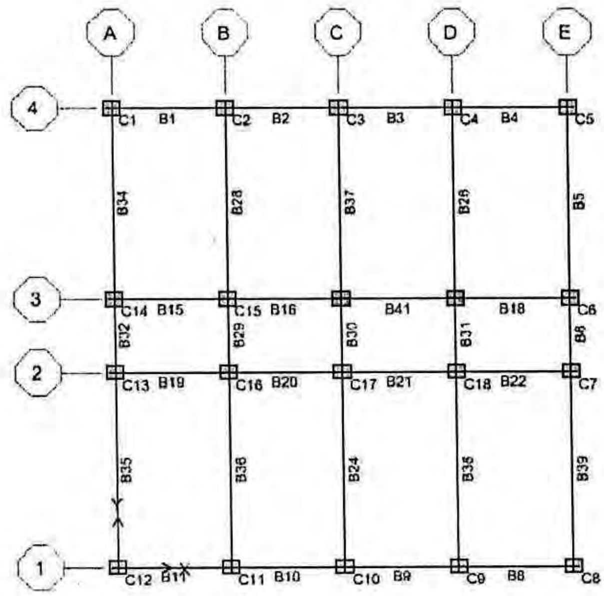

Figure 4.21: Plan of a typical cyclone shelter funded by Saudi Arabia

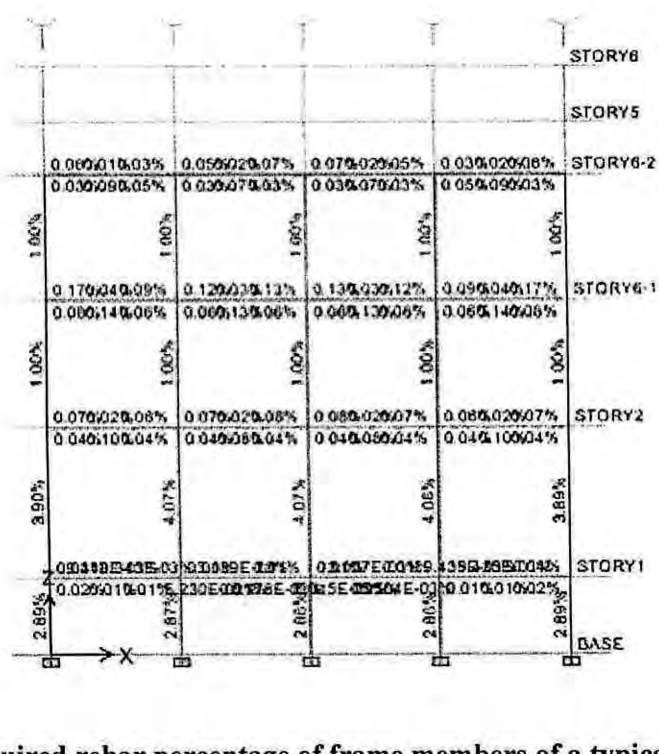

Figure 4.22: Required rebar percentage of frame members of a typical cyclone shelter funded by Saudi Arabia

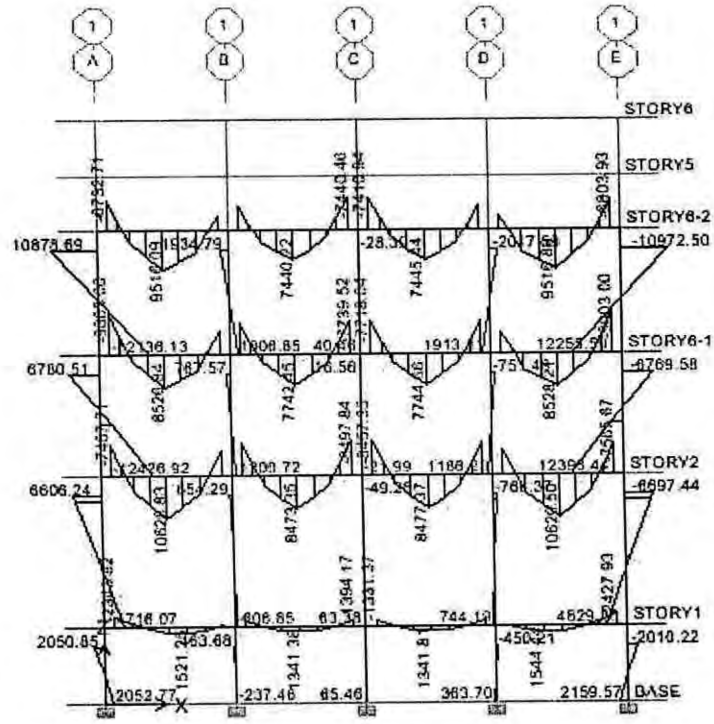

Figure 4.23: Moment Diagram of a typical cyclone shelter funded by Saudi Arabia

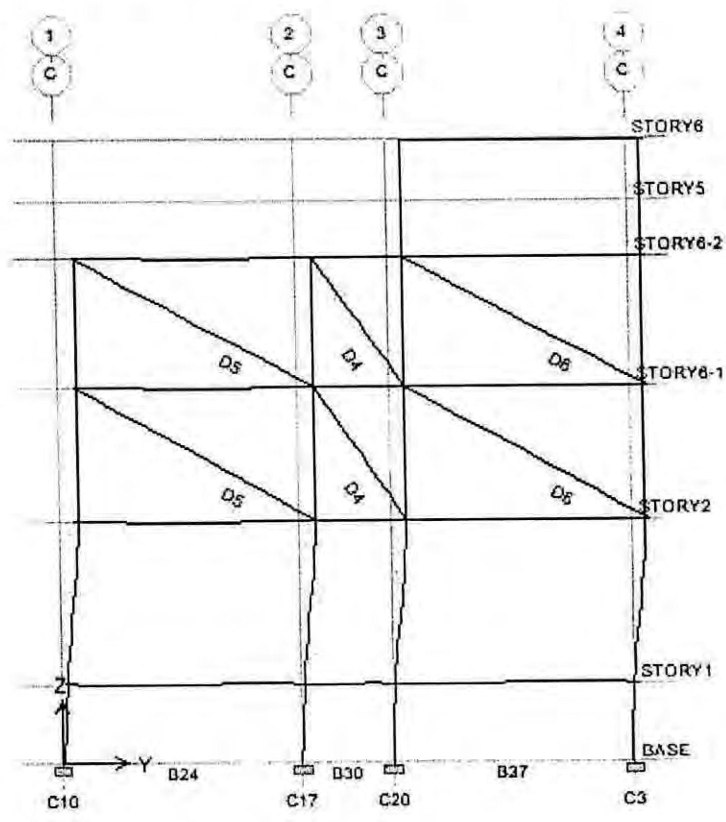

Figure 4.24: Deflected Shape of a typical cyclone shelter funded by Saudi Arabia

Tables 4.7 and 4.8 show the reinforcement requirement of columns for different loading conditions for two types of model-(1) without bracing and (2) with bracing. From the results it is found that for 3m high tsunami all the columns fail and maximum 5.72% steel reinforcement is required for column. If strut is considered instead of wall the required steel reduces but it is not significant. Time History loading effect is not significant. This structure seems to be safe up to 2m high tsunami in different loading categories.

Table 4.7: Beam and column condition of a typical cyclone shelter(Model without bracing) funded by Saudi Arabia due to Tsunami Load

Height of Tsunami	Inadequate Beams	Beam Vulnerability Condition	Inadequate Columns	Maximum Required Reinforcement in Column
1m	–	–	–	Ok
2m	–	–	–	Ok
3m	–	–	All columns	C17= 5.72%
Time History loading for 1.18m tsunami	–	–	–	Ok

Table 4.8: Beam and column condition of a typical cyclone shelter (Model with bracing) funded by Saudi Arabia due to Tsunami Load

Height of Tsunami	Inadequate Beams	Beam Vulnerability Condition	Inadequate Columns	Maximum Required Reinforcement in Column
1m	–	–	–	Ok
2m	–	–	–	Ok
3m	–	–	All columns	C17= 5.64%
Time History loading for 1.18m tsunami	–	–	–	Ok

4.6 Cyclone shelter built by Grameen Bank

This is a hexagon shaped two-storied building. Being hexagonal the plan is regular. The open ground story height is about 11 ft. Figures 4.25 and 4.26 show, respectively, the 3D view without bracing and 3D view with bracing. Plan of the FE model of the building is shown in Figure 4.27. Figure 4.28 shows the required rebar percentage of the frame member of the same structure. Moment diagram and deflected shape are shown in Figure 4.29 and 4.30 respectively.

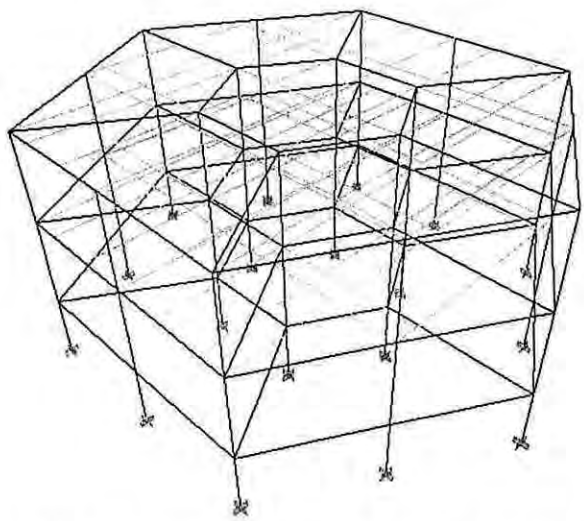

Figure 4.25: 3D view of the FE model of a typical cyclone shelter built by Grameen Bank (without bracing)

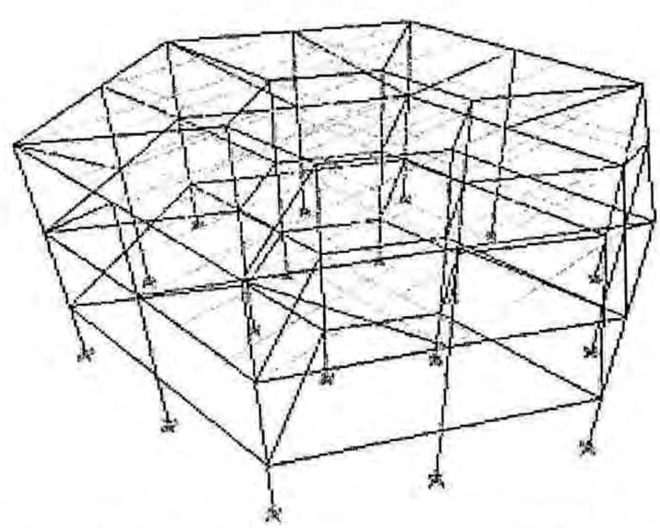

Figure 4.26: 3D view of the FE model of a typical cyclone shelter built by Grameen Bank (with bracing)

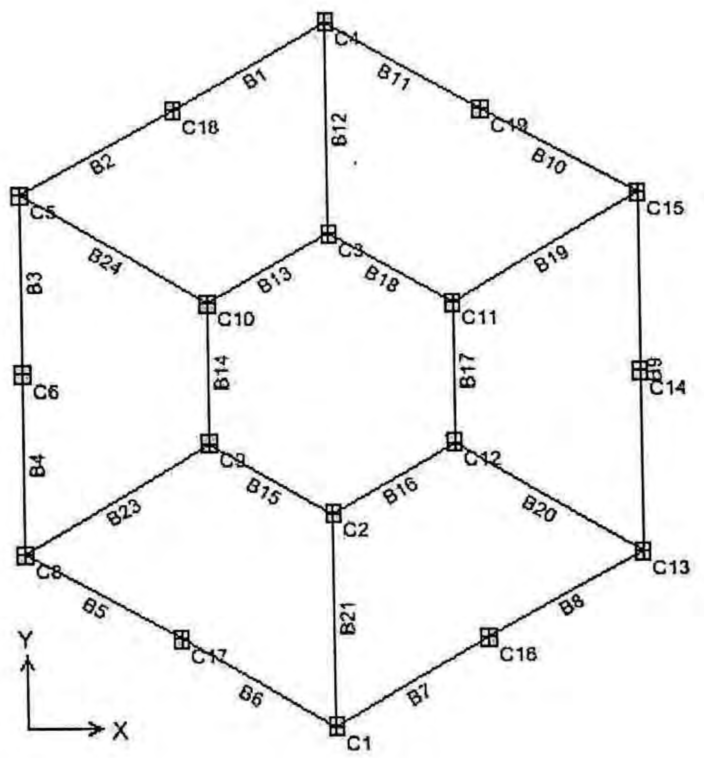

Figure 4.27: Plan of a typical cyclone shelter built by Grameen Bank

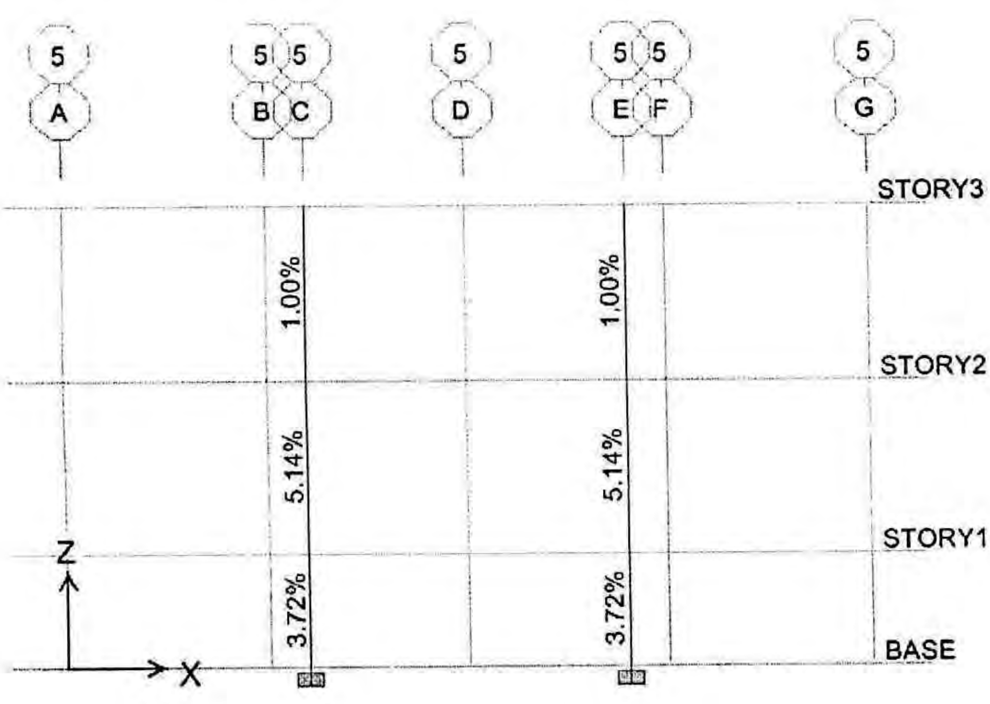

Figure 4.28: Required rebar percentage of frame members of a typical cyclone shelter funded by Grameen Bank

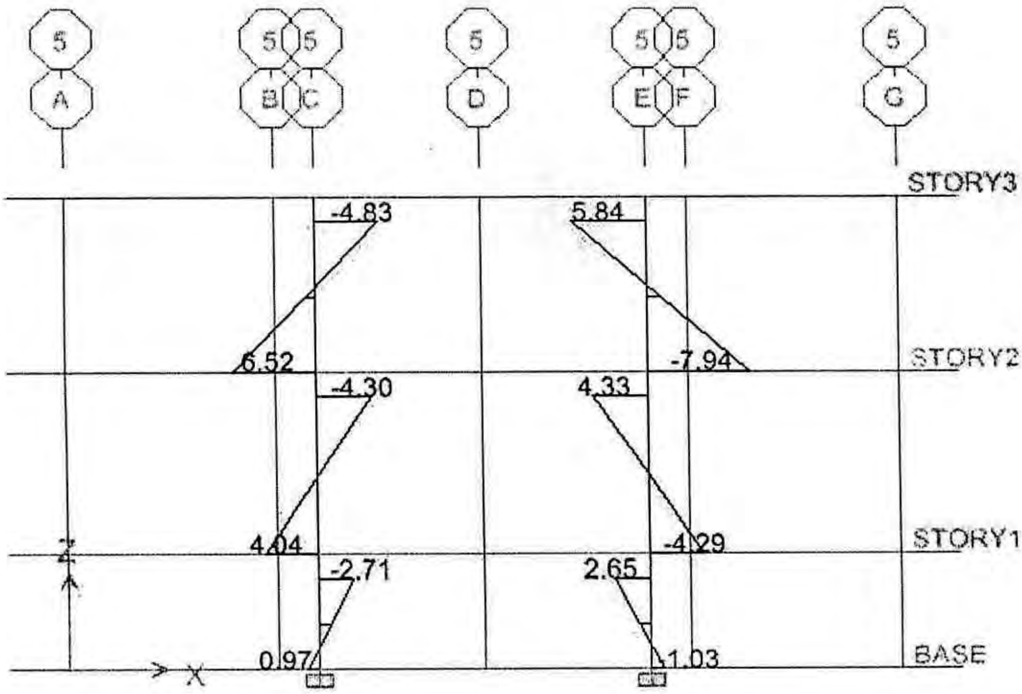

Figure 4.29: Moment Diagram of a typical cyclone shelter built by Grameen Bank

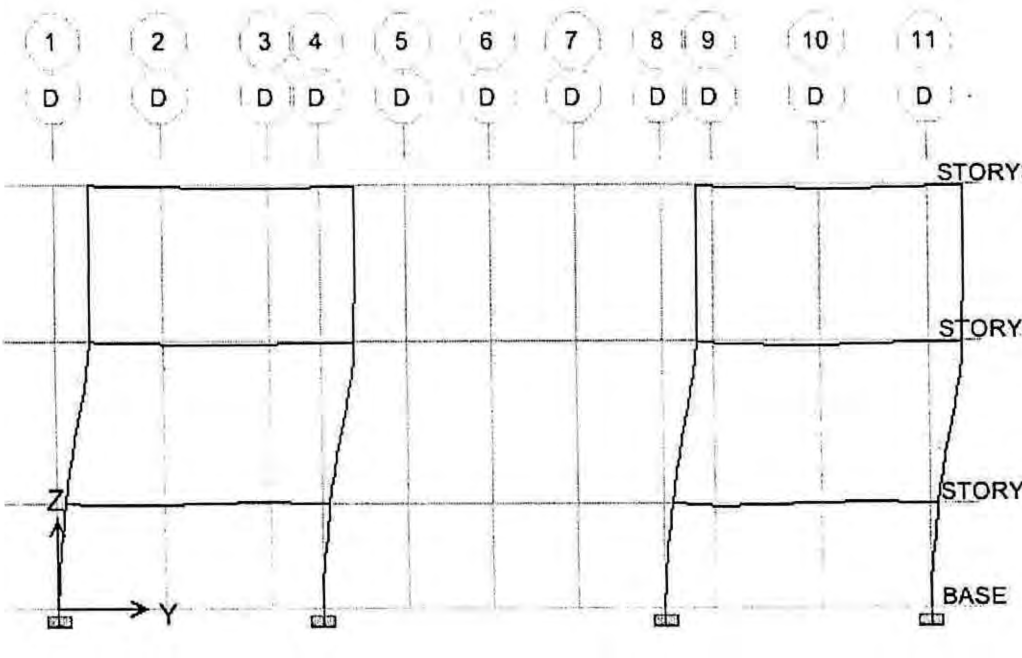

Figure 4.30: Deflected Shape of a typical cyclone shelter built by Grameen Bank

Tables 4.9 and 4.10 show the reinforcement requirement of columns for different loading conditions for two types of model-(1) without bracing and (2) with bracing.

From the results it is found that for 3m high tsunami some columns will fail. If strut is considered instead of wall the required steel reduces but it is not significant. Time History loading effect is not significant for columns. But it is seen that a number beams are failing due to torsion effect on beams for all categories of loading. However, in reality, beam torsion is largely balanced due to the underlying masonry infill.

Table 4.9: Beam and column condition of a typical cyclone shelter (Model without bracing) built by Grameen Bank due to Tsunami Load

Height of Tsunami	Inadequate Beams	Beam Vulnerability Condition	Inadequate Columns	Maximum Required Reinforcement in Column
1m	B19,20,23,24	Torsion	–	Ok
2m	B19,20,23,24	Torsion	–	Ok
3m	B19,20,23,24	Torsion	C1-5,8-13, 15	C8,13= 5.09%
			C6,14	>8%
Time History loading for 1.18m tsunami	B19,20,23,24	Torsion	–	Ok

Table 4.10: Beam and column condition of a typical cyclone shelter (Model with bracing) built by Grameen Bank due to Tsunami Load

Height of Tsunami	Inadequate Beams	Beam Vulnerability Condition	Inadequate Columns	Maximum Required Reinforcement in Column
1m	B19,20,23,24	Torsion	–	Ok
2m	B19,20,23,24	Torsion	–	Ok
3m	B19,20,23,24	Torsion	C1-5,8-13, 15	C8,13= 5.06%
			C6, 14	>8%
Time History loading for 1.18m tsunami	B19,20,23,24	Torsion	–	Ok

4.7 CDSP-2 type cyclone shelter

This is a two-storied building with open ground story. The ground story height is about 10 ft. Figures 4.31 and 4.32 show, respectively, the 3D view without bracing and 3D view with bracing. Plan of the FE model of the building is shown in Figure 4.33. Figure 4.34 shows the required rebar percentage of the frame member of the same structure. Moment diagram and deflected shape are shown in Figure 4.35 and 4.36 respectively.

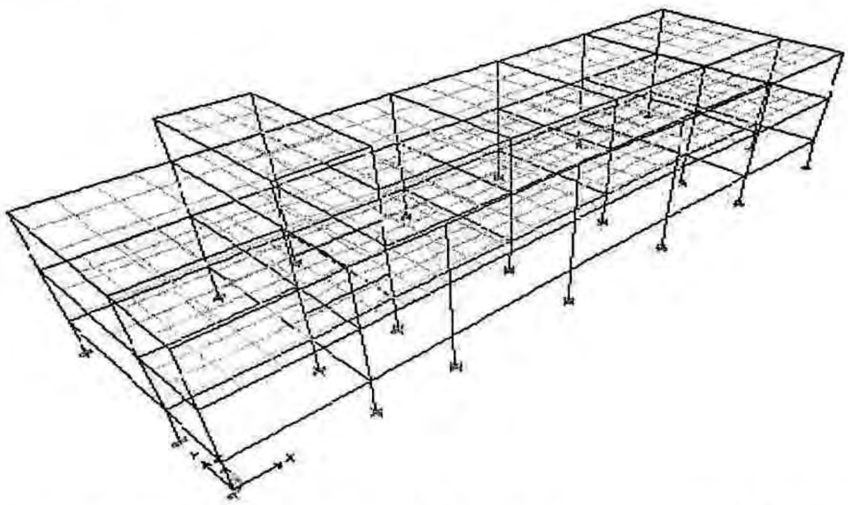

Figure 4.31: 3D view of the FE model of a CDSP-2 type cyclone shelter (without bracing)

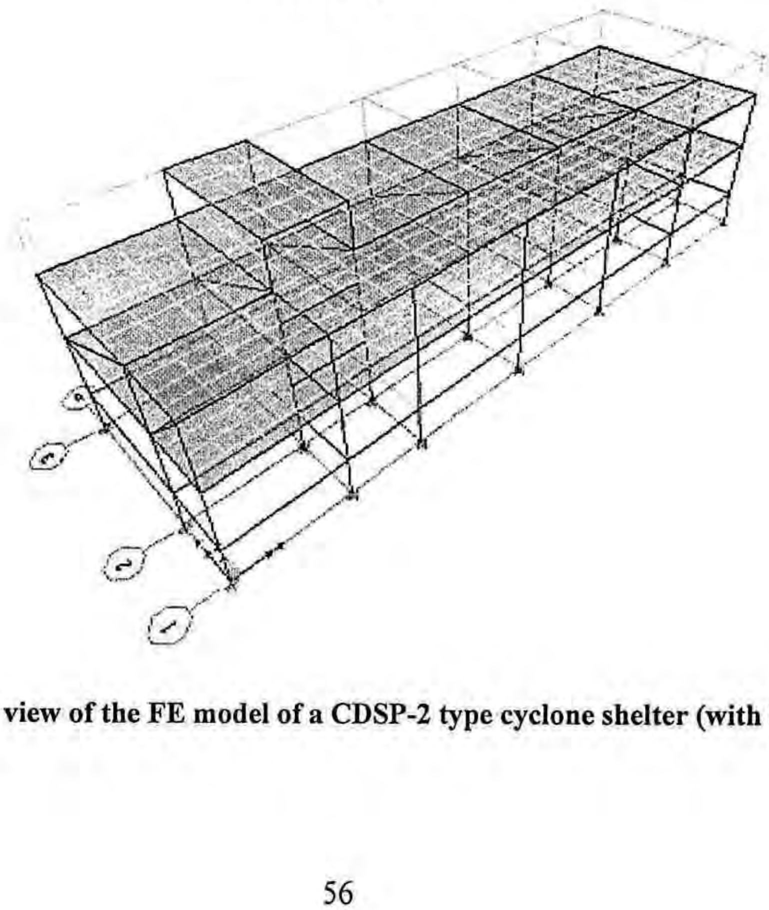

Figure 4.32: 3D view of the FE model of a CDSP-2 type cyclone shelter (with bracing)

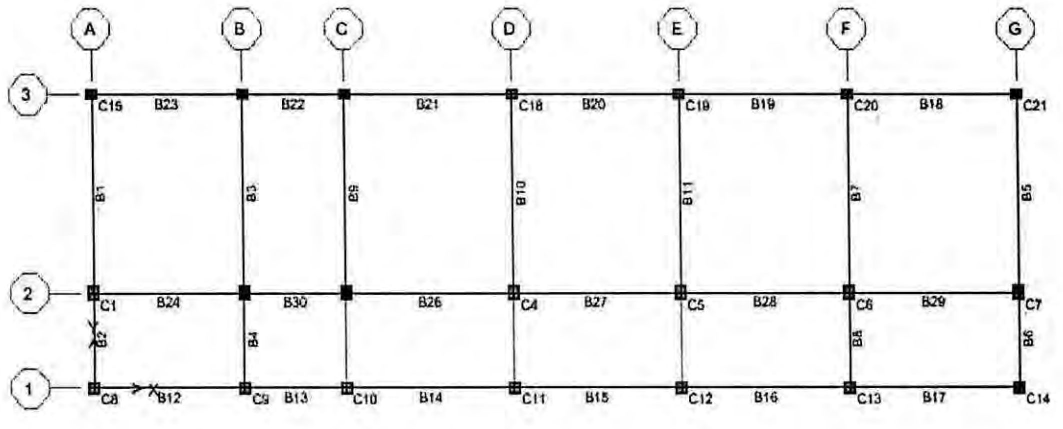

Figure 4.33: Plan of a CDSP-2 type cyclone shelter

Figure 4.34: Required rebar percentage of frame members of a CDSP-2 type cyclone shelter

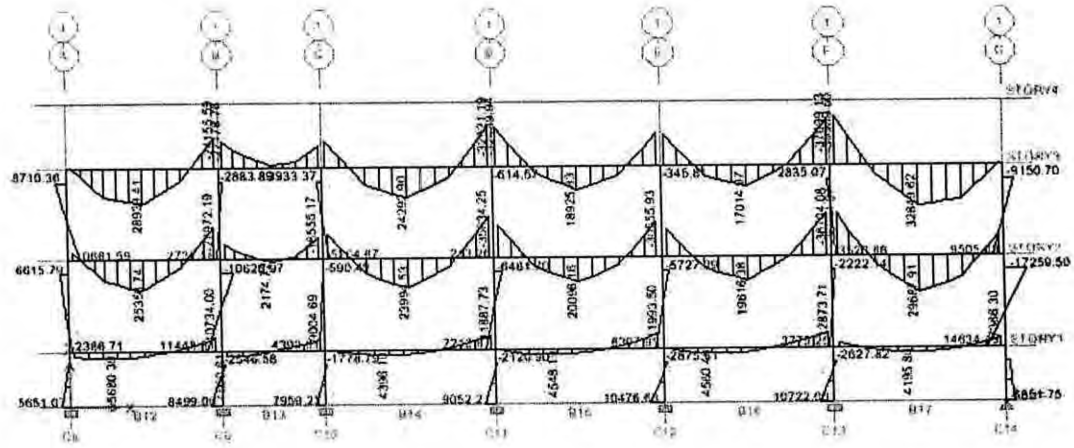

Figure 4.35: Moment Diagram of a CDSP-2 type cyclone shelter

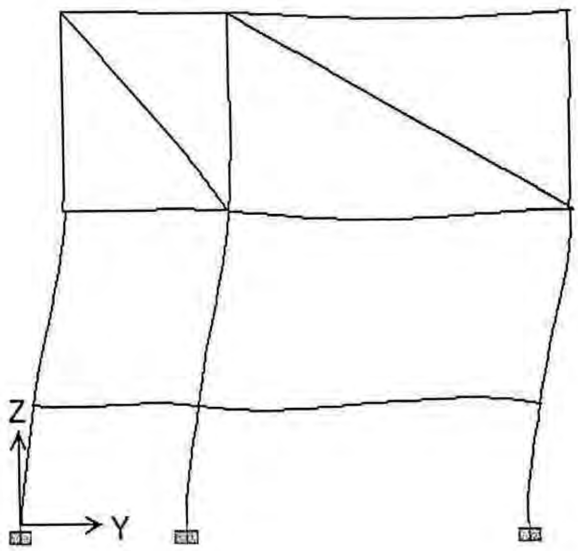

Figure 4.36: Deflected Shape of a CDSP-2 type cyclone shelter

Tables 4.11 and 4.12 show the reinforcement requirement of columns for different loading conditions for two types of model-(1) without bracing and (2) with bracing. From the results it is found that for 3m high tsunami some columns will fail. If strut is considered instead of wall the required steel reduces but it is not significant. Time History loading effect is not significant for columns. But it is seen that a number beams are failing due to torsion effect on beams for all categories of loading.

However, in reality, beam torsion is largely balanced due to the underlying masonry infill.

Table 4.11: Beam and column condition of a typical CDSP-2 Type cyclone shelter (Model without bracing) due to Tsunami Load

Height of Tsunami	Inadequate Beams	Beam Vulnerability Condition	Inadequate Columns	Maximum Required Reinforcement in Column
1m	B1,5,18-21,23	Torsion	–	Ok
2m	B1,5,18-21,23	Torsion	–	Ok
3m	B14-17,24, 26-29, 23,1,5, 7,10,11,18-21	Torsion	C1-5,8,9,14-17	C9= 5.94%
			C6,13,18-20	>8%
Time History loading for 1.18m tsunami	B1,5, 18-21,23	Torsion	–	Ok

Table 4.12: Beam and column condition of a typical CDSP-2 Type cyclone shelter (Model with bracing) due to Tsunami Load

Height of Tsunami	Inadequate Beams	Beam Vulnerability Condition	Inadequate Columns	Maximum Required Reinforcement in Column
1m	B1,5, 18-21,23	Torsion	–	Ok
2m	B1,5,18-21,23	Torsion	–	Ok
3m	B1,5,14-16, 18-21,23,24, 26-29	Torsion	C1-5,8,9,14-17	C17= 5.75%
			C6,13,18-20	>8%
Time History loading for 1.18m tsunami	B1,5,18-21,23	Torsion	–	Ok

4.8 LGED-2 type cyclone shelter

This is a three-storied building with open ground story. The ground story height is about 10 ft. Figures 4.37 and 4.38 show, respectively, the 3D view without bracing and 3D view with bracing. Plan of the FE model of the building is shown in Figure 4.39. Figure 4.40 shows the required rebar percentage of the frame member of the

same structure. Moment diagram and deflected shape are shown in Figure 4.41 and 4.42 respectively.

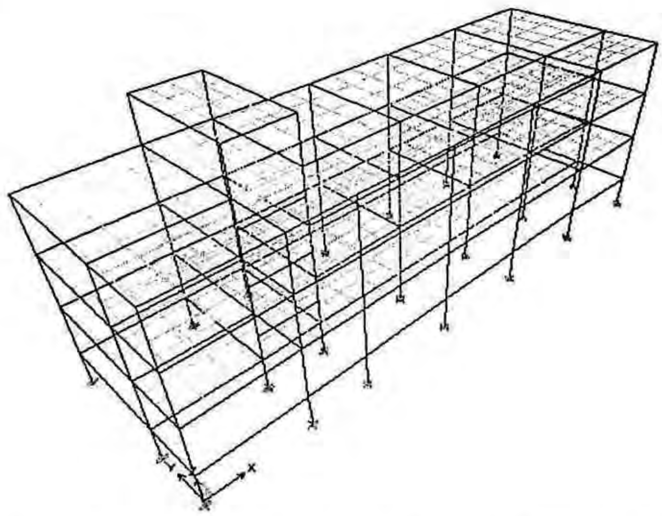

Figure 4.37: 3D view of the FE model of an LGED-2 type cyclone shelter (without bracing)

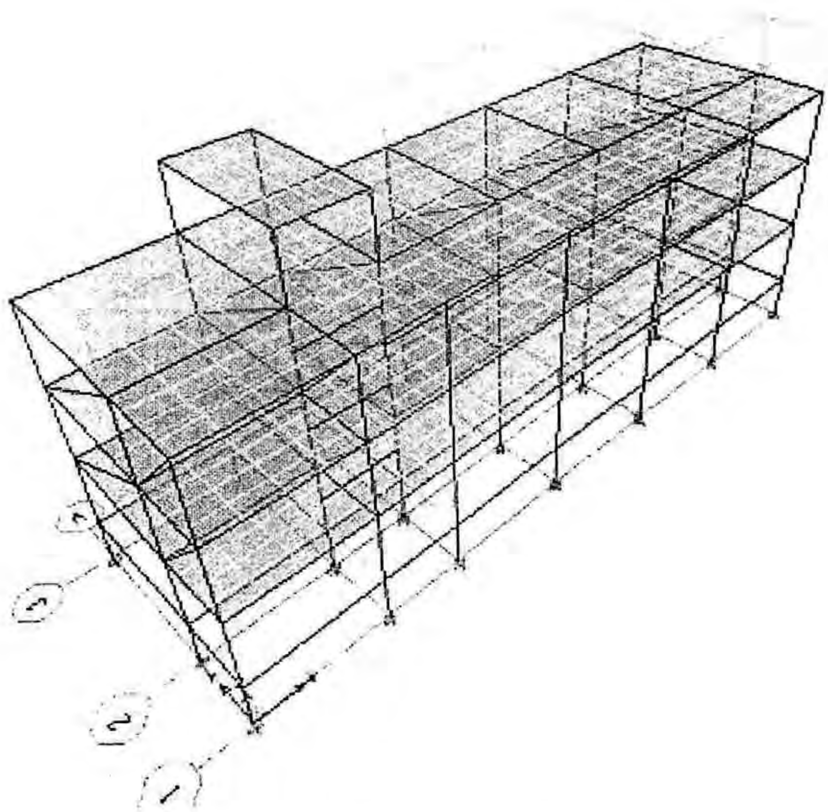

Figure 4.38: 3D view of the FE model of an LGED-2 type cyclone shelter (with bracing)

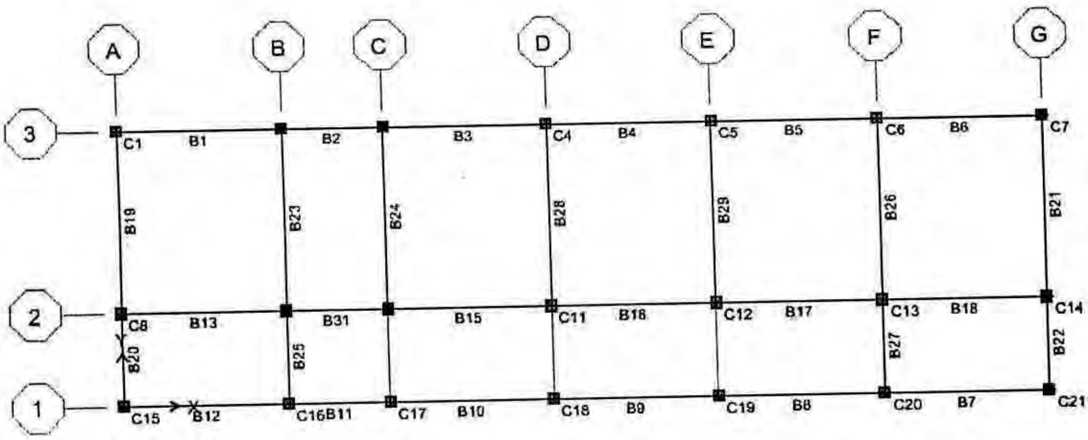

Figure 4.39: Plan of an LGED-2 type cyclone shelter

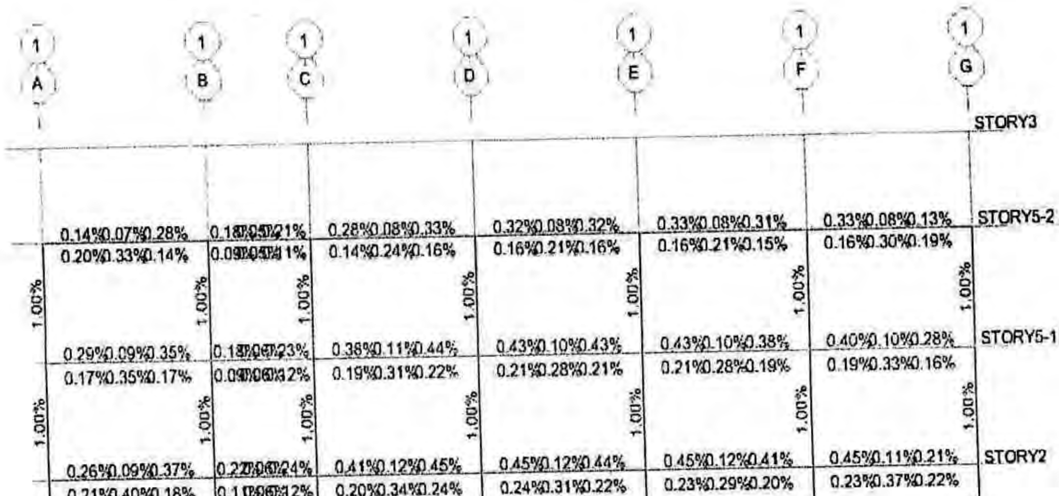

Figure 4.40: Required rebar percentage of frame members of an LGED-2 type cyclone shelter

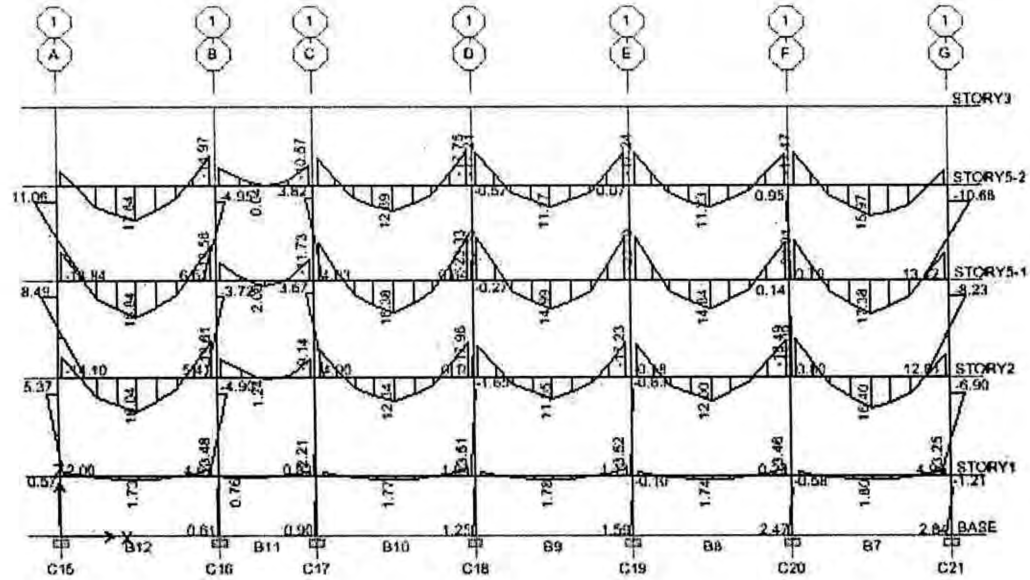

Figure 4.41: Moment Diagram of an LGED-2 type cyclone shelter

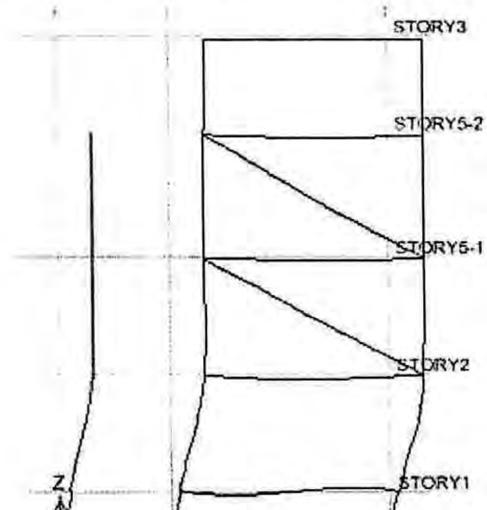

Figure 4.42: Deflected Shape of an LGED-2 type cyclone shelter

Tables 4.13 and 4.14 show the reinforcement requirement of columns for different loading conditions for two types of model-(1) without bracing and (2) with bracing. From the results it is found that for 3m high tsunami some columns fail and maximum reinforcement is required for column is 5.20%. If strut is considered instead of wall the required steel reduces but it is not significant. Time History loading effect is not significant for columns.

Table 4.13: Beam and column condition of a typical LGED-2 Type cyclone (Model without bracing) shelter due to Tsunami Load

Height of Tsunami	Inadequate Beams	Beam Vulnerability Condition	Inadequate Columns	Maximum Required Reinforcement in Column
1m	–	–	–	Ok
2m	–	–	–	Ok
3m	–	–	C8-14,15,21	C14= 5.20%
Time History loading for 1.18m tsunami	–	–	–	Ok

Table 4.14: Beam and column condition of a typical LGED-2 Type cyclone shelter (Model with bracing) due to Tsunami Load

Height of Tsunami	Inadequate Beams	Beam Vulnerability Condition	Inadequate Columns	Maximum Required Reinforcement in Column
1m	–	–	–	Ok
2m	–	–	–	Ok
3m	–	–	C8-14,21	C14= 5.16%
Time History loading for 1.18m tsunami	–	–	–	Ok

4.9 Cyclone shelter funded by German

This is a two-storied building with planar regularity. The open ground story height is about 11 ft. Figures 4.43 and 4.44 show, respectively, the 3D view without bracing and 3D view with bracing. Plan of the FE model of the building is shown in Figure 4.45. Figure 4.46 shows the required rebar percentage of the frame member of the

same structure. Moment diagram and deflected shape are shown in Figure 4.47 and 4.48 respectively.

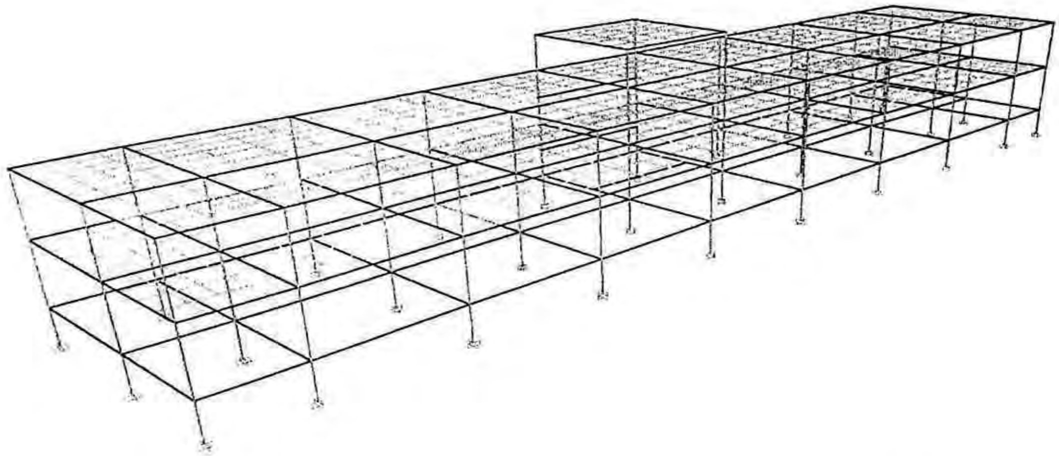

Figure 4.43: 3D view of the FE model of a typical cyclone shelter funded by Germany (without bracing)

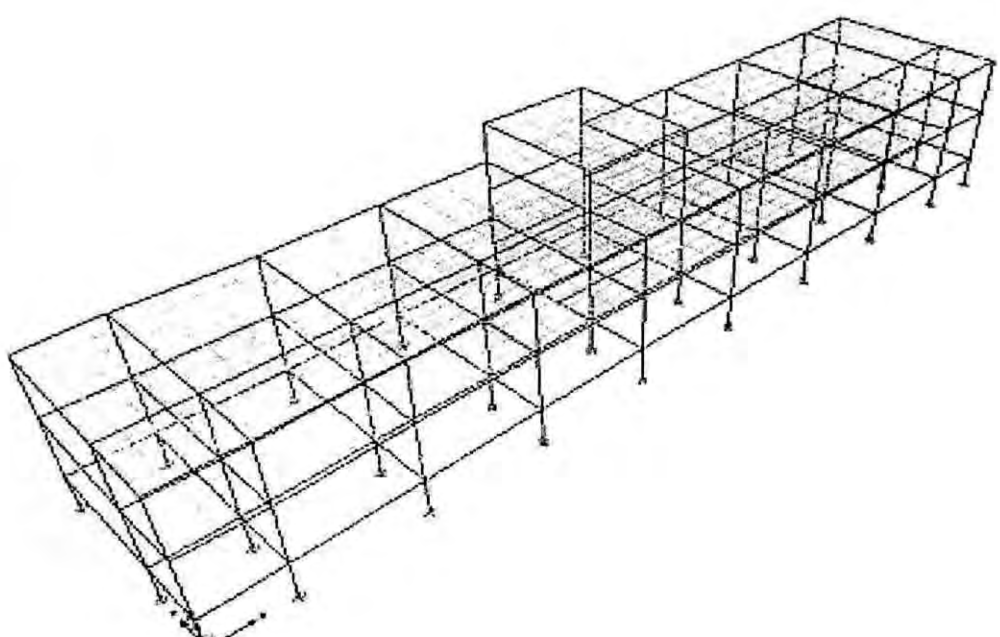

Figure 4.44: 3D view of the FE model of a typical cyclone shelter funded by Germany (with bracing)

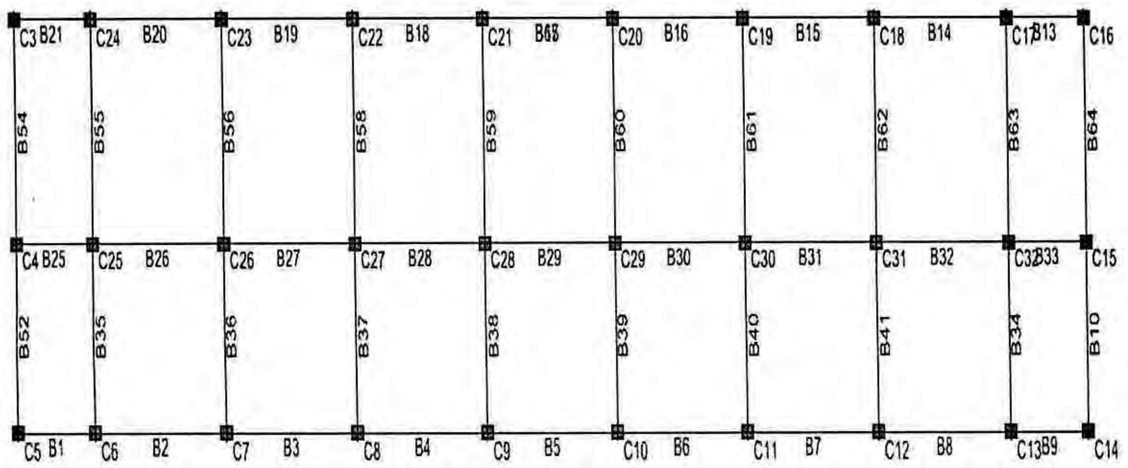

Figure 4.45: Plan of a typical cyclone shelter funded by Germany

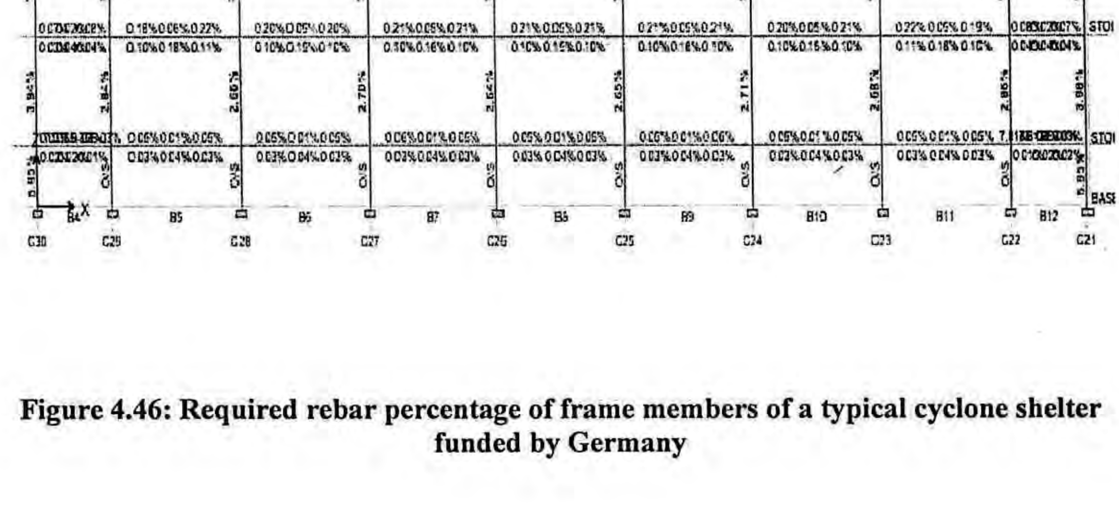

Figure 4.46: Required rebar percentage of frame members of a typical cyclone shelter funded by Germany

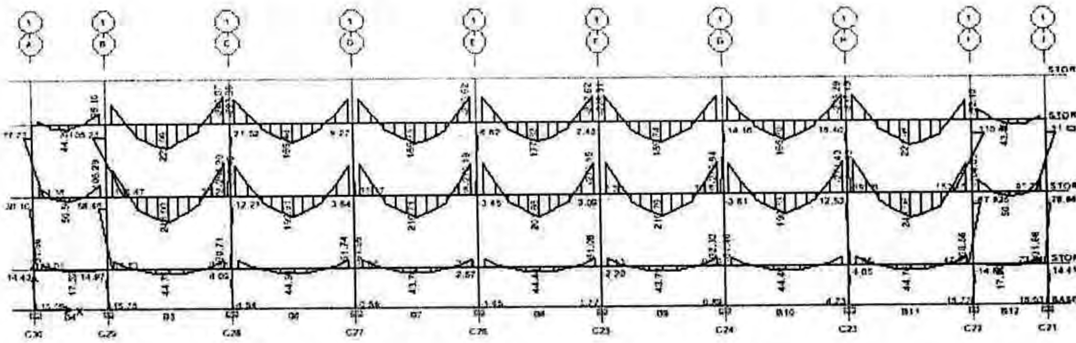

Figure 4.47: Moment Diagram of a typical cyclone shelter funded by Germany

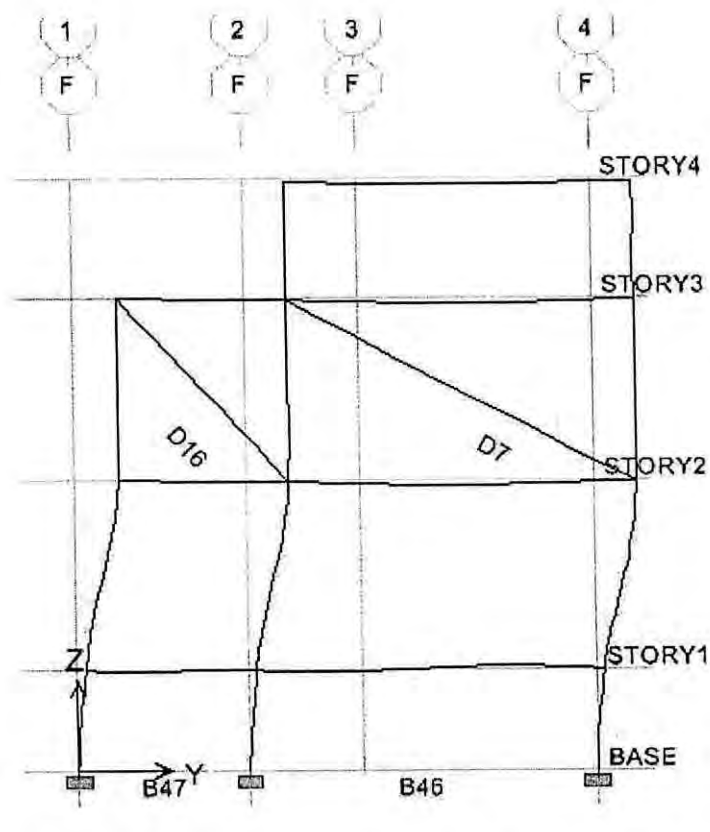

Figure 4.48: Deflected Shape of a typical cyclone shelter funded by Germany

Tables 4.15 and 4.16 show the reinforcement requirement of columns for different loading conditions for two types of model-(1) without bracing and (2) with bracing. From the results it is found that for 3m high tsunami some columns fail and maximum reinforcement required for column is 4.13%. If strut is considered instead of wall the required steel reduces but it is not significant. Time History loading effect is not significant for columns.

Table 4.15: Beam and column condition of a typical cyclone shelter (Model without bracing) funded by Germany due to Tsunami Load

Height of Tsunami	Inadequate Beams	Beam Vulnerability Condition	Inadequate Columns	Maximum Required Reinforcement in Column
1m	–	–	–	Ok
2m	–	–	–	Ok
3m	–	–	C9,21,30	4.13%
Time History loading for 1.18m tsunami	–	–	–	Ok

Table 4.16: Beam and column condition of a typical cyclone shelter (Model with bracing) funded by Germany due to Tsunami Load

Height of Tsunami	Inadequate Beams	Beam Vulnerability Condition	Inadequate Columns	Maximum Required Reinforcement in Column
1m	–	–	–	Ok
2m	–	–	–	Ok
3m	–	–	C9,21,30	3.98%
Time History loading for 1.18m tsunami	–	–	–	Ok

4.10 BDRCS cyclone shelter

This is a two-storied building with open ground story. The ground story height is about 10 ft. Figures 4.49 and 4.50 show, respectively, the 3D view without bracing and 3D view with bracing. Plan of the FE model of the building is shown in Figure 4.51. Figure 4.52 shows the required rebar percentage of the frame member of the same structure. Moment diagram and deflected shape are shown in Figure 4.53 and 4.54 respectively.

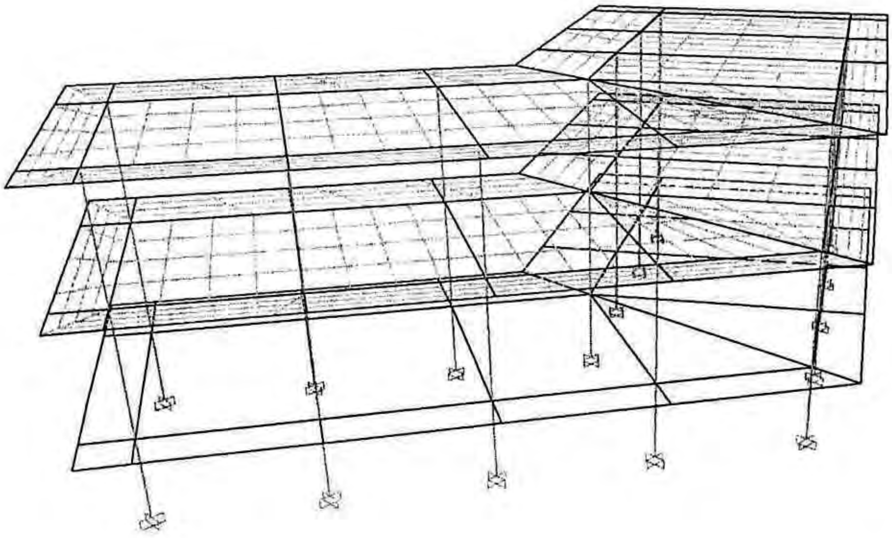

Figure 4.49: 3D view of the FE model of the BDRCS cyclone shelter (without bracing)

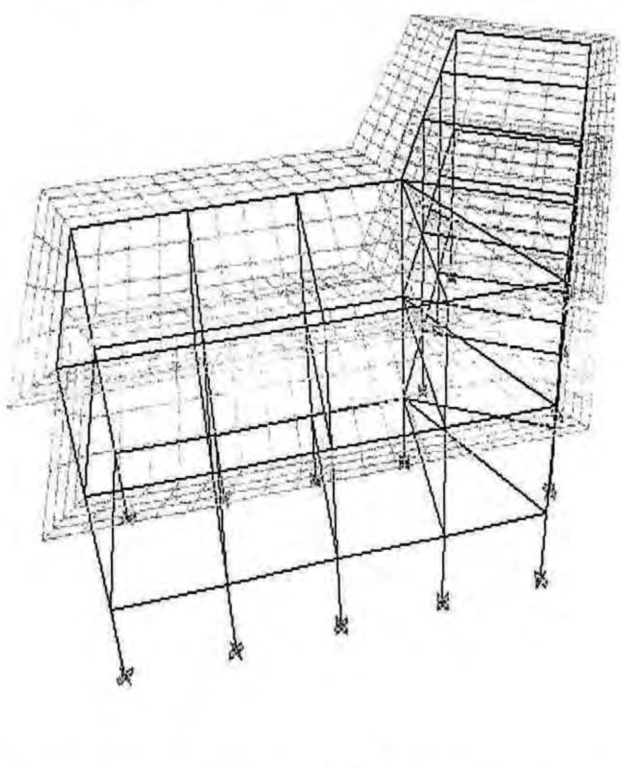

Figure 4.50: 3D view of the FE model of the BDRCS cyclone shelter (with bracing)

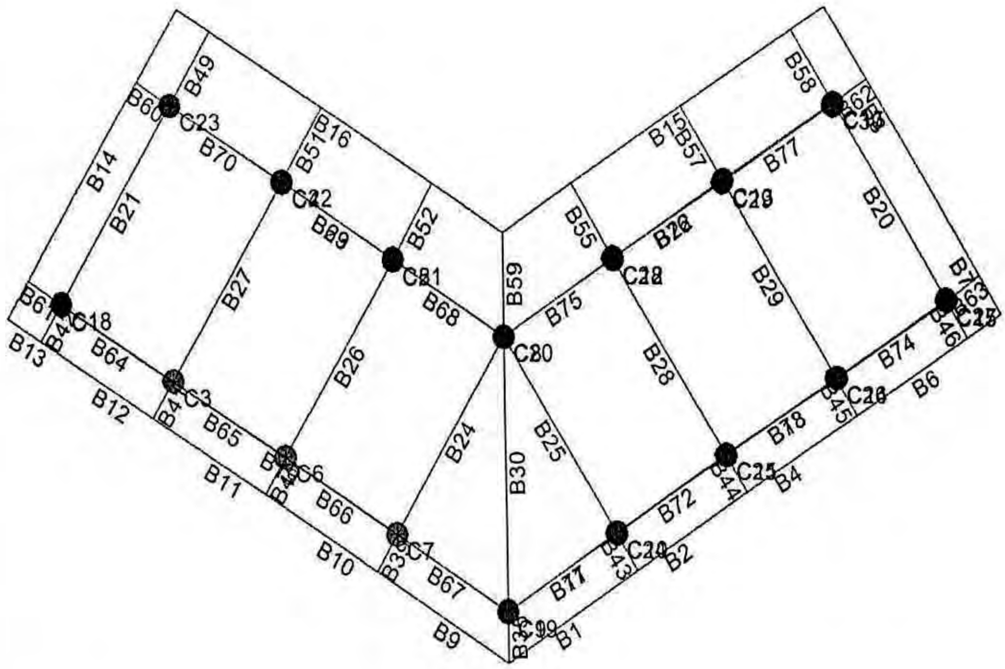

Figure 4.51: Plan of the BDRCS cyclone shelter

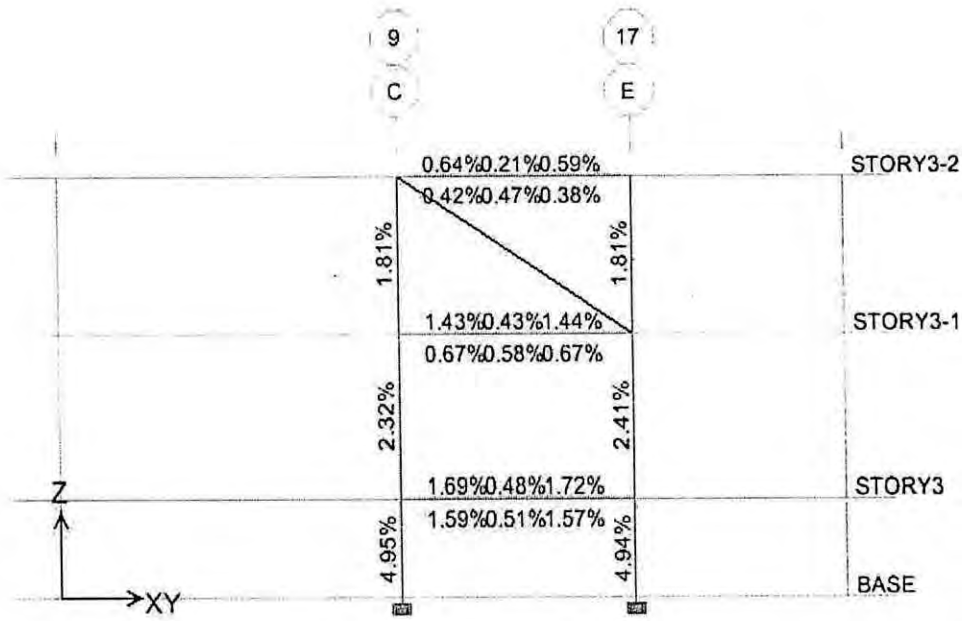

Figure 4.52: Required rebar percentage of frame members of the BDRCS cyclone shelter

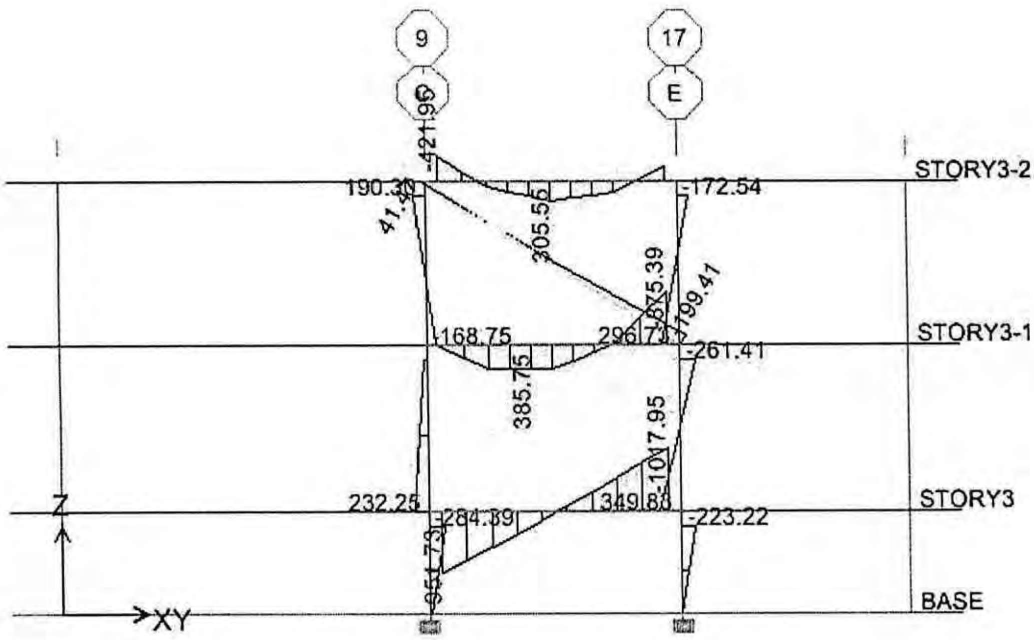

Figure 4.53: Moment Diagram of the BDRCS cyclone shelter

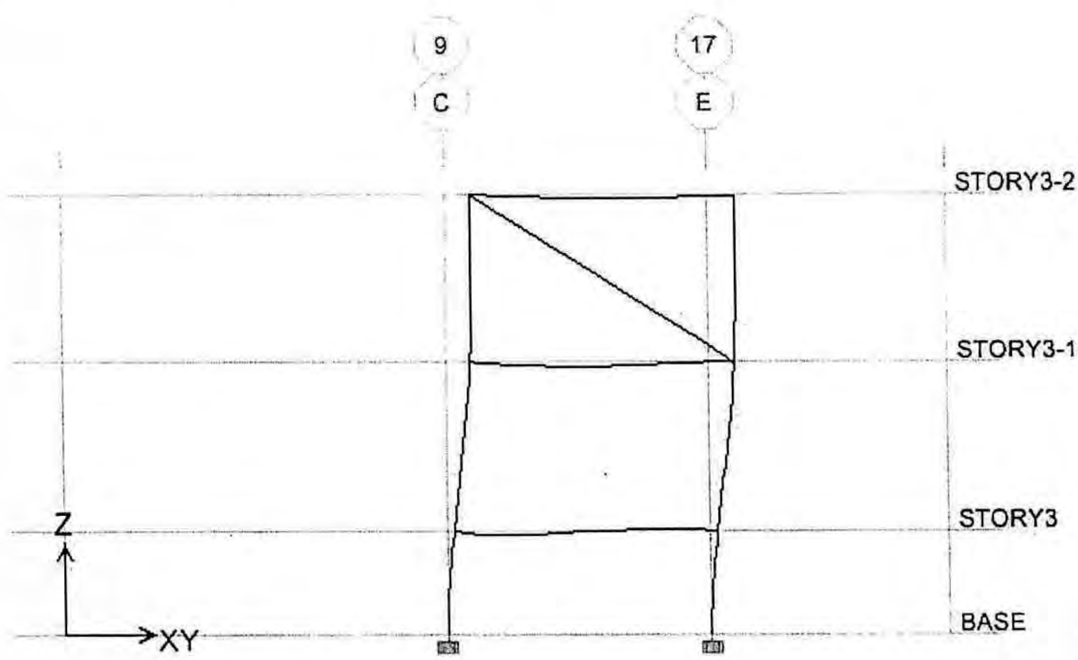

Figure 4.54: Deflected Shape of the BDRCS cyclone shelter

Tables 4.17 and 4.18 show the reinforcement requirement of columns for different loading conditions for two types of model-(1) without bracing and (2) with bracing. From the results it is found that for 3m high tsunami column C8 fails and maximum reinforcement required for column is 3.82%. If strut is considered instead of wall the required steel reduces but it is not significant. Time History loading effect is not significant for columns. But it is seen that a number beams are failing due to torsion effect on beams for all categories of loading. However, in reality, beam torsion is largely balanced due to the underlying masonry infill.

Table 4.17: Beam and column condition of a typical BDRCS Type cyclone shelter (Model without bracing) due to Tsunami Load

Height of Tsunami	Inadequate Beams	Beam Vulnerability Condition	Inadequate Columns	Maximum Required Reinforcement in Column
1m	B48-53, 46,54,56	Torsion	-	Ok
2m	B48-53, 46,54,56	Torsion	-	Ok
3m	B48-53, 54,56,46, 47, 44	Torsion	C8	3.82%
Time History loading for 1.18m tsunami	B48-53, 46,54,56	Torsion	-	Ok

Table 4.18: Beam and column condition of a typical BDRCS Type cyclone shelter (Model with bracing) due to Tsunami Load

Height of Tsunami	Inadequate Beams	Beam Vulnerability Condition	Inadequate Columns	Maximum Required Reinforcement in Column
1m	B48-53,46,54, 56	Torsion	-	Ok
2m	B48-53,46,54, 56	Torsion	-	Ok
3m	B48-53,46,47,54, 56	Torsion	C8	3.49%
Time History loading for 1.18m tsunami	B48-53,46,54, 56	Torsion	-	Ok

4.11 Type-D cyclone shelter

This is a two-storied building with planar regularity. The open ground story height is about 22 ft. Figures 4.55 and 4.56 show, respectively, the 3D view without bracing and 3D view with bracing. Plan of the FE model of the building is shown in Figure 4.57. Figure 4.58 shows the required rebar percentage of the frame member of the same structure. Moment diagram and deflected shape are shown in Figure 4.59 and 4.60 respectively.

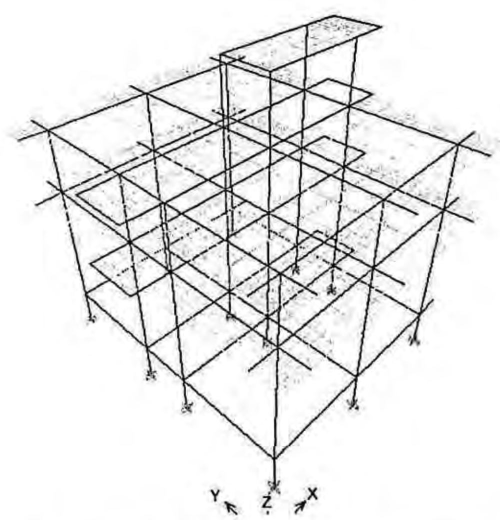

Figure 4.55: 3D view of the FE model of a Type-D cyclone shelter (without bracing)

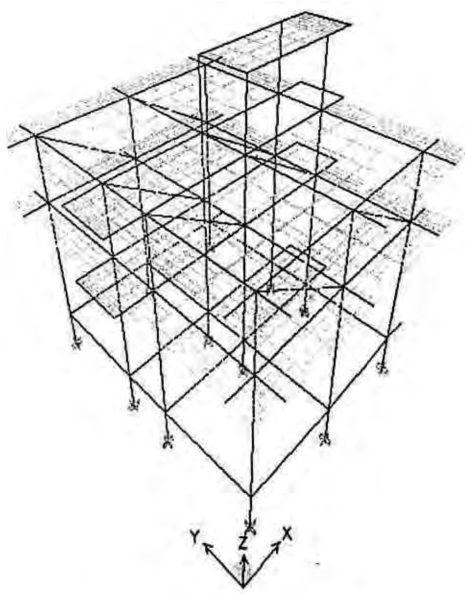

Figure 4.56: 3D view of the FE model of a Type-D cyclone shelter (with bracing)

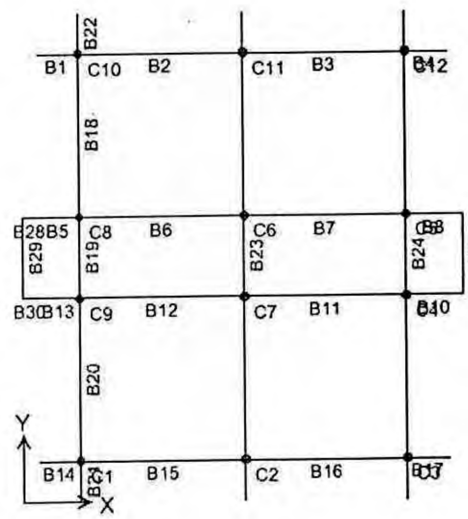

Figure 4.57: Plan of a Type-D cyclone shelter

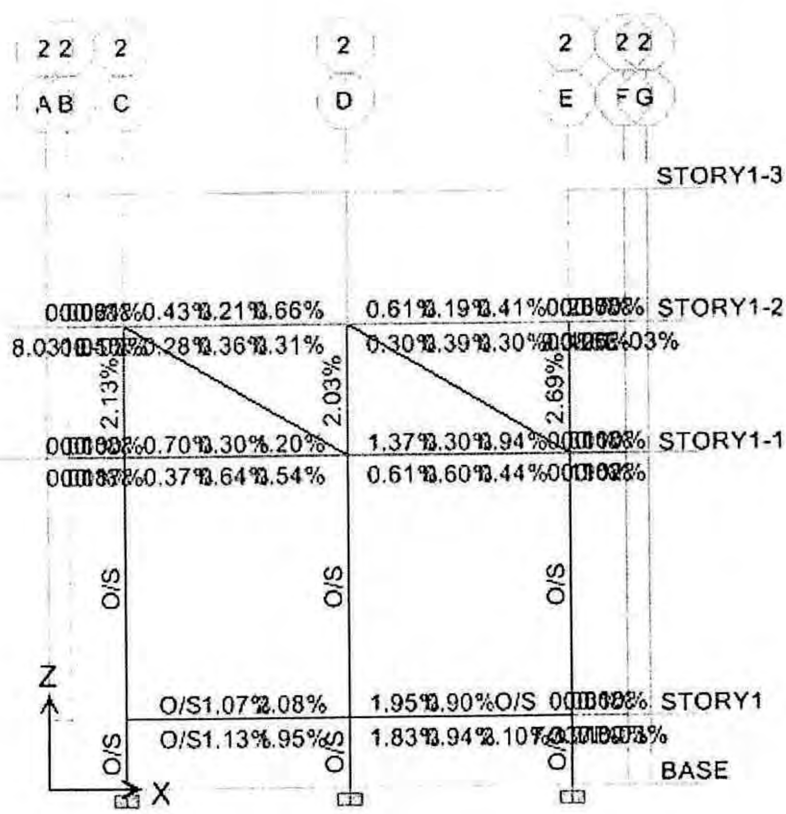

Figure 4.58: Required rebar percentage of frame members of a Type-D cyclone shelter

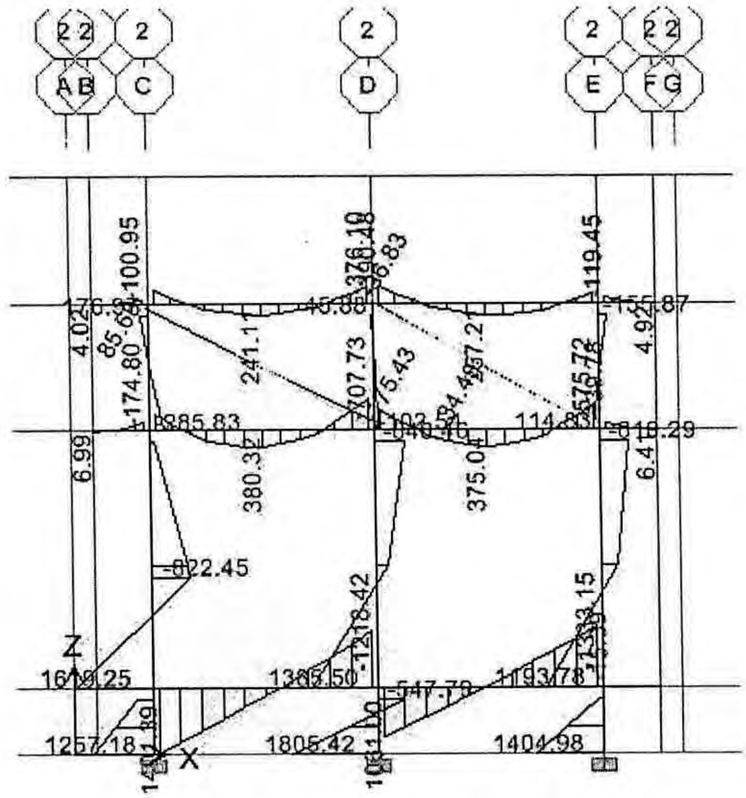

Figure 4.59: Moment Diagram of a Type-D cyclone shelter

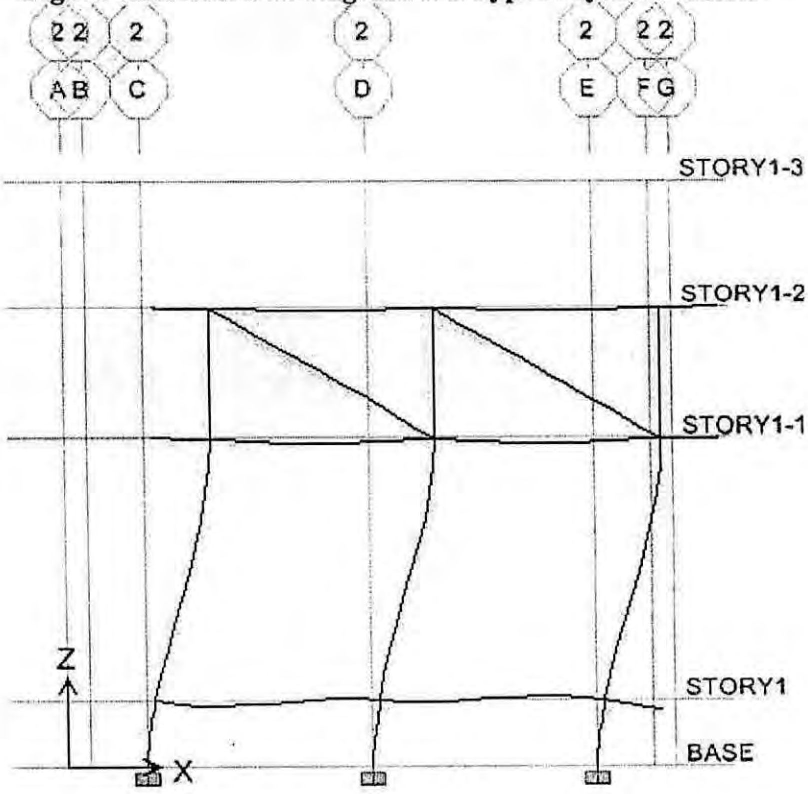

Figure 4.60: Deflected Shape of a Type-D cyclone shelter

Tables 4.19 and 4.20 show the reinforcement requirement of columns for different loading conditions for two types of model-(1) without bracing and (2) with bracing. From the results it is found that even for 1m high tsunami all columns will fail. If strut is considered instead of wall the result is almost same. Time History loading also causes all the columns to fail. A number beams are failing due to torsion effect on beams for all categories of loading. However, in reality, beam torsion is largely balanced due to the underlying masonry infill.

Table 4.19: Beam and column condition of a Type-D cyclone shelter (Model without bracing) due to Tsunami Load

Height of Tsunami	Inadequate Beams	Beam Vulnerability Condition	Inadequate Columns	Maximum Required Reinforcement in Column
1m	B11,7,25	Torsion	All column fails	>8%
2m	B11,7,25	Torsion	All column fails	>8%
3m	B11,7,25,31, 32,33, 35	Torsion	All column fails	>8%
Time History loading for 1.18m tsunami	B11,7,25	Torsion	All column fails	>8%

Table 4.20: Beam and column condition of a Type-D cyclone shelter (Model with bracing) due to Tsunami Load

Height of Tsunami	Inadequate Beams	Beam Vulnerability Condition	Inadequate Columns	Maximum Required Reinforcement in Column
1m	B11,7,25	Torsion	All column fails	>8%
2m	B11,7,25	Torsion	All column fails	>8%
3m	B11,7,25	Torsion	All column fails	>8%
Time History loading for 1.18m tsunami	B11,7,25	Torsion	All column fails	>8%

4.12 Type-E cyclone shelter

This is a two-storied building with planar regularity. The open ground story height is about 22 ft. Figures 4.61 and 4.62 show, respectively, the 3D view without bracing and 3D view with bracing.

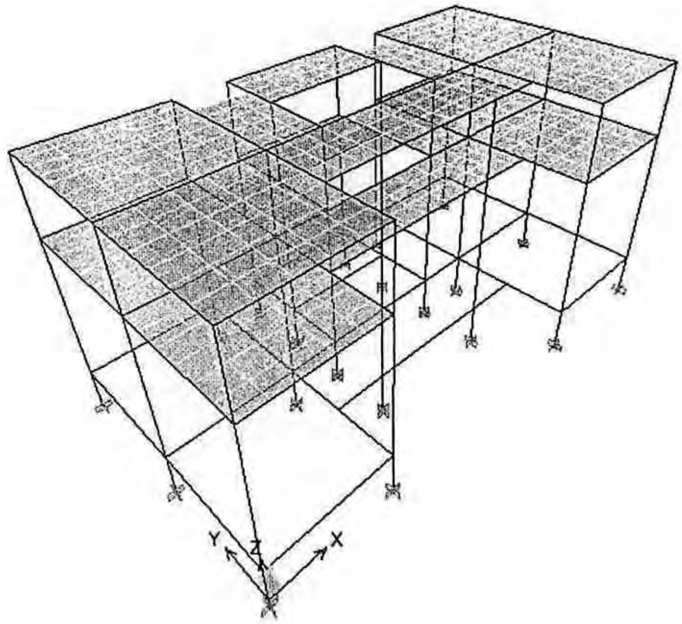

Figure 4.61: 3D view of the FE model of a Type-E cyclone shelter (without bracing)

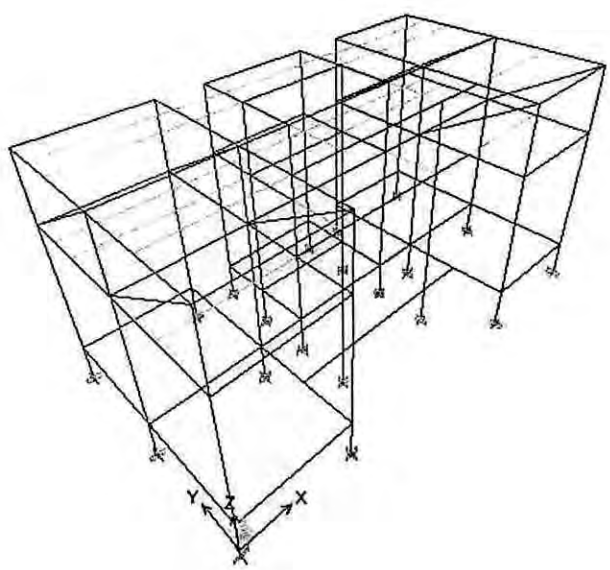

Figure 4.62: 3D view of the FE model of a Type-E cyclone shelter (with bracing)

Plan of the FE model of the building is shown in Figure 4.63. Figure 4.64 shows the required rebar percentage of the frame member of the same structure. Moment diagram and deflected shape are shown in Figure 4.65 and 4.66 respectively.

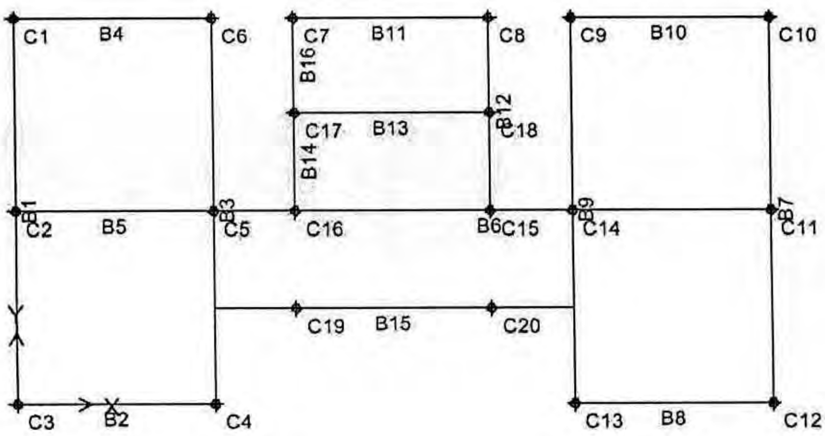

Figure 4.63: Plan of a Type-E cyclone shelter

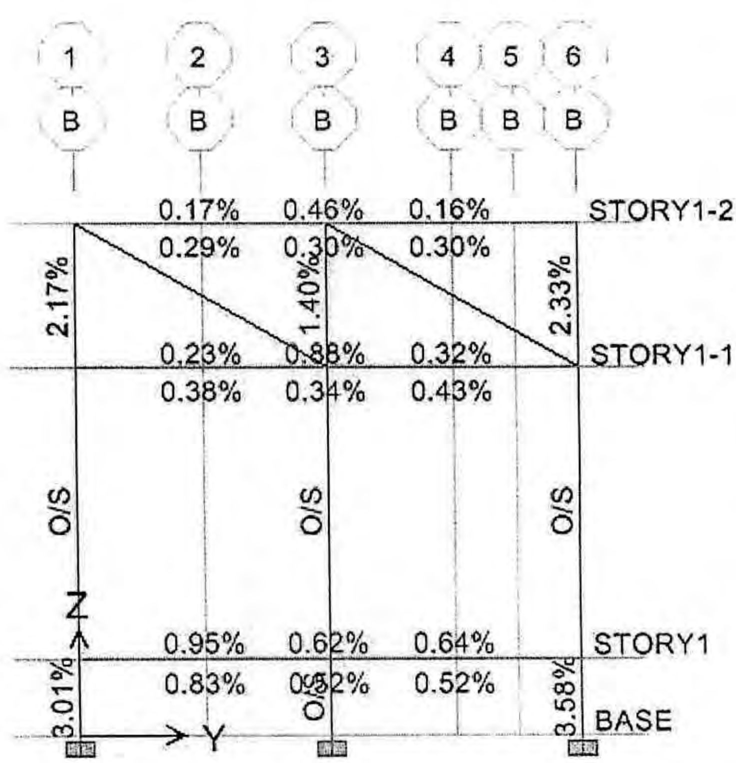

Figure 4.64: Required rebar percentage of frame members of a Type-E cyclone shelter

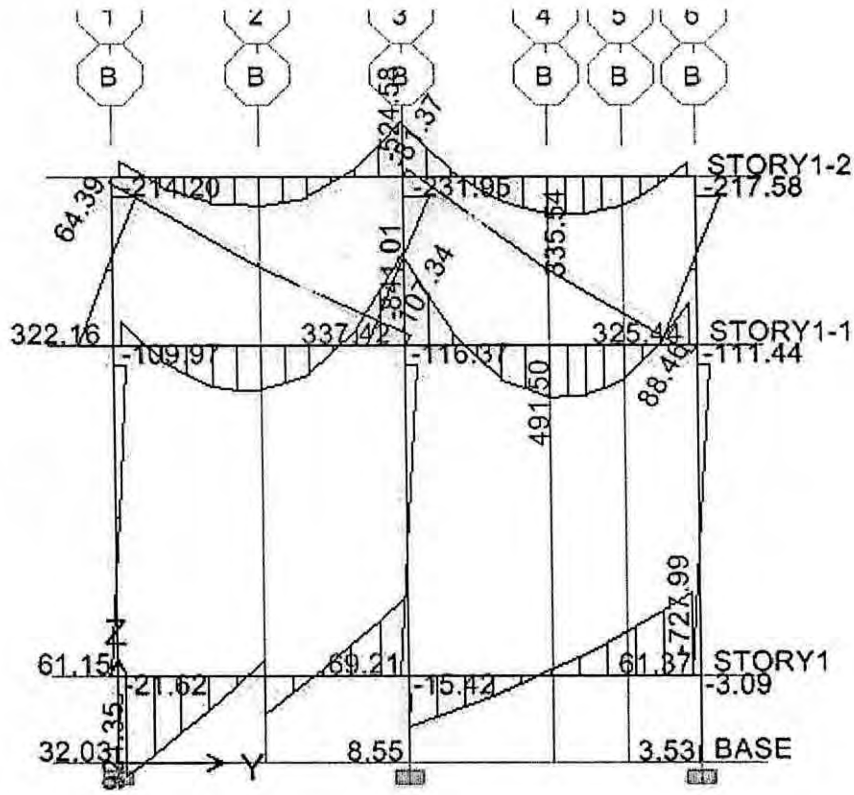

Figure 4.65: Moment Diagram of a Type-E cyclone shelter

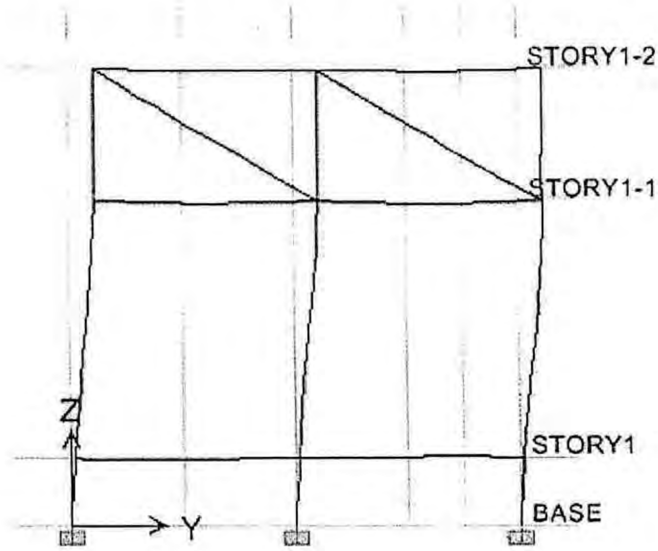

Figure 4.66: Deflected Shape of a Type-E cyclone shelter

Tables 4.21 to 4.22 show the reinforcement requirement of columns for different loading conditions for two types of model-(1) without bracing and (2) with bracing.

From the results it is found that even for 1m high tsunami most of the columns will fail. If strut is considered instead of wall the required steel reduces but it is not significant. Time History loading also causes most of the columns to fail. A number beams are failing due to torsion effect on beams for all categories of loading. However, in reality, beam torsion is largely balanced due to the underlying masonry infill.

Table 4.21: Beam and column condition of a Type-E cyclone shelter (Model without bracing) due to Tsunami Load

Height of Tsunami	Inadequate Beams	Beam Vulnerability Condition	Inadequate Columns	Maximum Required Reinforcement in Column
1m	B4,10,1,7	Torsion	C19,20,2, 5,14,11	>8%
			C3,4,12, 13,17,18	C17,18=3.89%
2m	B4,10,1,7	Torsion	C15,16, 7,8,1	>5.65%
			Others	>8%
3m	B4,10,1,7	Torsion	All	>8%
Time History loading for 1.18m tsunami	B4,10,1,7	Torsion	C19,20,2, 5,14,11	>8%
			C3,4,12,13, 17,18	C17,18=5.12%

Table 4.22: Beam and column condition of a Type-E cyclone shelter (Model with bracing) due to Tsunami Load

Height of Tsunami	Inadequate Beams	Beam Vulnerability Condition	Inadequate Columns	Maximum Required Reinforcement in Column
1m	B4,10,1,7	Torsion	C19,20,2, 5,14,11	>8%
			C3,4,12,13	C4,13=3.82%
2m	B4,10,1,7	Torsion	C15,16, 7,8	C16,15=5.64%
			Others	>8%
3m	B4,10,1,7	Torsion	All	>8%
Time History loading for 1.18m tsunami	B4,10,1,7	Torsion	C19,20,2, 5,14,11	>8%
			C3,4,12,13, 18	C4,13=5.12%

4.13 Type-J cyclone shelter

This is a two-storied building with planar regularity. The open ground story height is about 22 ft. Figures 4.67 and Figure 4.68 show, respectively, the 3D view without bracing and 3D view with bracing. Plan of the FE model of the building is shown in Figure 4.69. Figure 4.70 shows the required rebar percentage of the frame member of the same structure. Moment diagram and deflected shape are shown in Figure 4.71 and 4.72 respectively.

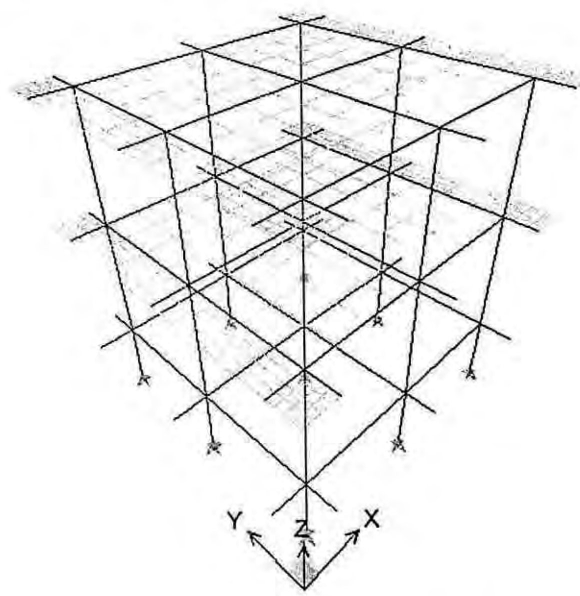

Figure 4.67: 3D view of the FE model of a Type-J cyclone shelter (without bracing)

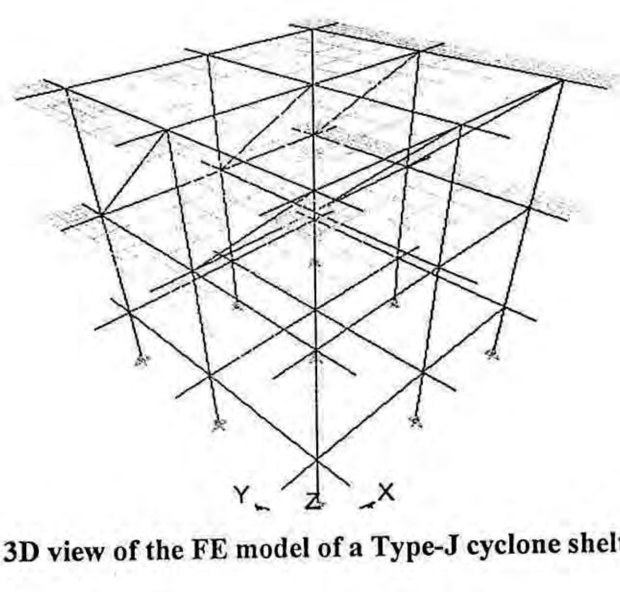

Figure 4.68: 3D view of the FE model of a Type-J cyclone shelter (with bracing)

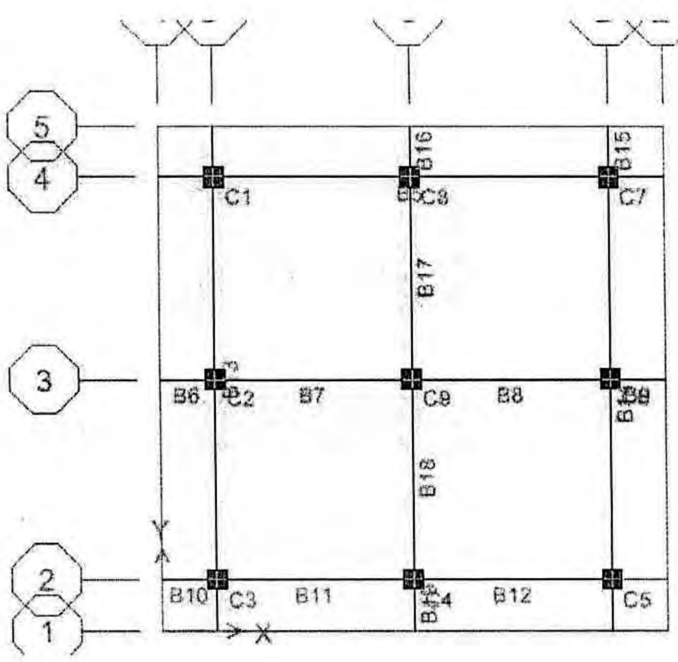

Figure 4.69: Plan of a Type-J cyclone shelter

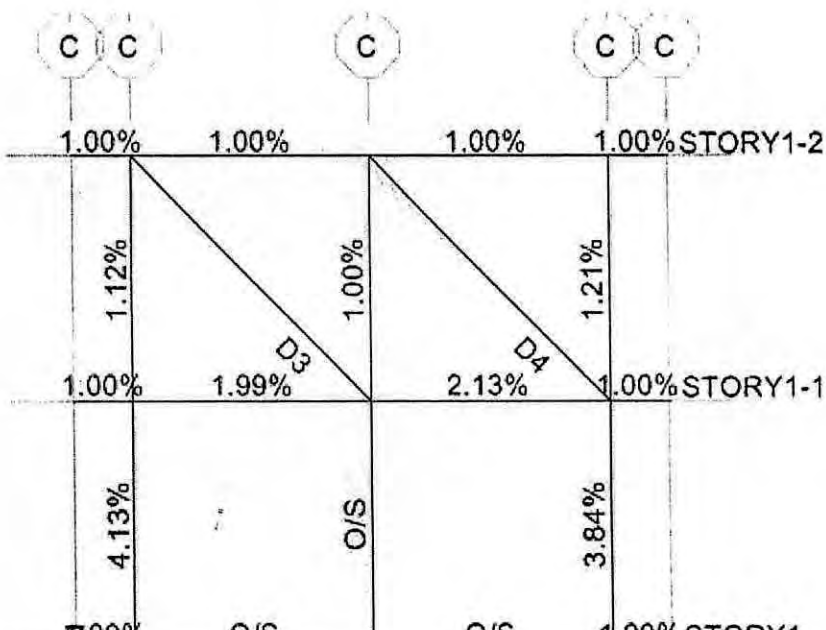

Figure 4.70: Required rebar percentage of frame members of a Type-J cyclone shelter

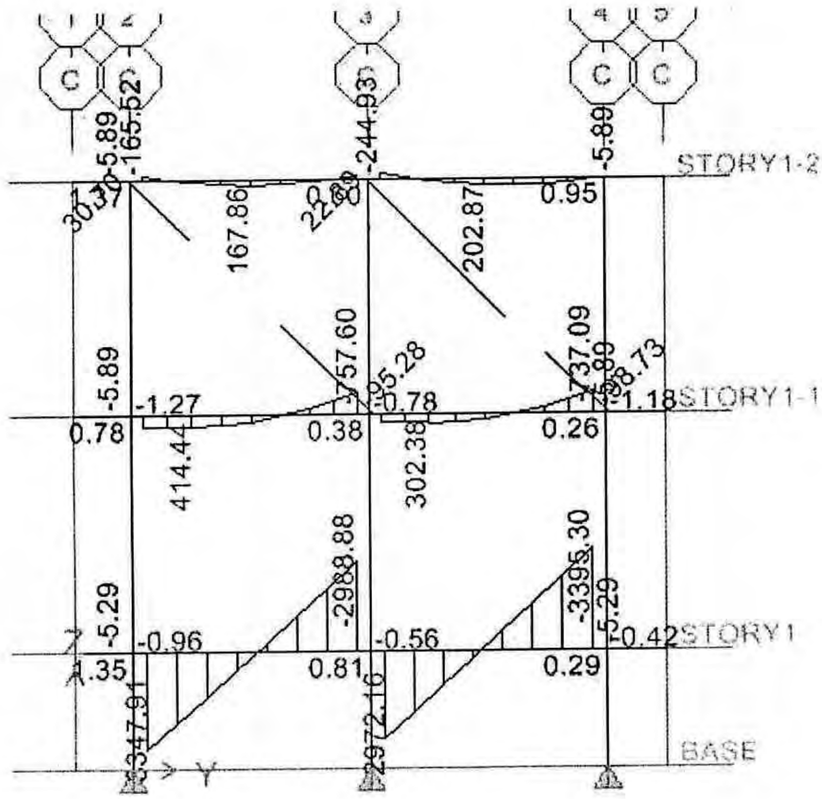

Figure 4.71: Moment Diagram of a Type-J cyclone shelter

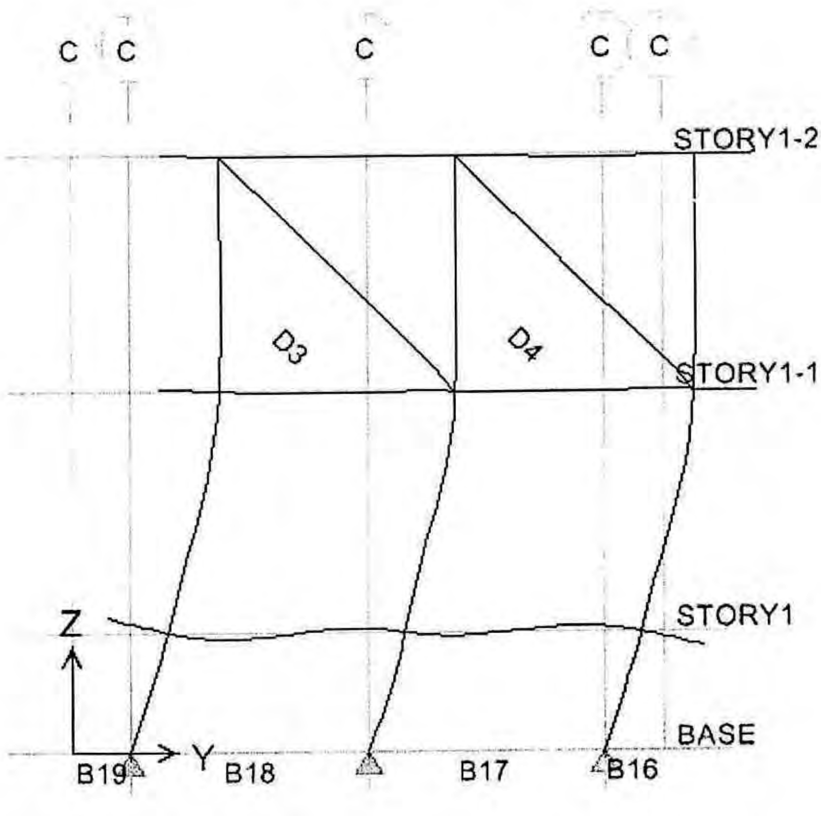

Figure 4.72: Deflected Shape of a Type-J cyclone shelter

Tables 4.23 and 4.24 show the reinforcement requirement of columns for different loading conditions for two types of model-(1) without bracing and (2) with bracing. From the results it is found that for 3m high tsunami some columns fail. If strut is considered instead of wall the required steel reduces but it is not significant. Time History loading effect is not significant for columns

Table 4.23: Beam and column condition of a Type-J cyclone shelter (Model without bracing) due to Tsunami Load

Height of Tsunami	Inadequate Beams	Beam Vulnerability Condition	Inadequate Columns	Maximum Required Reinforcement in Column
1m	–	–	–	Ok
2m	–	–	–	Ok
3m	–	–	C1,3,5,7, 8	C3,5= 4.19%
			C2,6,9	>8%
Time History loading for 1.18m tsunami	–	–	–	Ok

Table 4.24: Beam and column condition of a Type-J cyclone shelter (Model with bracing) due to Tsunami Load

Height of Tsunami	Inadequate Beams	Beam Vulnerability Condition	Inadequate Columns	Maximum Required Reinforcement in Column
1m	–	–	–	Ok
2m	–	–	–	Ok
3m	–	–	C1,3,5,7, 8	C3,5= 4.17%
			C2,6,9	>8%
Time History loading for 1.18m tsunami	–	–	–	Ok

4.14 PWD Type cyclone shelter

These are three storied buildings with open ground story. The ground story height is 17 ft. Figures 4.73 and 4.74 Figure show, respectively, the 3D view without bracing and 3D view with bracing. Plan of the FE model of the building is shown in Figure 4.75. Figure 4.76 shows the required rebar percentage of the frame member of the same structure. Moment diagram and deflected shape are shown in Figure 4.77 and 4.78 respectively.

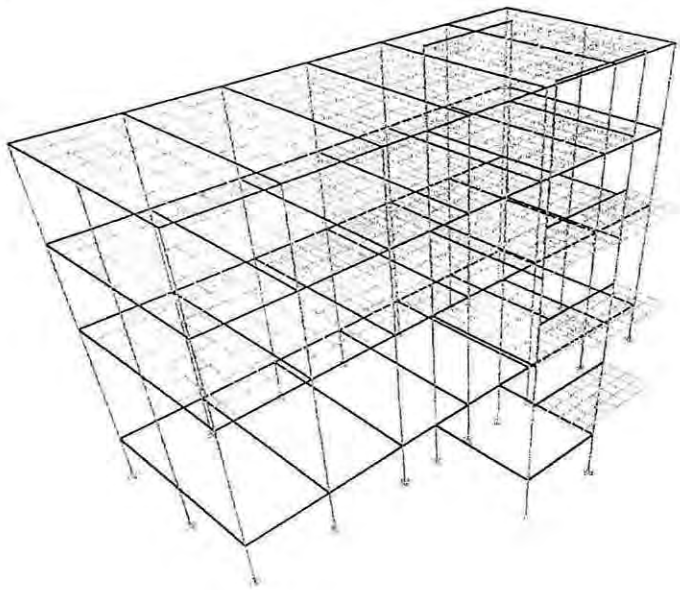

Figure 4.73: 3D view of the FE model of a PWD Type cyclone shelter (without bracing)

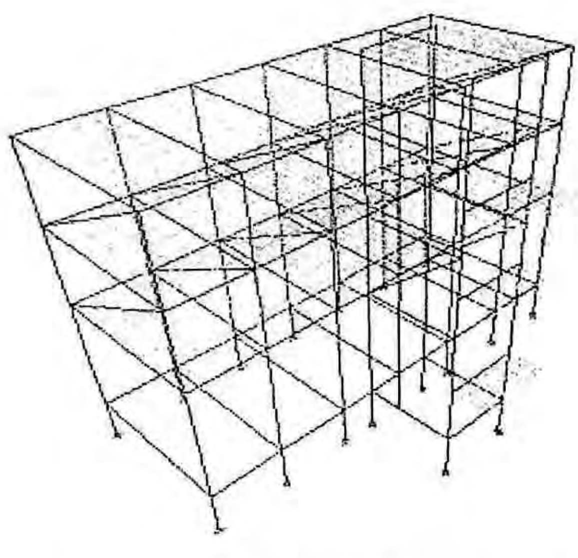

Figure 4.74: 3D view of the FE model of a PWD Type cyclone shelter (with bracing)

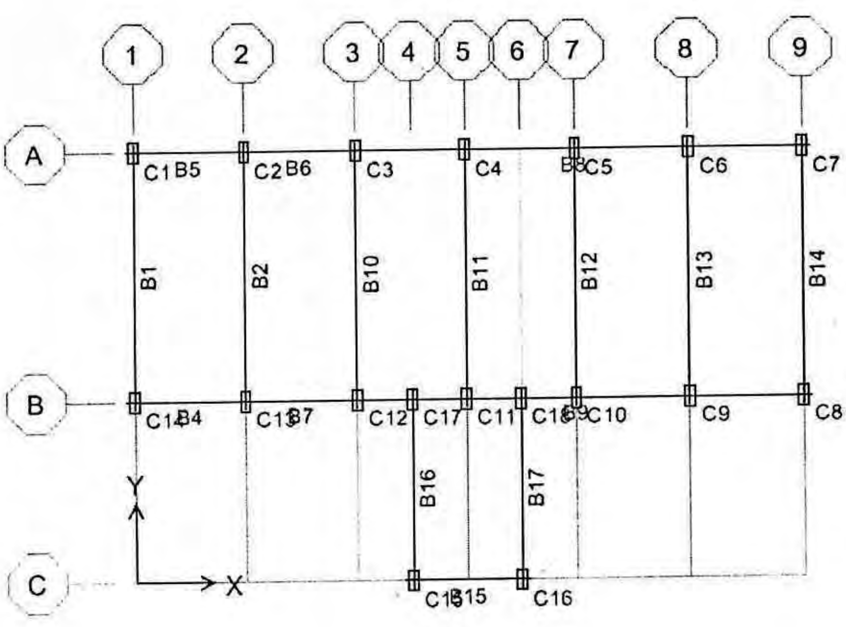

Figure 4.75: Plan of a PWD Type cyclone shelter

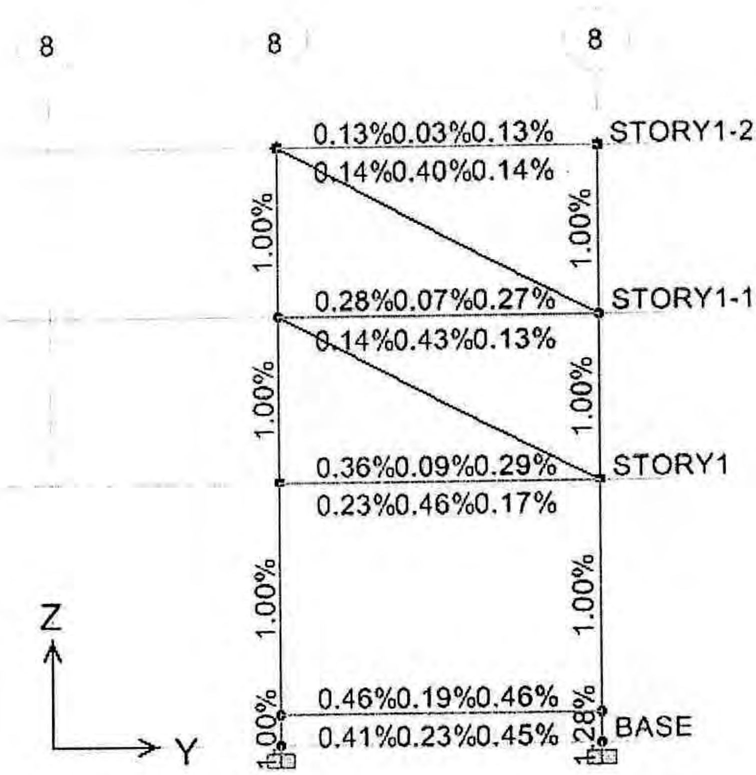

Figure 4.76: Required rebar percentage of frame members of a PWD Type cyclone shelter

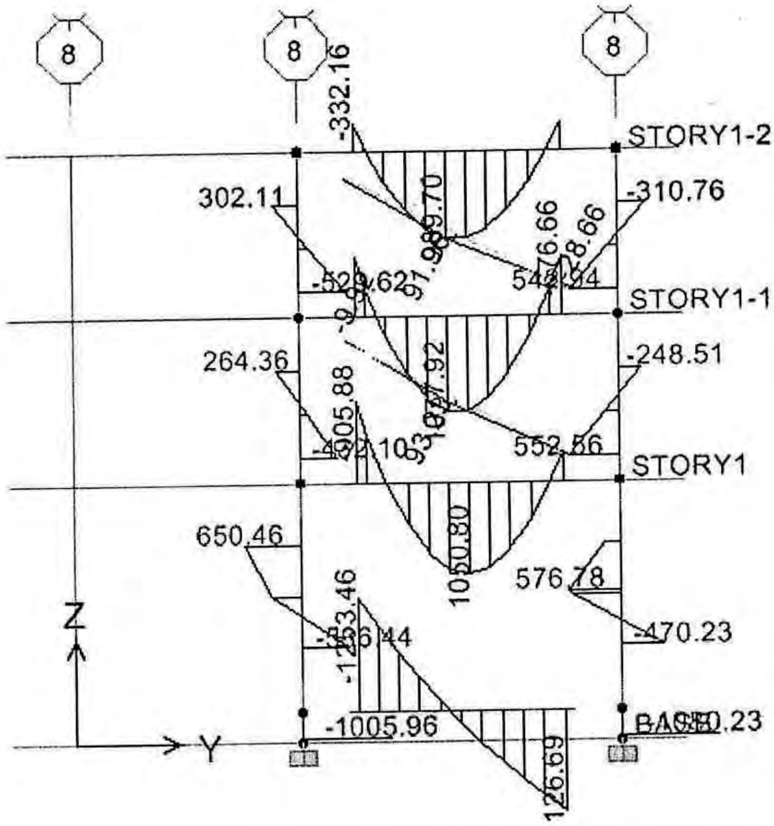

Figure 4.77: Moment Diagram of a PWD Type cyclone shelter

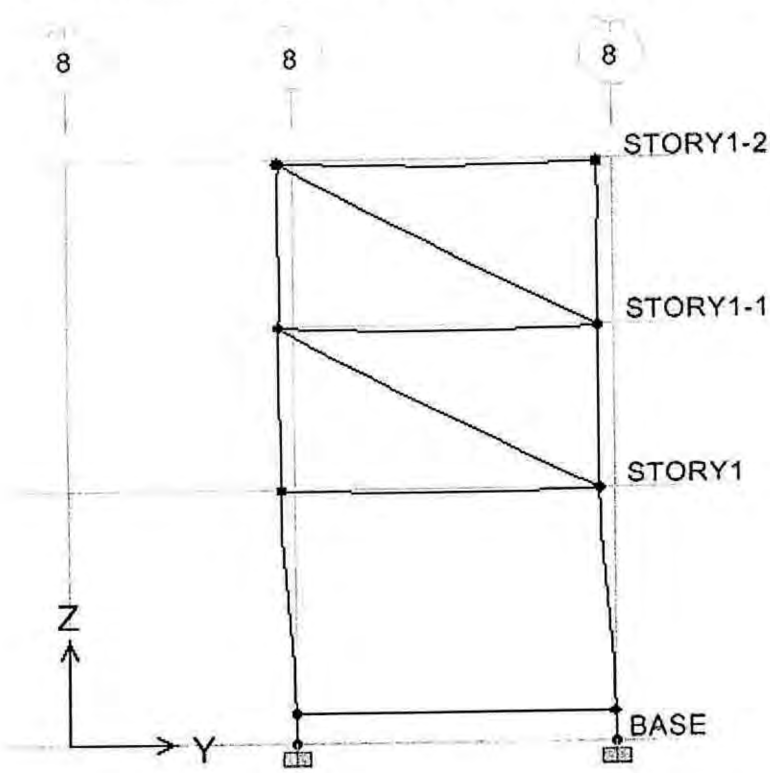

Figure 4.78: Deflected Shape of a PWD Type cyclone shelter

Tables 4.25 and 4.26 show the reinforcement requirement of columns for different loading conditions for two types of model-(1) without bracing and (2) with bracing. From the results it is found that for 3m high tsunami some columns fail. If strut is considered instead of wall the required steel reduces but it is not significant. Time History loading effect is not significant for columns.

Table 4.25: Beam and column condition of a PWD Type cyclone shelter (Model without bracing) due to Tsunami Load

Height of Tsunami	Inadequate Beams	Beam Vulnerability Condition	Inadequate Columns	Maximum Required Reinforcement in Column
1m	–	–	–	Ok
2m	–	–	–	Ok
3m	–	–	C1,7-18	C1,7= 5.79%
			C2-6	>8%
Time History loading for 1.18m tsunami	–	–	–	Ok

Table 4.26: Beam and column condition of a PWD Type cyclone shelter (Model with bracing) due to Tsunami Load

Height of Tsunami	Inadequate Beams	Beam Vulnerability Condition	Inadequate Columns	Maximum Required Reinforcement in Column
1m	–	–	–	Ok
2m	–	–	–	Ok
3m	–	–	C1, 7-18	C1,7= 5.78%
			C2-6	>8%
Time History loading for 1.18m tsunami	–	–	–	Ok

4.15 College building which can be used as a cyclone shelter

This is a two-storied building with open ground story. The ground story height is about 12.50 ft. Figures 4.79 and Figure 4.80 show, respectively, the 3D view without bracing and 3D view with bracing.

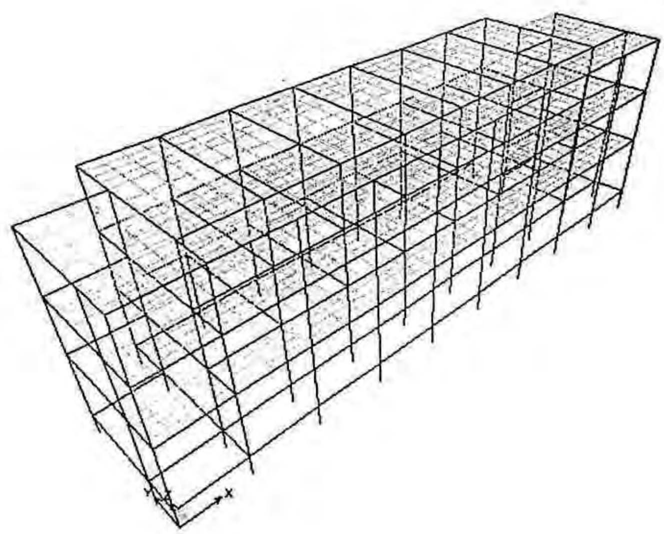

Figure 4.79: 3D view of the FE model of the College building (without bracing)

Plan of the FE model of the building is shown in Figure 4.81. Figure 4.82 shows the required rebar percentage of the frame member of the same structure. Moment diagram and deflected shape are shown in Figure 4.83 and 4.84 respectively.

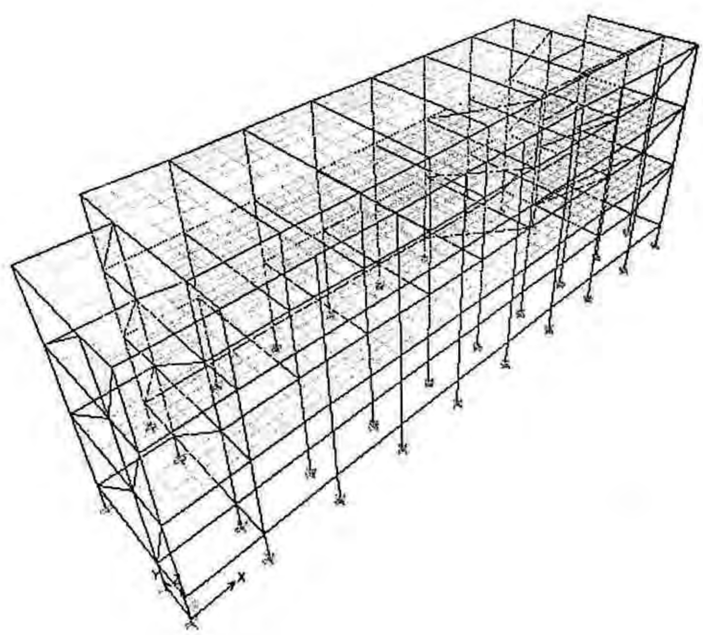

Figure 4.80: 3D view of the FE model of the College building (with bracing)

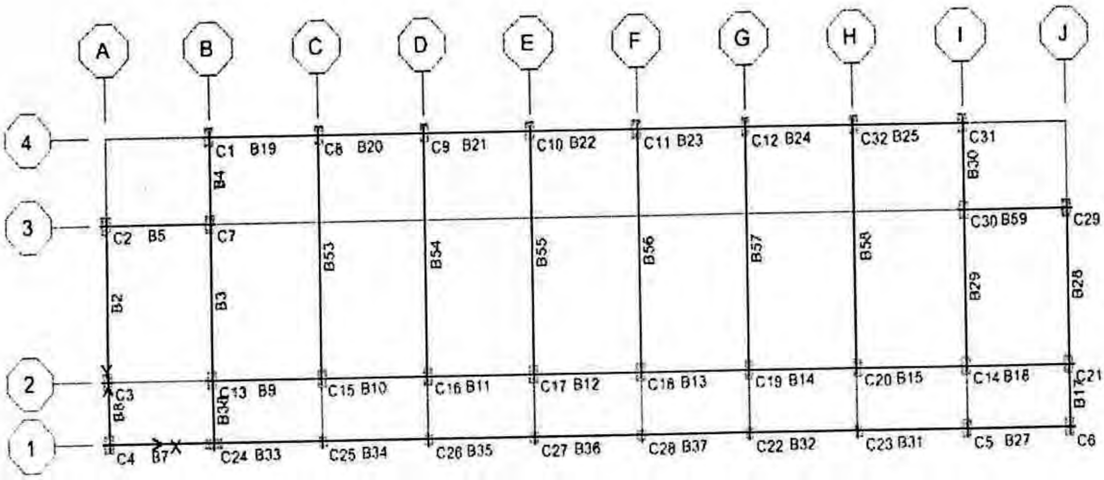

Figure 4.81: Plan of the College building

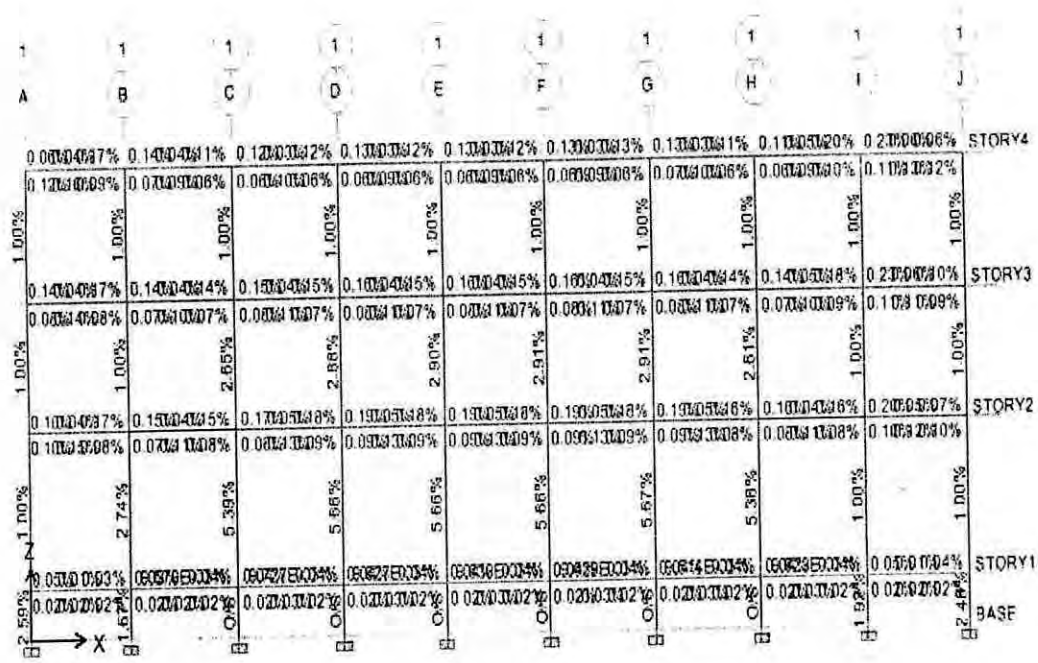

Figure 4.82: Required rebar percentage of frame members of the College building

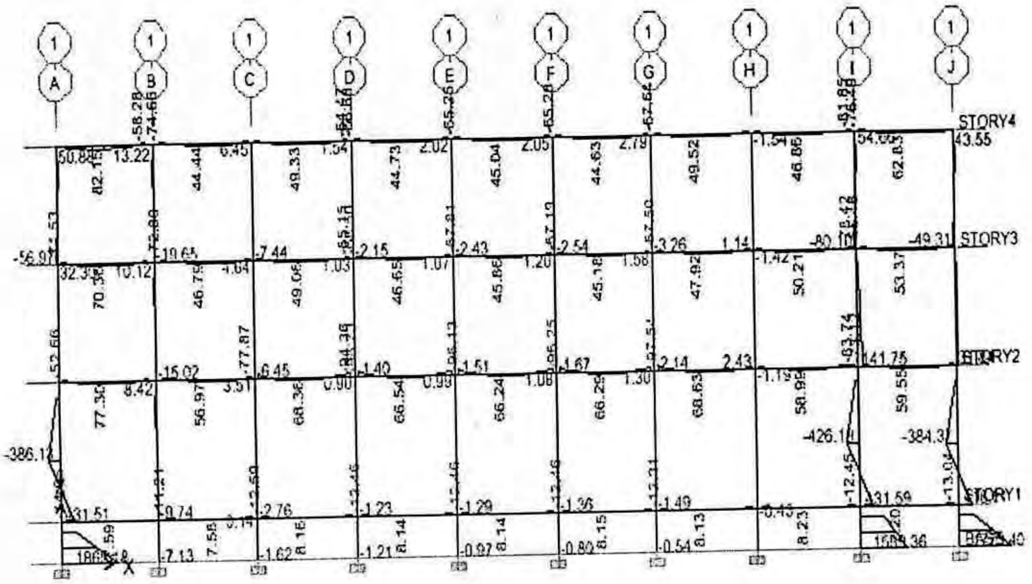

Figure 4.83: Moment Diagram of the College building

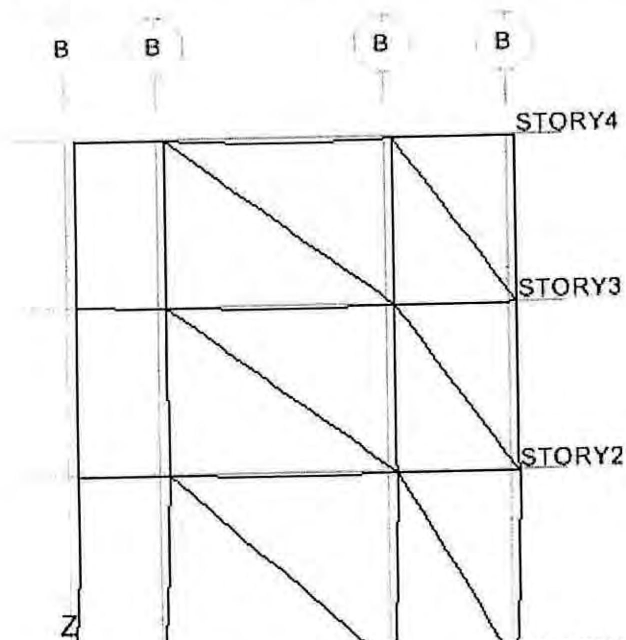

Figure 4.84: Deflected Shape of the College building

Tables 4.27 and 4.28 show the reinforcement requirement of columns for different loading conditions for two types of model-(1) without bracing and (2) with bracing. From the results it is found that for 2m high tsunami some columns fail. If strut is considered instead of wall the required steel reduces but it is not significant. Time History loading effect is not significant for columns.

Table 4.27: Beam and column condition of the College building (Model without bracing) due to Tsunami Load

Height of Tsunami	Inadequate Beams	Beam Vulnerability Condition	Inadequate Columns	Maximum Required Reinforcement in Column
1m	–	–	–	Ok
2m	–	–	C8-13,22-26, 28	C22,26,27,28 = 5.58%
3m	–	–	C22-28, 30	>8%
			Others	C19= 5.92%
Time History loading for 1.18m tsunami	–	–	–	Ok

Table 4.28: Beam and column condition of the College building (Model with bracing) due to Tsunami Load

Height of Tsunami	Inadequate Beams	Beam Vulnerability Condition	Inadequate Columns	Maximum Required Reinforcement in Column
1m	–	–	–	Ok
2m	–	–	C8-12,22, 23,25-28,32	C22= 5.33%
3m	B31, B33	Torsion	C8-12,22, 23,25-28,32	C22,26-28= 5.67%
Time History loading for 1.18m tsunami	–	–	–	Ok

Vulnerability of the structures were tested for dead load, live load and wind also. The results are given in the following table.

Table 4.29: Failure of Structures due to Deal Load, Live Load and Wind Load

Structure Type	Member	Vulnerable members	
		DL & LL	DL, LL & Wind
European Union	Beam	B1,3, 4,6-9, 12-14, 16-19,35-37	B1-3, 4,6-14, 16-20,35-37,
	Column	Ok	Ok
JICA	Beam	Ok	Ok
	Column	Ok	Ok
LGED	Beam	Ok	B1,3,6-12,18,19,21-24
	Column	C11-13,19-21	C1-7,8-147,16-22
Saudi Arabia	Beam	Ok	Ok
	Column	Ok	Ok
Grameen Bank	Beam	B19,20,23,24	B19,20,23,24
	Column	Ok	Ok
CDSP-II	Beam	B1,5,18-21,23	Ok
	Column	B1,5,7,10,11,13-21,24,26-29	C6,13,18,19
LGED-II	Beam	Ok	B24,29
	Column	Ok	C4-6,8-16,20
German	Beam	Ok	B16,24
	Column	Ok	Ok
BDRCS	Beam	B48-53, 46,54,56	B48-53, 46,54,56
	Column	Ok	Ok
Type-D	Beam	B7,11,25	B7,11,25
	Column	All columns	All columns
Type-E	Beam	B1,4,7,10	B1,4,7,10
	Column	C2,5,11,14,18-20	C2,5,11,14
Type-J	Beam	Ok	Ok
	Column	Ok	Ok
PWD	Beam	Ok	Ok
	Column	Ok	Ok
College Building	Beam	Ok	B31-33,54-57
	Column	Ok	C13,16,17,19,21,30

The results found in this study depend on a lot of assumptions. All the data required for the analysis were not available. For this reason some of the data were needed to be assumed for the analysis.

CHAPTER 5

CONCLUSIONS AND RECOMMENDATIONS

5.1 General

Different kinds of cyclone shelters exist in Bangladesh. The structural property of these structures varies a lot from one to another. For this reason they behave differently in same loading condition. The outcome of the study is presented here briefly. A discussion of the limitations of the study is followed subsequently. Then provision for further study is discussed.

5.2 Study output

The objective of the present study is to determine vulnerability of the existing cyclone shelters to tsunami. For this purpose three dimensional finite elements models of fourteen types of cyclone shelters are created. Two models are created for each type of shelters. One type is created considering the bare frame action of the structure. Only the bare frame stiffness is considered for this case. The other model is created with bracing considering the infill wall action. Then with the available information, the structures have been analyzed for tsunami. Both static (ESMA) and dynamic analyses are performed and their results are compared. Since these structures are typical examples, in reality they are situated at different locations and at different conditions. The hazard situation varies from place to place. For this reason each type of structure has been subjected to different loading conditions at different submergence depths. The purpose of applying different loading conditions is to identify the limit of loading, up to which a particular structure can sustain. Knowing the limits, the safety condition of each individual shelter can be identified.

Based on the information available, engineering judgments have been made in the analysis of these structures. As no information is available regarding the grade of steel reinforcement, it is assumed that 40-grade steel has been used. While determining adequacy of column sections, it is assumed that they contain 3% longitudinal reinforcement. However, if the exact steel ratio is known, a more accurate estimation of adequacy is possible. Concrete strength of the existing structures was assumed 3000 psi as actual concrete strength was not available. No study to check the adequacy of foundation has been carried out since these structures may have different types of foundation.

For tsunami, due to its excessive flood velocity, few structures can sustain a tsunami height of more than 6 ft. Thus, cyclone shelters in the regions susceptible to the hazard of tsunami may need to be strengthened.

In order to examine the dynamic effect, Time History Analysis is performed. But the data of the analysis is found from a test result.

The analysis results of the structures are briefly stated below:

Cyclone shelter funded by the European Union: The columns of this structure fail for 3m tsunami loading but the beams fail in torsion even for 1m tsunami loading.

JICA type cyclone shelter: The structure is found safe up to 3m tsunami loading in analysis.

LGED type cyclone shelter: This structure fails due to column failure when 2m tsunami loading is applied.

Cyclone shelter funded by Saudi Arabia: This structure fails due to column failure when 3m tsunami loading is applied.

Cyclone shelter built by Grameen Bank: The columns of this structure fail for 3m tsunami loading but the beams fail in torsion even for 1m tsunami loading.

CDSP-2 type cyclone shelter: The columns of this structure fail for 2m tsunami loading but the beams fail in torsion even for 1m tsunami loading.

LGED-2 type cyclone shelter: This structure fails due to column failure when 3m tsunami loading is applied.

Cyclone shelter funded by German: This structure fails due to column failure when 3m tsunami loading is applied.

BDRCS cyclone shelter: One column of this structure fails for 3m tsunami loading but the beams fail in torsion even for 1m tsunami loading.

Type-D cyclone shelter: This structure fails due to column and beam failure even for 1m tsunami loading.

Type-E cyclone shelter: This structure fails due to column and beam failure even for 1m tsunami loading.

Type-J cyclone shelter: This structure fails due to column failure when 3m tsunami loading is applied.

PWD Type cyclone shelter: This structure fails due to column failure when 3m tsunami loading is applied.

College building which can be used as a cyclone shelter: This structure fails due to column and beam failure when 2m tsunami loading is applied.

5.3 Recommendations for further study

Due to certain limitations this study is not conclusive. This is only the beginning. Here is an attempt to give direction to future studies.

- The infill walls can be modeled with some other methods to compare with each other.
- Determination of adequacy of foundation is very important.
- Study should be performed for earthquake, wind and storm surge along with tsunami for coastal structures. Response Spectrum analysis should be performed also.
- Models of standard cyclone shelters can be tested physically using wind tunnel, wave basin and shaking table.

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Appendix-A

Sample Calculation for Tsunami Load :

Tsunami Load is calculated from Hydrodynamic Load and Impact Load.

1. Hydrodynamic Force, $F_d = (\rho C_d A u_p^2)/2$

Where C_d is the drag coefficient, ρ is the water density, A is the projected area and u_p is the design flood velocity.

Flow depth, ds(m) =	3	m	
=	9.84	ft	
ρ =	1.99	$ld*sec^2/ft^4$	
Cd =	2		
Column width, d =	0.984	ft	
A =	9.686	sft	$v = 2\sqrt{(g*ds)}$
g =	32.2	ft/sec^2	
v =	35.600	ft/sec	
Fd =	24427.927	ld	
=	24.428	kip	
Wd =	Fd/ds		
Drag force, Wd =	2.483	kip/ft	
=	2482.513	lb/ft	

2. Impact Force, $F_I = (WV)/(g\Delta t)$

Where W is the weight of debris, V is the design flood velocity, g is the gravitational constant and Δt = 0.1 sec Code assumed impact duration in seconds

V =	35.60	ft/sec
Δt =	0.1	sec
W =	1000	lb
Ft =	11056.04	lb
Ft =	11.056	kip

Sample Calculation of Strut Width, "a"

$\lambda 1H = H[(Emtsin2\Theta)/(4EcIcolhw)]^{1/4}$

$a = 0.175D(\lambda 1H)^{-0.4}$

a = Equivalent Strut width
t = Thickness of the masonry infill
Em = Modulus of Elasticity of the masonry unit
Ec = Modulus of Elasticity of concrete
hw = Clear ht of Column member
Icol = Moment of Inertia of the Column
Θ = Angle Produced by the strut with the horizontal
D = Diagonal

Here,
t = 5 in
Em = 1200 psi
Ec = 3586.6 psi

hw = 102.36 in

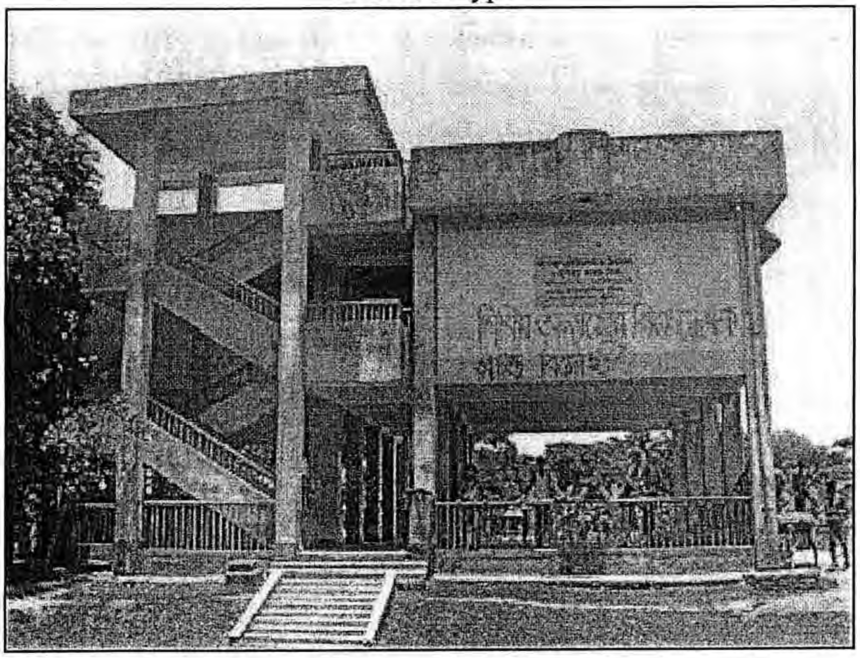
Icol = 1621.68 in^4
Θ = 36.87 degree
D = 196.85 inch

$I = b*h^3/12$
$= 1621.68\ in^4$
Where,
b= 11.811 in
h = 11.811 in

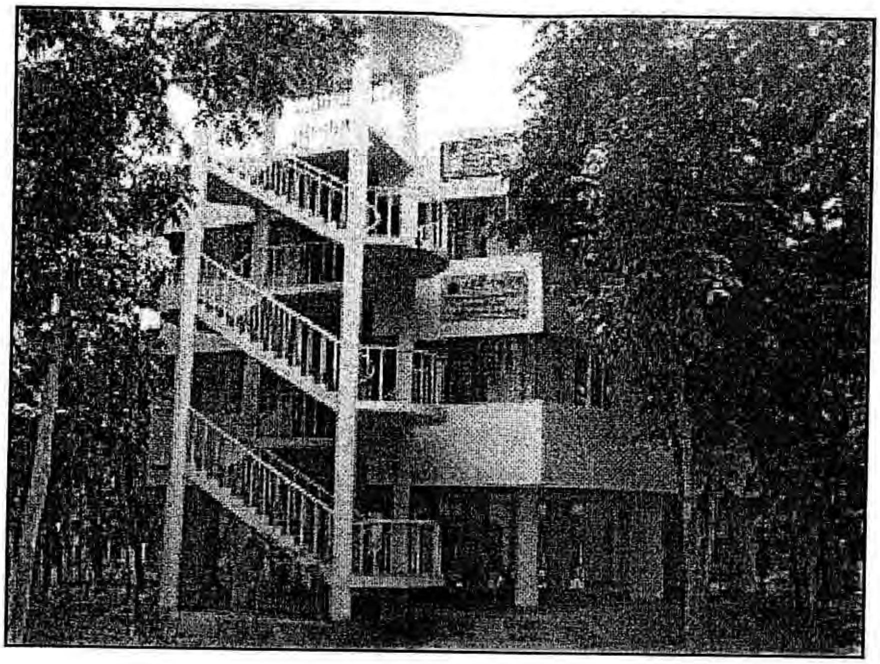
λ1H = 4.65 in
a = 18.62 in

Pictures of Cyclone Shelters

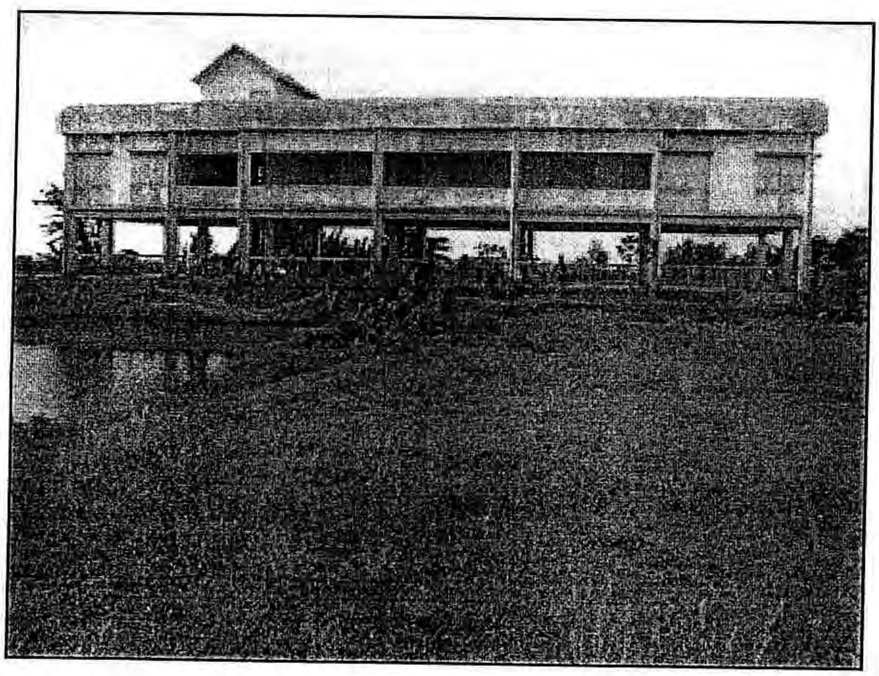
BDRCS Type



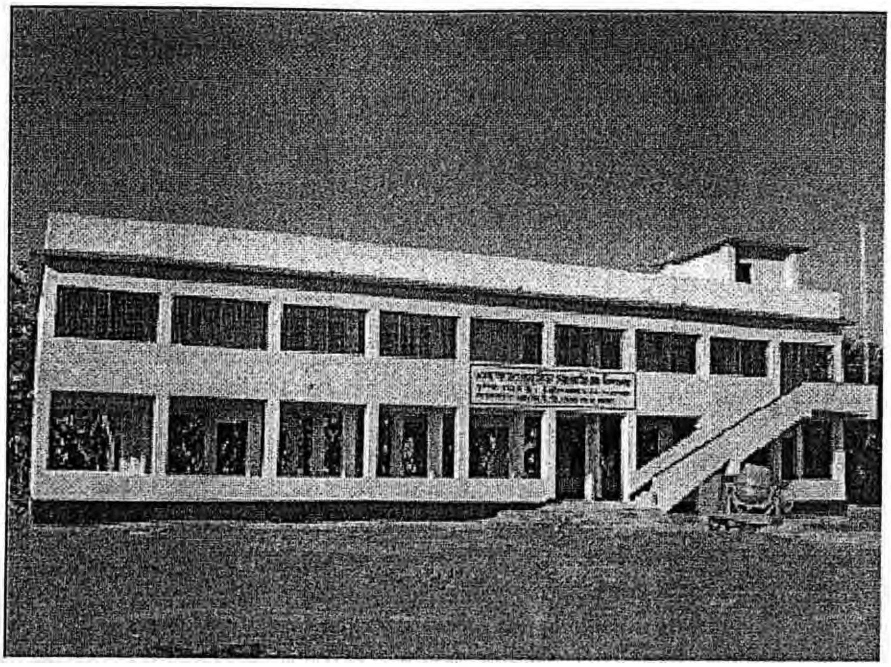
European Union Type



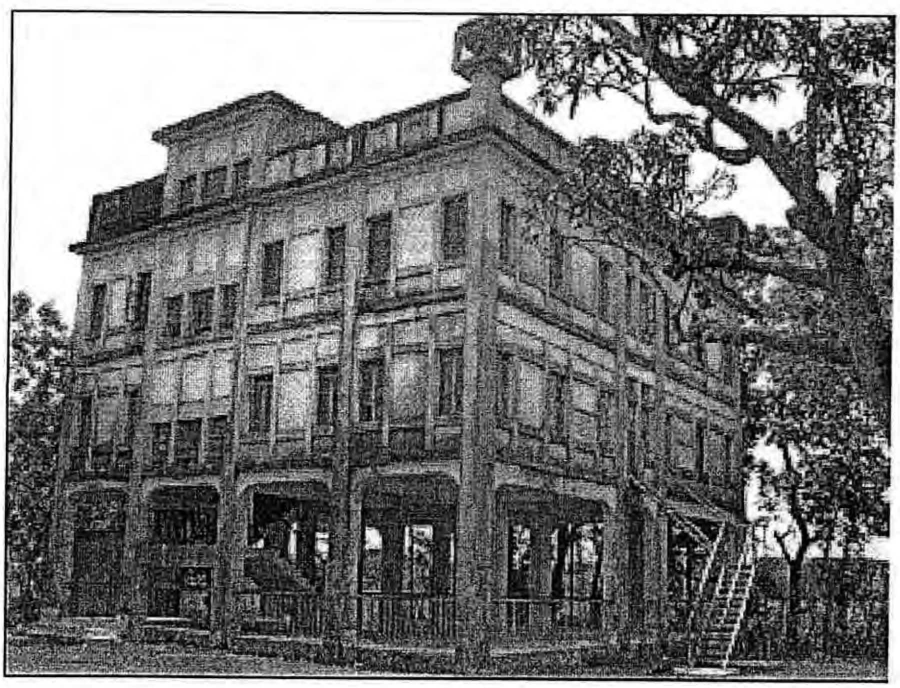
Grameen Bank Type



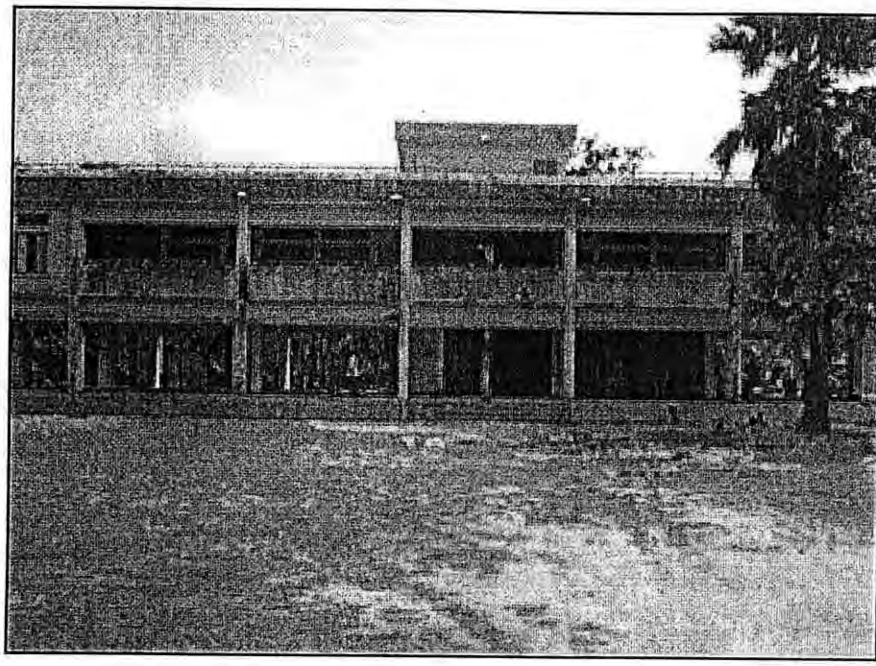
LGED Type



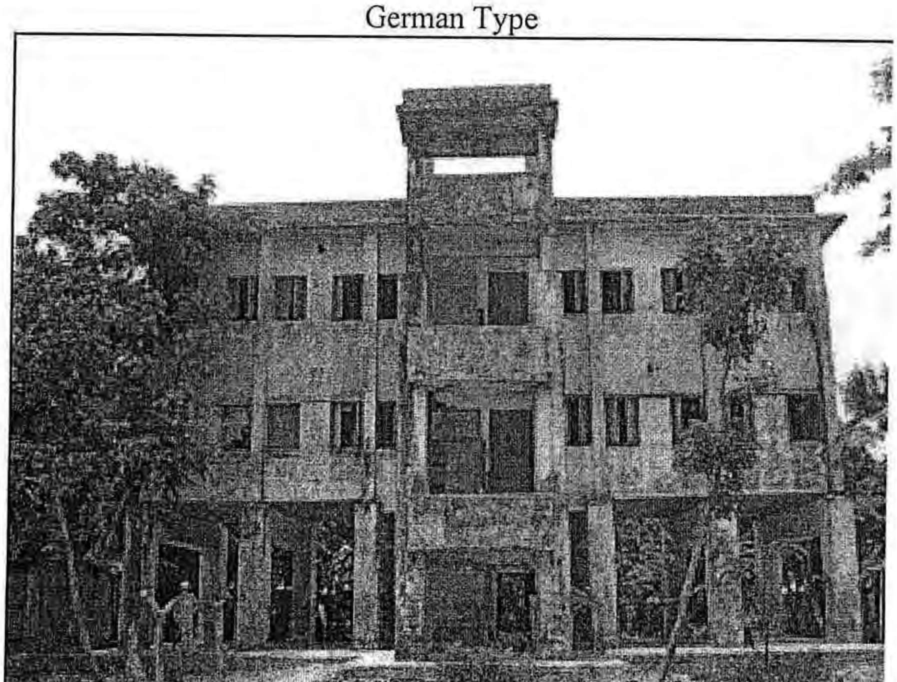
LGED-II Type



Saudi Type



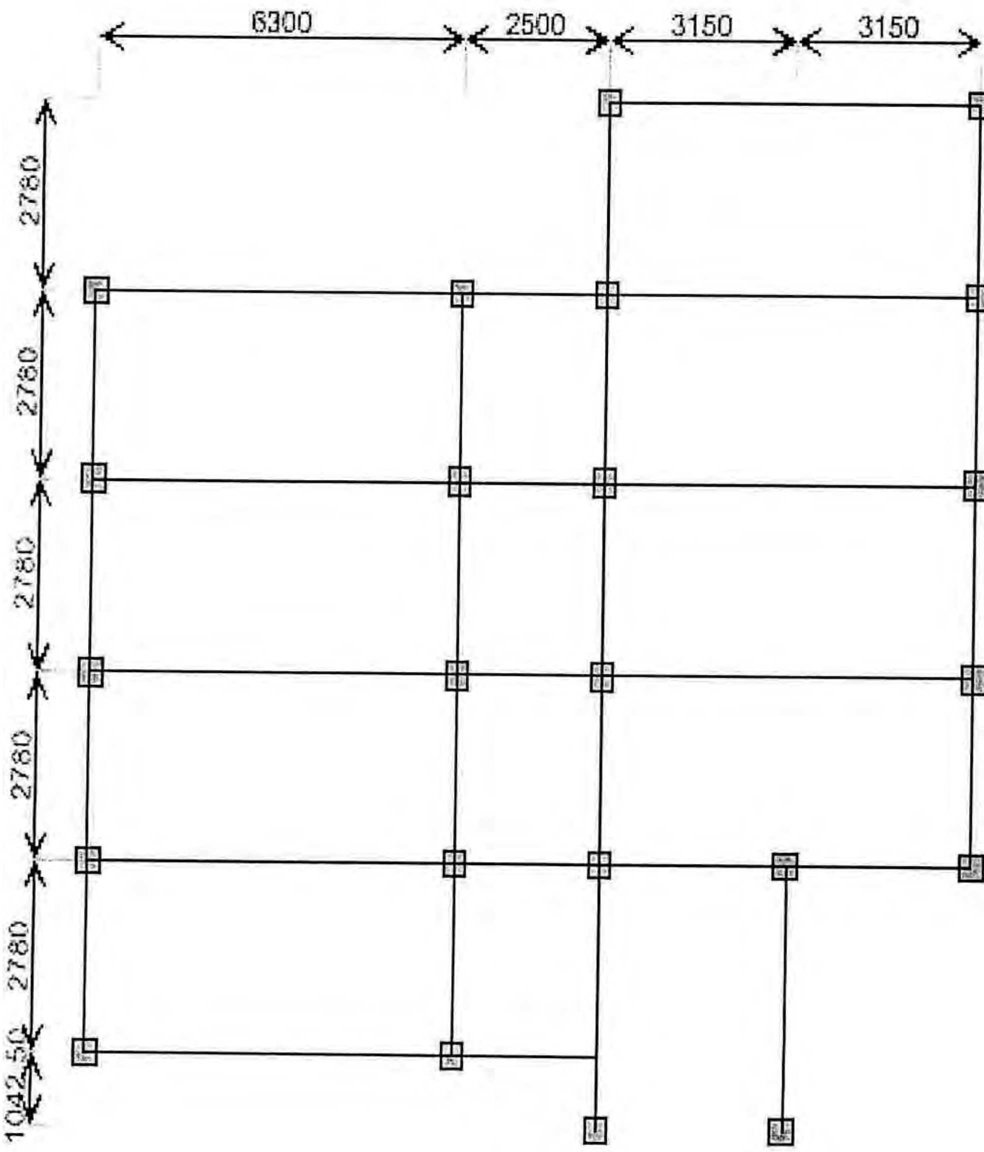
German Type



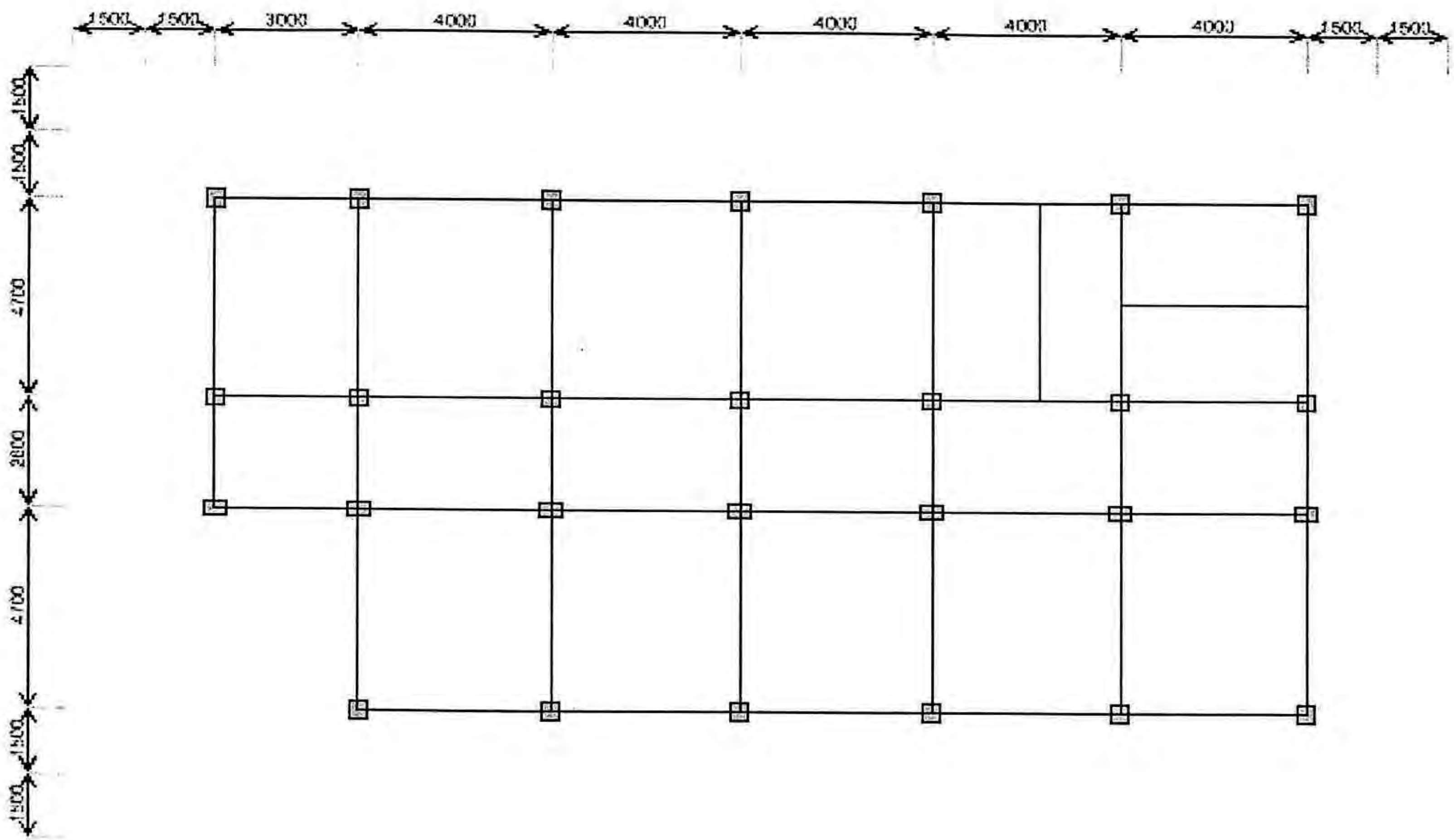
PWD Type

Appendix-D

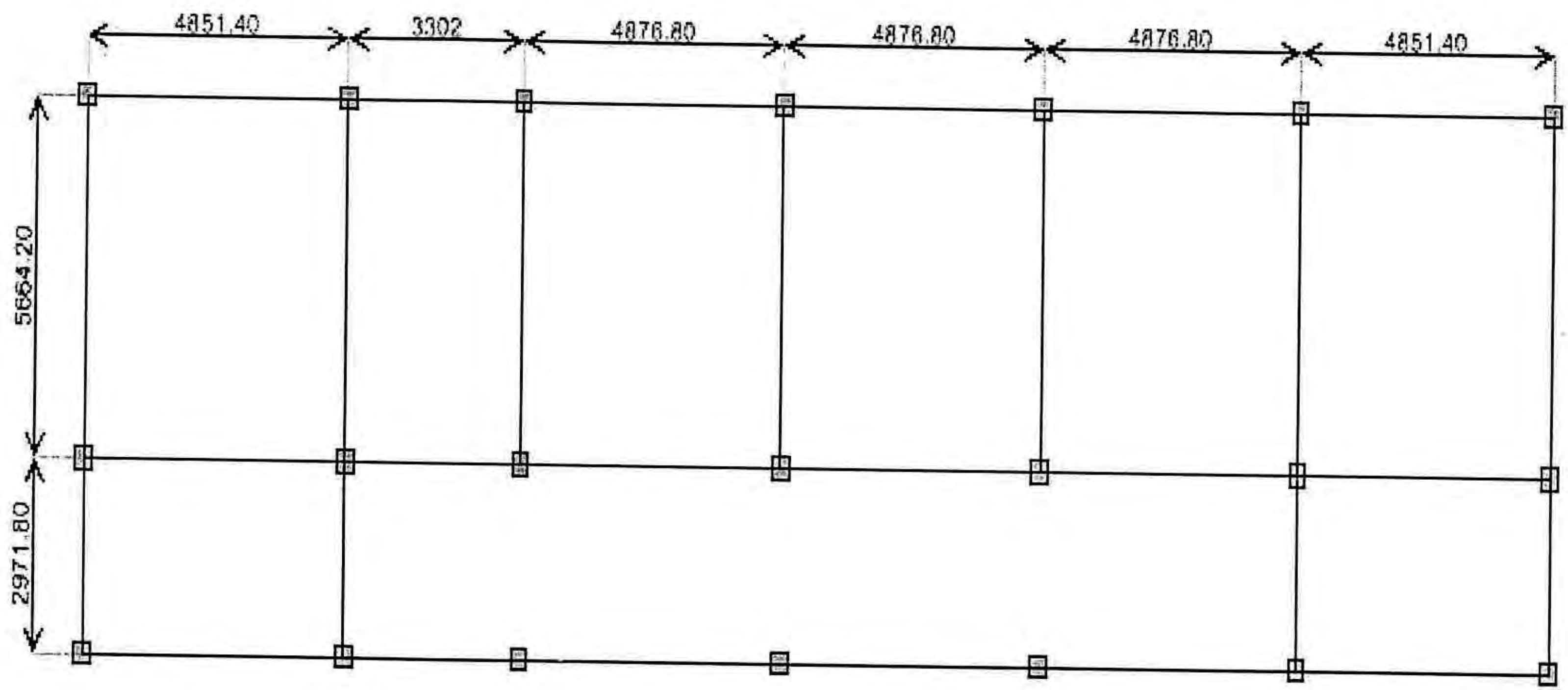
Plans of the Cyclone Shelters



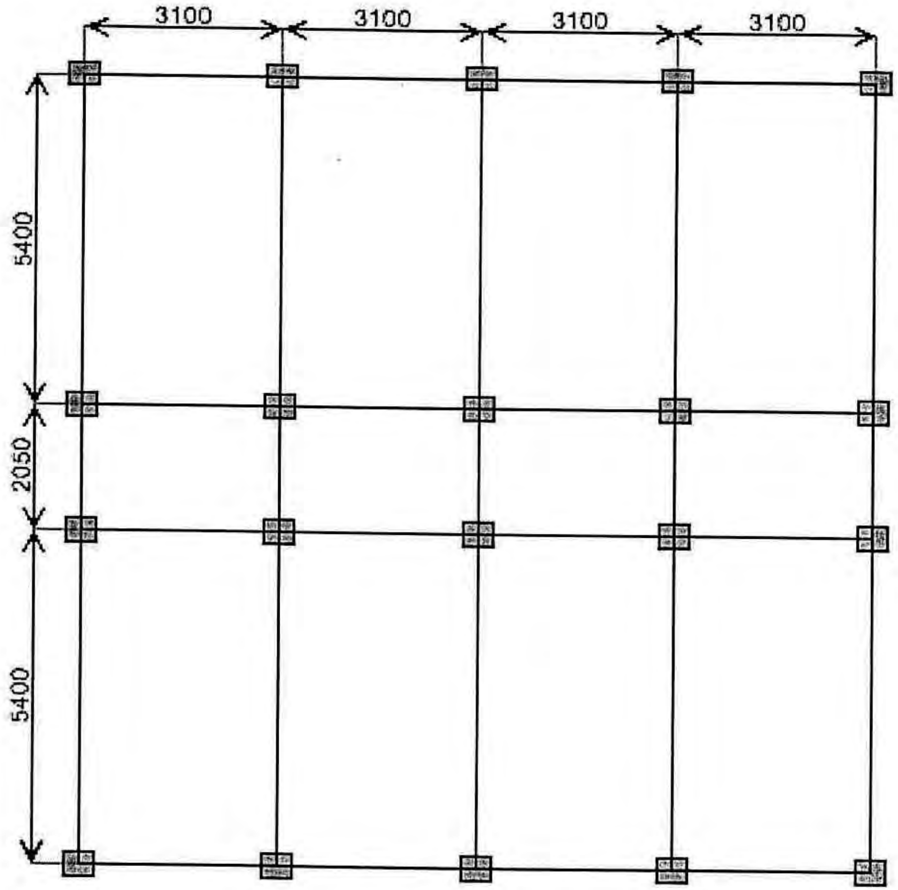
1. Figure: Cyclone Shelter funded by European Union (mm)



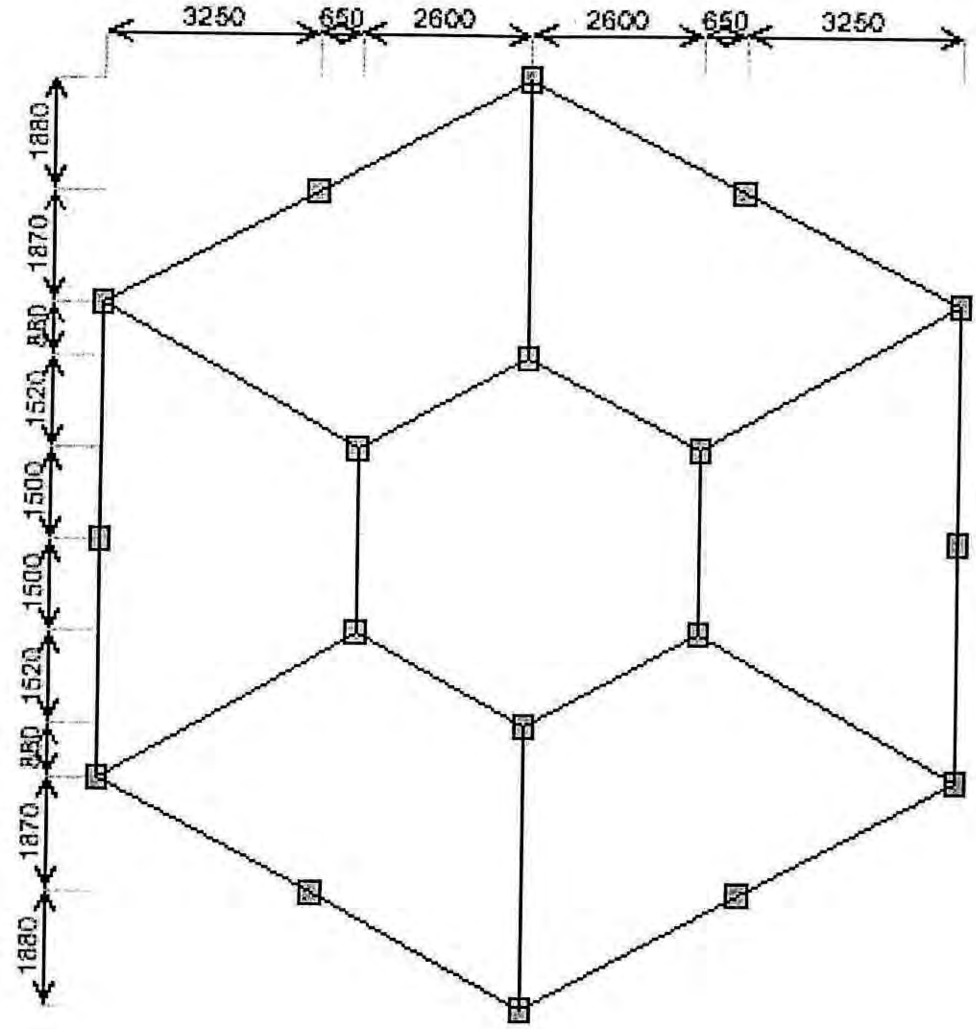
2. Figure: JICA Type Cyclone Shelter (mm)



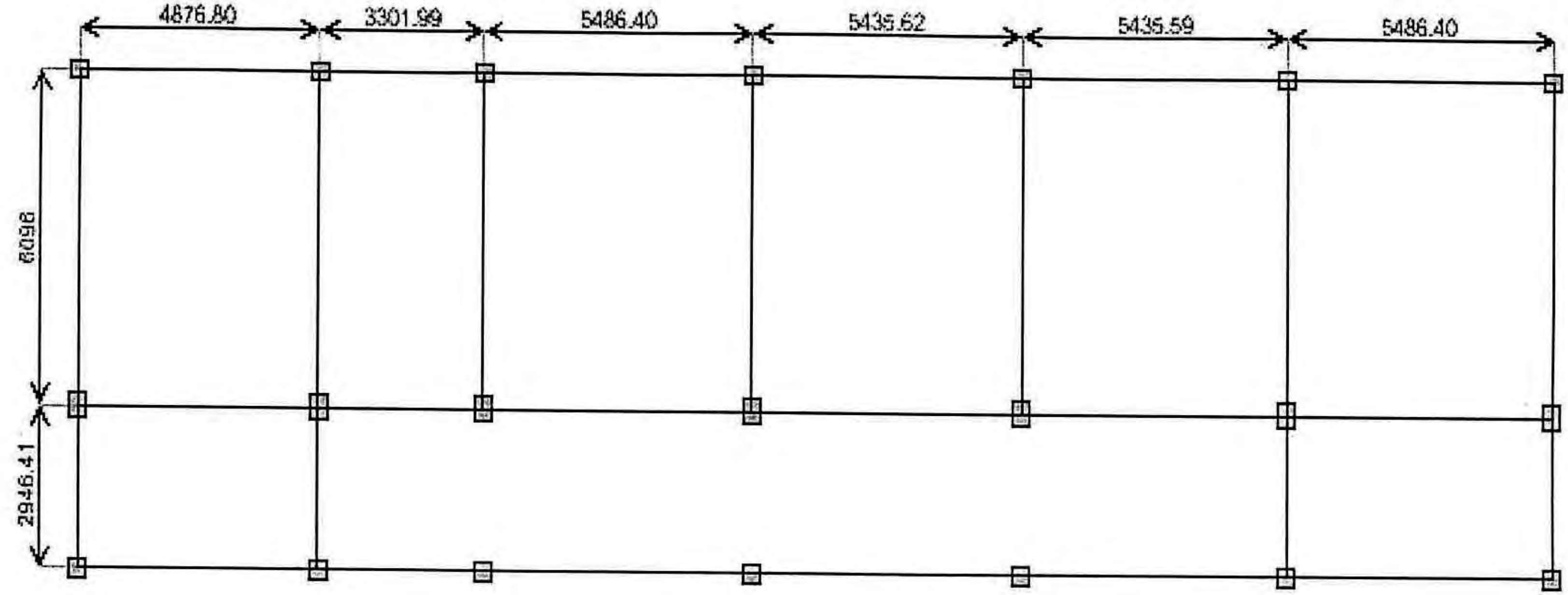
3. Figure: LGED Type Cyclone Shelter (mm)



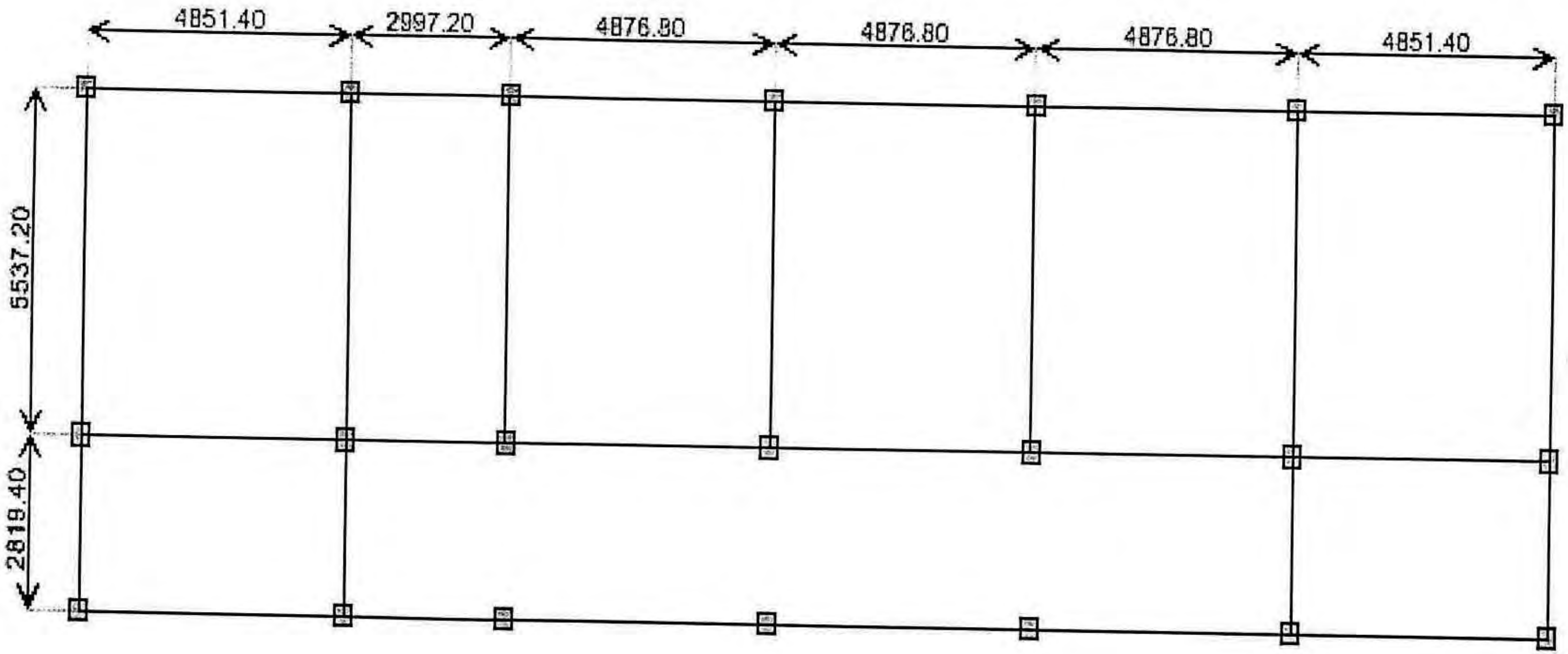
4. Figure: Cyclone Shelter funded by Saudi Arabia (mm)



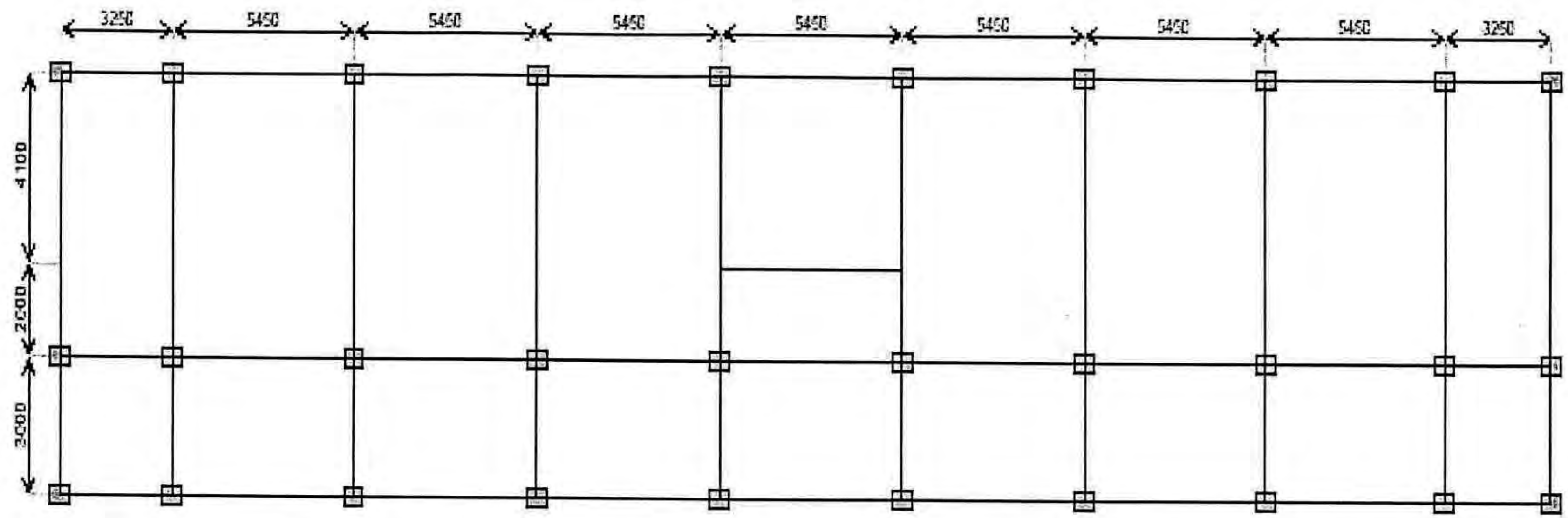
5. Figure: Grameen Bank Cyclone Shelter (mm)



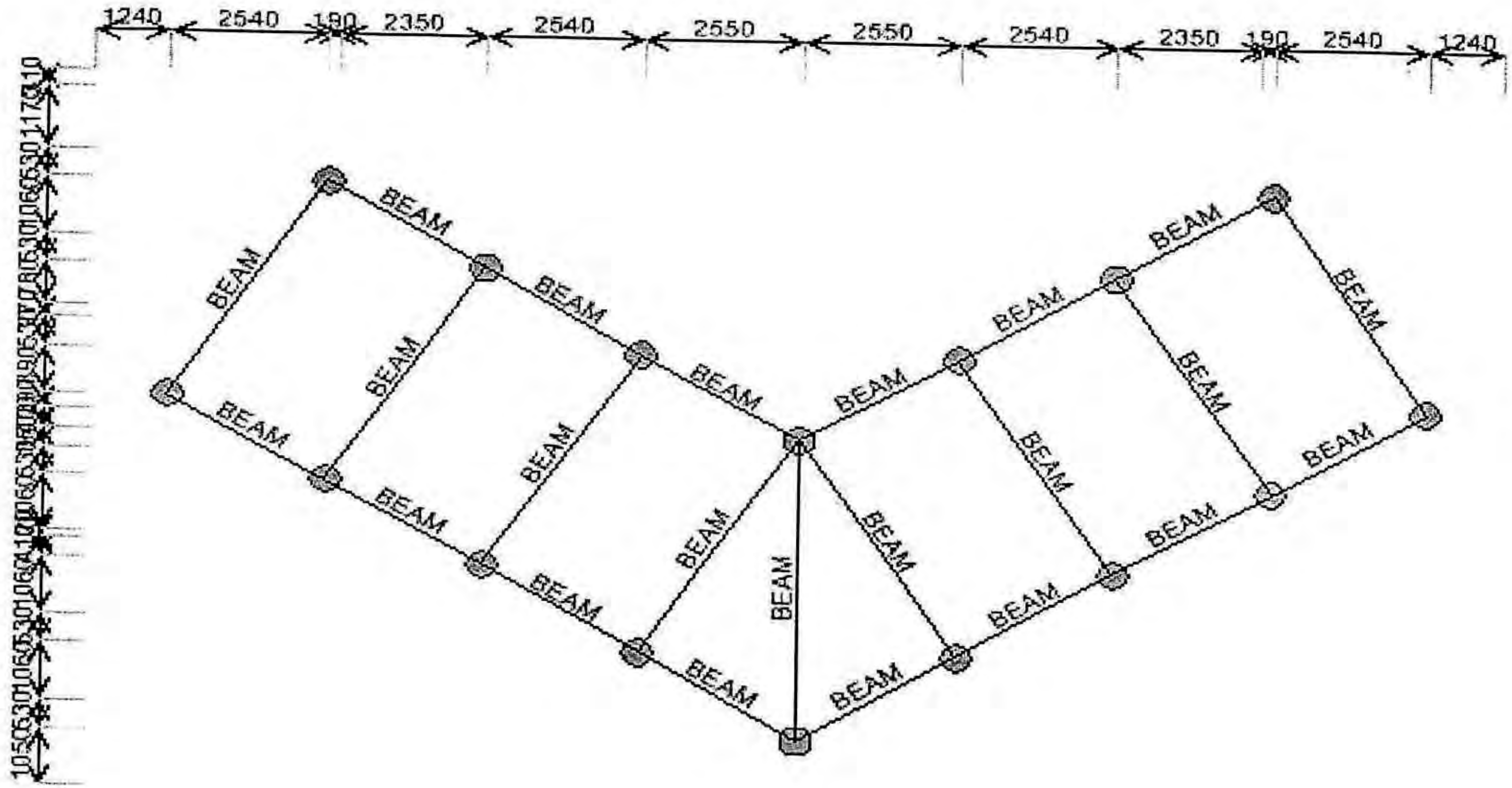
6. Figure: CDSP-2 Type Cyclone Shelter (mm)



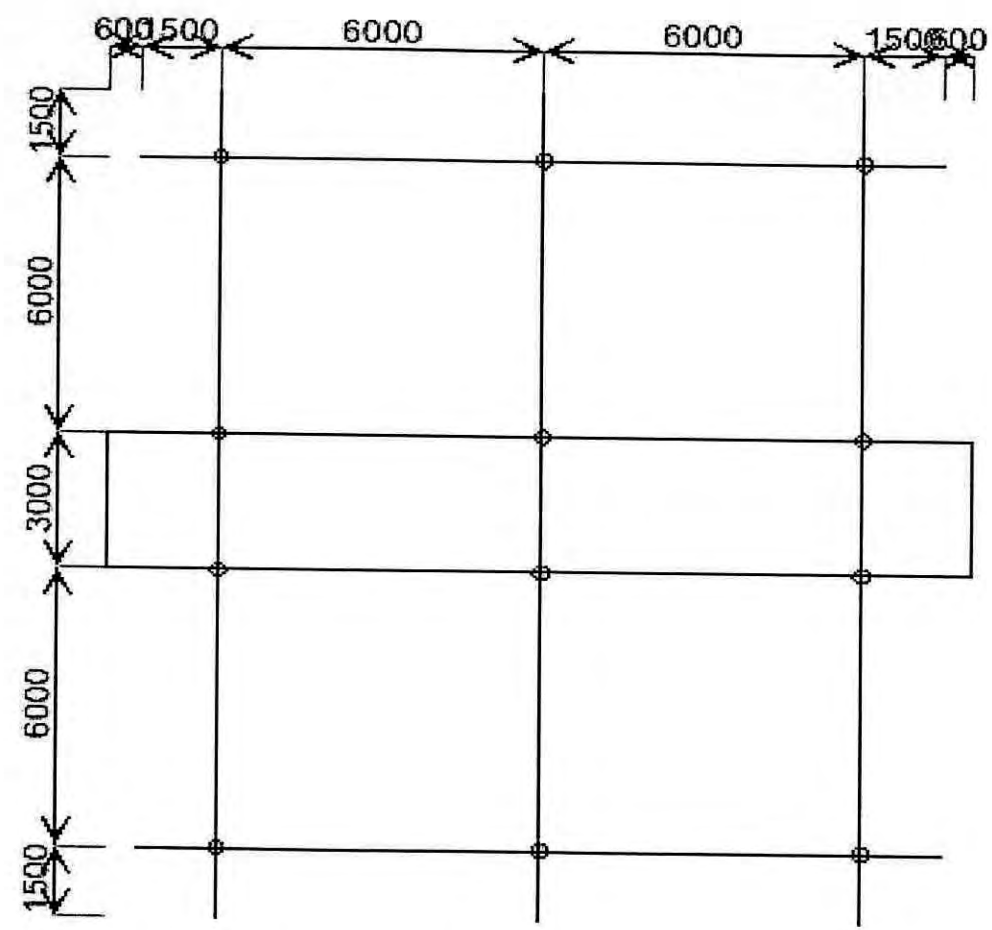
7. Figure: LGED-2 Type Cyclone Shelter (mm)



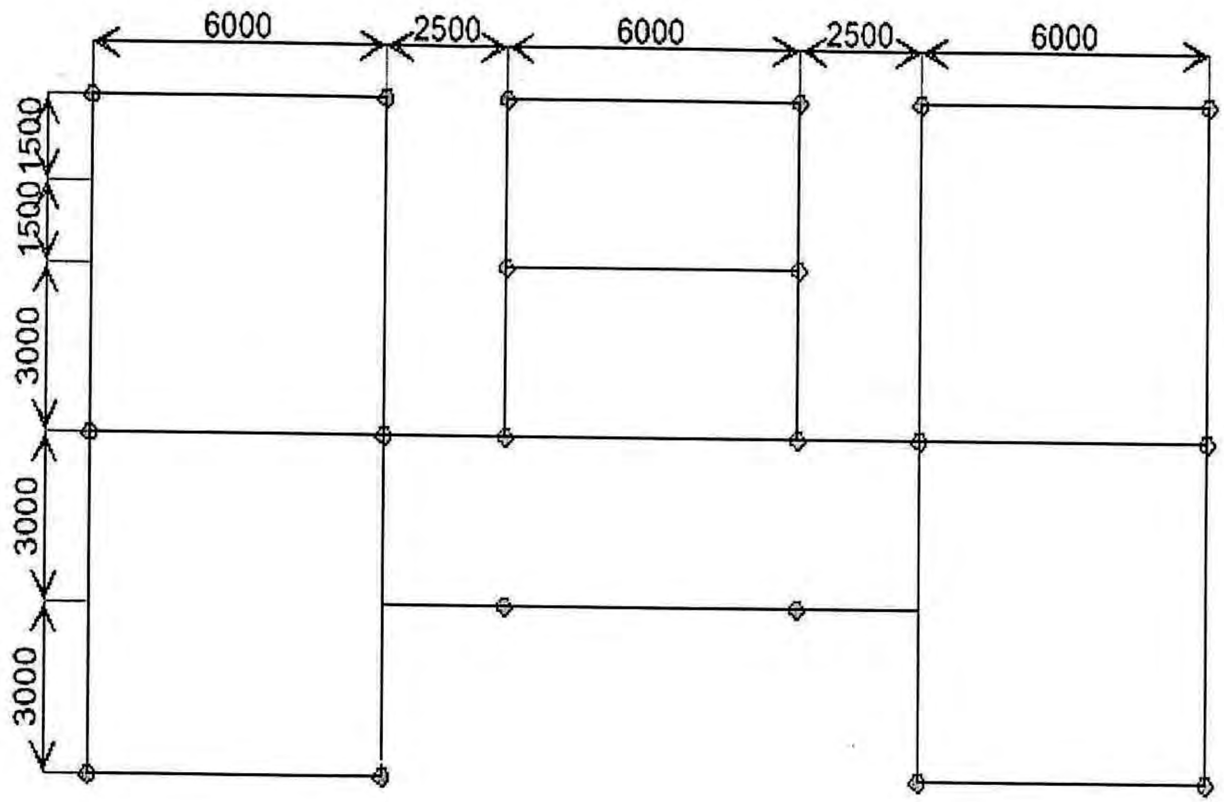
8. Figure: Cyclone Shelter funded by Germany (mm)



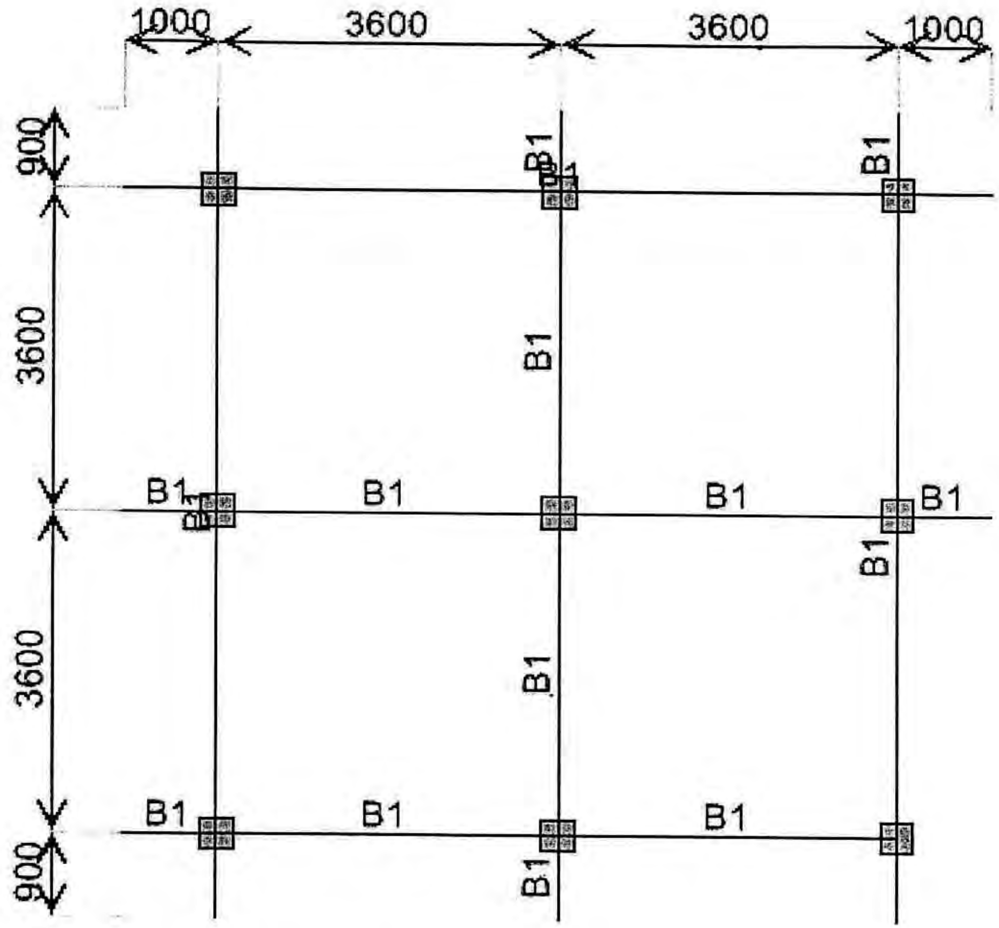
9. Figure: BDRCS Type Cyclone Shelter (mm)



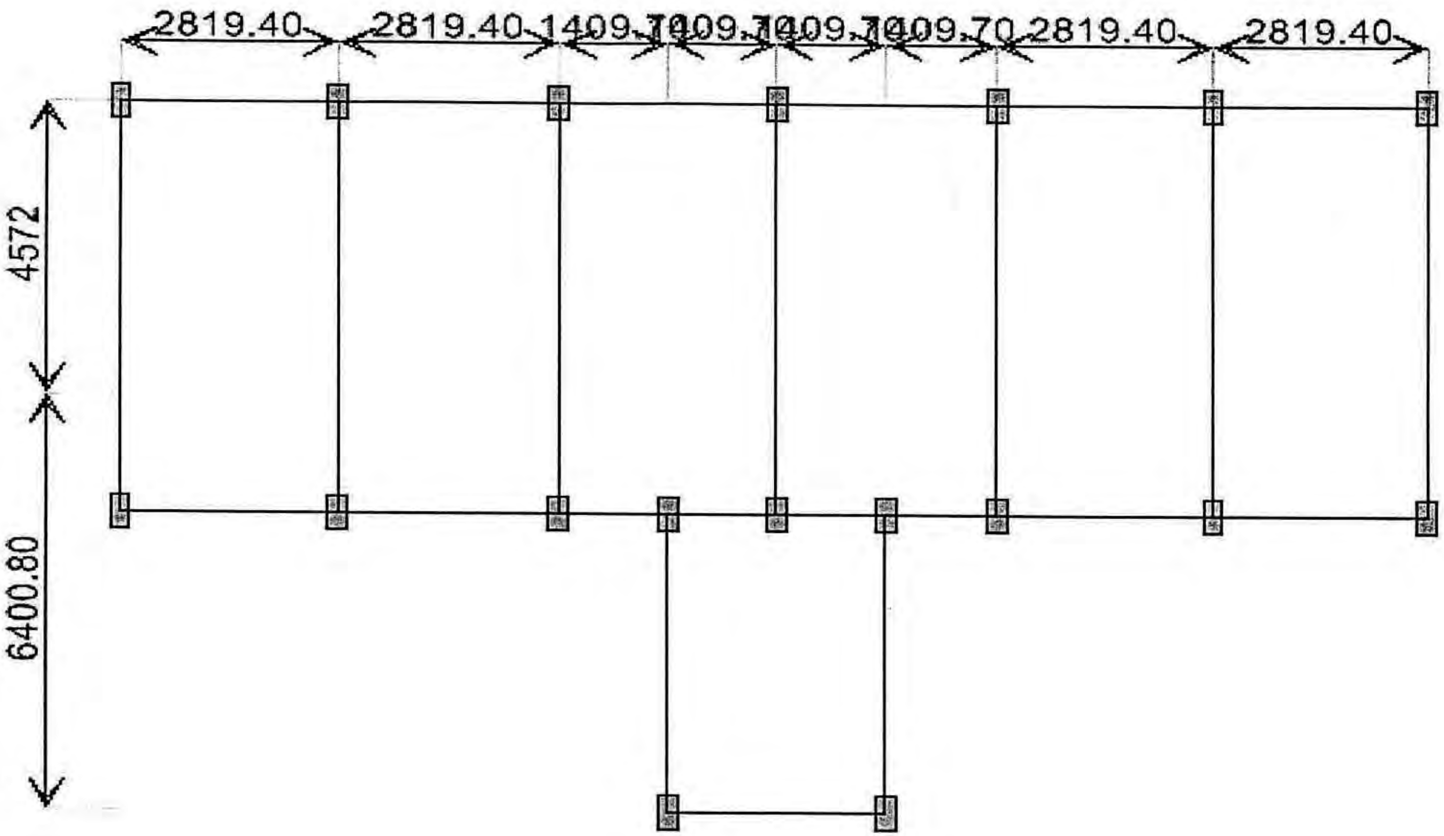
10. Figure: Type-D Cyclone Shelter (mm)



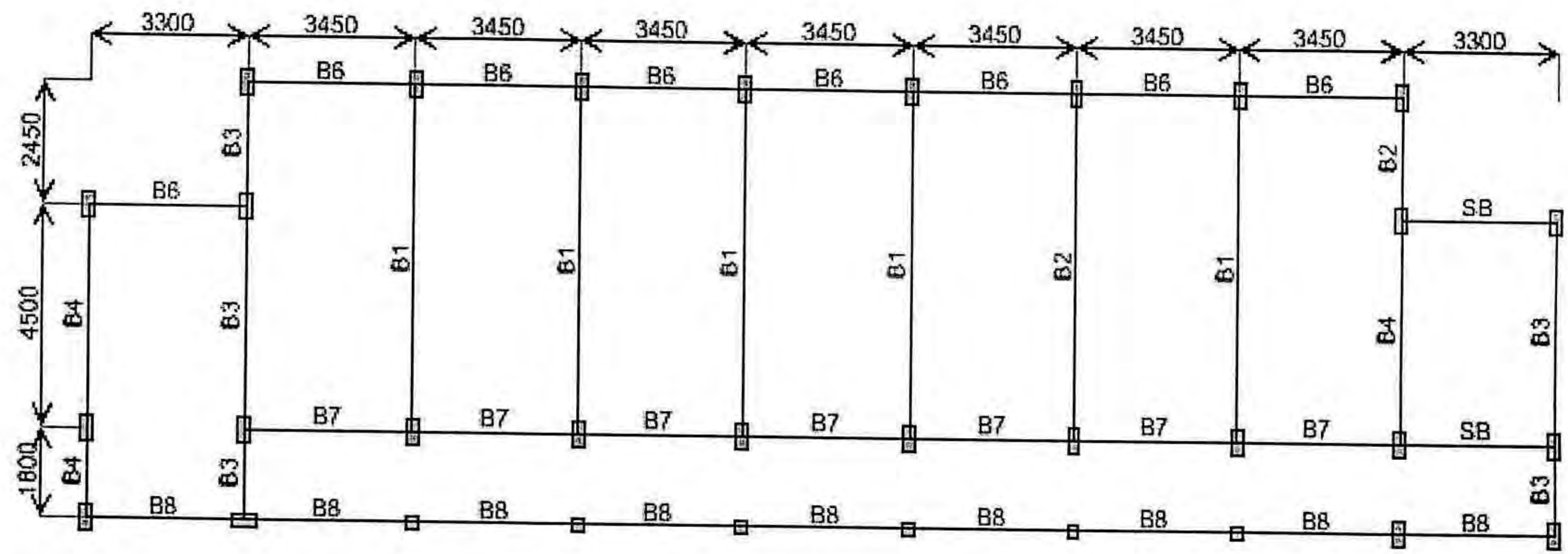
11. Figure: Type-E Cyclone Shelter (mm)



12. Figure: Type-J Cyclone Shelter (mm)



13. Figure: PWD Type Cyclone Shelter (mm)



14. Figure: College Building (mm)

Appendix-E

Beam and Column Dimension

Structure Type	Members		Member Size
European Union	Beam	All Beams (Including Grade Beams)	12 in x15 in
	Column	All Columns	15in x 15 in
JICA	Beam	All Beams (Including Grade Beams)	12 in x 18 in
	Column	C1-6,14-20	16 in x 16 in
		C7-13, 21-27	12 in x 18 in
LGED	Beam	Beams	12 in x 20 in
		Grade Beam	10 in x15 in
	Column	C8-14	12 in x15 in
		Rest of the columns	15in x 15 in
Saudi Arabia	Beam	All Beams (Including Grade Beams)	10 in x 20 in
	Column	All Columns	15in x 18 in
Grameen Bank	Beam	B13-18	10 in x 16 in
		Rest of the Beams	12 in x 18 in
		Grade Beam	10 in x15 in
	Column	All Columns	12in x 15 in
CDSP-II	Beam	All Beams (Including Grade Beams)	10 in x 18 in
	Column	C1-C7	10 in x15 in
		Rest of the columns	10 in x 10 in
LGED-II	Beam	B7-12	12 in x15 in
		Rest of the Beams	12 in x 20 in
		Grade Beam	10 in x15 in
	Column	C8-14	12 in x15 in
		Rest of the columns	12 in x 12 in
German	Beam	Beams	10 in x 24 in
		Grade Beam	12in x 20 in
	Column	All Columns	12in x 20 in
BDRCS	Beam	All Beams (Including Grade Beams)	10 in x 15 in
	Column	All Columns	20 in dia
Type-D	Beam	All Beams (Including Grade Beams)	10 in x 12 in
	Column	All Columns	12 in dia
Type-E	Beam	All Beams (Including Grade Beams)	10 in x 16in
	Column	All Columns	12 in dia
Type-J	Beam	All Beams	10 in x 16in

Structure Type	Members		Member Size
		(Including Grade Beams)	
	Column	All Columns	14 in x 14 in
PWD	Beam	B1,2,10-14	10 in x 30 in
		Rest of the Beams	10 in x 18 in
	Column	All Columns	12 in x 20 in
College Building	Beam	B2-4,8,17,53-58,28-30	10 in x 18 in
		Rest of the Beams	10 in x 15 in
		Grade Beam	10 in x 12 in
	Column	C1-3, 7-21,24,29-32	10 in x 20 in
		C22,23,25-28	10 in x 10 in