BHIMPHEDI AWASUKA

भीमफेदी आवासुका

*Program developped by:*

**AMICS DEL NEPAL * BASE A * CCD-UPC University**

SERVAMUS

Rotary Club of Kantipur

RI DISTRICT 3292

# STRUCTURAL REPORT

## CONFINED HOLLOW CONCRETE BLOCK MODEL HOUSE


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## FOREWORD

It is our immense pleasure to present this Confined Hollow Concrete Block Model House to the Government of Nepal so that it can be used nationwide. After our small-scale work in Bhimphedi for twenty years, collaborating in community improvement projects, our NGO Amics del Nepal is now willing to share all these years' knowledge and experience with the national authorities so that many more people in Nepal can benefit from it in a larger scale.

Shortly after the 2015 earthquake, our main objective was to provide technical support to the people, in close collaboration with the Government of Nepal and following the Prototypes and Guidelines presented in the Design Catalogues. However, due to our focus in a small working area, we were able to work closely with the villagers and better understand their housing needs regarding cultural features, rooms' function, suitable budget, local materials, type of available manpower and local esthetics.

Our Confined Hollow Concrete Block House follows a similar technique as the one presented in the Design Catalogue Volume II, but with slight differences to make it more suitable to the villager's capabilities in terms of low cost, materials' availability and technique simplicity. After intense study and research of rural houses' typology, the final conclusion for the ideal layout was to produce a Minimum House-Unit consisting of Two Rooms with Verandah. This unit guarantees the minimum habitat needs for a family and is financially affordable. Later on, the family is able to extend the house with another Two-Room House, in order to obtain a Four-Room House (with or without corridor). These two possibilities of growth variations have been carefully calculated to be presented herewith, so that the people can chose to build them all at once or in two different stages.

We again express our gladness and contentment to be able to scale up our work in Bhimphedi to the rest of the country. We deeply hope that our models be useful to many people.

**Ar. Monica Sans Duran**
*Awasuka* Program Director *(Aawaas Sudhar Karyakram)*
Amics del Nepal NGO

**AKNOWLEDGEMENTS**

Awasuka Team would like to sincerely thank all the people and entities involved in this program for their strong commitment without which this program would not be possible.

All the people in Amics del Nepal (Barcelona), Base-A (Barcelona), CCD-UPC University (Barcelona), Agragaami Krishak Krishi Sahakaari (Bhimphed) and Rotary Club Kantipur (Kathmandu) have been extremely helpful. However, our special thanks go to the following persons who have been crutial for the program development:

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**Awasuka Program Team**

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## GLOSSARY

1. ***Base Dimension:*** Base dimension of the building along a direction is the dimension at its base along that direction.
2. ***Base Shear:*** Base Shear is an estimate of the maximum expected lateral force that will occur due to seismic ground motion at the base of a structure.
3. ***Building Height:*** It is the difference in levels between its base and its highest level.
4. ***Centre of Mass:*** The point through which the resultant of the masses of a system acts. This point corresponds to the centre of gravity of masses of system.
5. ***Centre of Stiffness:*** The point through which the resultant of the restoring forces of a system acts.
6. ***Confined Masonry:*** Confined Masonry construction consists of masonry walls (made of either clay brick or concrete block units) and horizontal and vertical reinforced concrete confining members built on all foursides of a masonry wall panel.
7. ***Confining Elements:*** Confining elements (bond beams and tie columns) provide restraint to masonry walls and protect them from complete disintegration even in major earthquakes. These elements resist gravity loads and have important role in ensuring vertical stability of a building in an earthquake.
8. ***Design Lateral force:*** It is the horizontal seismic force that shall be used to design a structure.
9. ***Design Eccentricity:*** It is the value of eccentricity to be used at floor in torsion calculations for design.
10. ***Design Horizontal Acceleration Coefficient:*** It is a horizontal acceleration coefficient that shall be used for design of structures.
11. ***Eccentricity:*** It is the distance between centre of mass and centre of rigidity of floor.
12. ***Masonry:*** An assemblage of masonry units properly bonded together with mortar.
13. ***Masonry Units:*** Individual units which are bonded together with the help of mortar to form a masonry element, such as wall, column, pier and buttress.

14. ***Masonry Walls:*** Masonry walls transmit the gravity load from the slab(s) above down to the foundation. The walls act as bracing panels, which resist horizontal earthquake forces. The walls must be confined by concrete tie-beams and tie-columns to ensure satisfactory earthquake performance.
15. ***Response Reduction Factor:*** It is the factor by which the actual base shear force, that would be generated if the structure were to remain elastic during its response to the Design Basis Earthquake (DBE) shaking, shall be reduced to obtain the design lateral force.
16. ***Seismic Weight:*** It is the total dead load plus appropriate amounts of specified imposed load.
17. ***Slenderness Ratio:*** Ratio of effective height or effective length to effective thickness of a masonry element.
18. ***Structural Response Factor:*** It is a factor denoting the acceleration response spectrum of the structure subjected to earthquake ground vibrations, and depends on natural period of vibration and damping of the structure.
19. ***Wall Density:*** Wall density can be defined as the total cross-sectional area of all walls in one direction divided by the total floor area.
20. ***Zone Factor:*** It is a factor to obtain the design spectrum depending on the perceived maximum seismic risk characterized by Maximum Considered Earthquake (MCE) in the zone in which the structure is located.

## ABREVATION

| | |
|---|---|
| $A_h$ | Design horizontal acceleration coefficient |
| $A_p$ | Plan area of floor |
| $A_{st}$ | Area of reinforcement |
| $A_{sv}$ | Area of stirrups |
| $A_w$ | Cross-sectional area of wall |
| b | Column dimension |
| B | Width of beam |
| | Floor plan dimension perpendicular to the direction of force |
| $b_i$ | Base |
| | dimension of the building at the plinth level along the |
| d | considered |
| D | direction of the lateral force Depth of beam |

| | |
|---|---|
| D' | Lateral dimension of column in the direction under consideration |
| $d_{eff}$ | Effective depth of beam |
| $S_{di}$ | Design eccentricity Design |
| $e_{dx}$ | eccentricity in X direction |
| $e_{dy}$ | Design eccentricity in Y direction |
| $E_m$ | Young's modulus of masonry |
| $e_{min}$ | Minimum eccentricity Static |
| $e_x$ | eccentricity in X direction Static |
| $e_y$ | eccentricity in Y direction |
| $f_b$ | Compressive strength of brick |
| $f_{ck}$ | Concrete grade |
| $f_g$ | Safety factor for gravity load |
| $f_s$ | Safety factor for seismic load |
| F | Seismic load per unit area of wall panel |
| $F_{ixi}$ | Force due to uniform lateral translation in X direction |
| $F_{lyi}$ | Force due to uniform lateral translation in Y direction |
| $f_m$ | Compressive strength of masonry |
| $f_{mo}$ | Compressive strength of mortar |
| $F_{txi}$ | Force due to torsion in X direction |
| $F_{tyi}$ | Force due to torsion in Y direction |
| $f_y$ | Steel grade |
| H | Height of building |
| $h_c$ | Height of Column |
| $h_i$ | Height of floor level measured from base |
| $h_0$ | Height of opening |
| $h_w$ | Height of wall |
| I | Importance factor |
| k | Number of storeys above the analysed story |
| $K^c$ | Stiffness of cantilever pier |
| $K_f$ | Stiffness of fixed pier |
| $k_s$ | Stress reduction factor |

| | |
|---|---|
| $K_t$ | Torsional stiffness |
| K' | Stiffness of wall panel |
| $K_x$ | Total stiffness in X direction |
| $I_{Cd}$ | Stiffness of individual wall panels in X direction |
| $K_y$ | Total stiffness in Y direction |
| $K_{yi}$ | Stiffness of individual wall panels in Y direction |
| L | Unsupported length of column |
| $L_b$ | Length of beam |
| $L_c$ | Length of column |
| $L_o$ | Length of opening |
| $L_w$ | Length of wall panel |
| $L_x$ | Length of wall in X direction |
| $l_x$ | Length of slab in X direction |
| $L_y$ | Length of wall in Y direction |
| ly | Length of slab in Y direction |
| M | Moment due to total lateral force acting on wall panel |
| $M_o$ | Overturning moment |
| $M_r$ | Resisting moment |
| $m_s$ | Mass of slab |
| $M_u$ | Ultimate bending moment due to seismic load |
| $M_u'$ | Moment in column |
| n | Number of longitudinal bars |
| $n_s$ | Number of stories in the building |
| P | Percentage of steel in column |
| $P_{comp}$ | Ultimate compressive strength of wall due to gravity load |
| $P_u$ | Total factored load acting on column |
| $P_i$ | Total lateral force acting on wall panel i |
| $P_{xi}$ | Total force acting on wall panel in X direction |
| $P_{yi}$ | Total force acting on wall panel in Y direction |
| $Q_i$ | Design lateral force at floor i |
| $Q_{ix}$ | Design lateral force in X direction |
| $Q_{iy}$ | Design lateral force in Y direction |

| | |
|---|---|
| $R$ | Response reduction factor |
| $S$ | Section modulus |
| $S_{c/g}$ | Average response acceleration coefficient |
| $S_R$ | Slenderness ratio |
| $s_v$ | Spacing of stirrups |
| $T$ | Design period of building |
| $t_c$ | Thickness of column |
| $T_L$ | Total gravity load acting on wall panel |
| $t_b$ | Thickness of beam |
| $t_o$ | Thickness of opening |
| $t_s$ | Thickness of slab |
| $t_w$ | Thickness of wall |
| $V_b$ | Base shear |
| $V_{us}$ | Shear to be resisted |
| $w$ | Weight of unit area of floor system |
| $W$ | Seismic weight of the building |
| $W_d$ | Wall density |
| $W_i$ | Weight of wall i |
| $Wi$ | Seismic weight of the $i^{th}$ floor |
| $X_{cm}$ | Centre of mass in X direction |
| $X_{cs}$ | Centre of stiffness in X direction |
| $X_i$ | Centroidal distance of wall panel in X direction |
| $X'_i$ | Distance of wall panel from centre of stiffness in X direction |
| $X_s$ | Centroidal distance of slab in X direction |
| $Y_{cm}$ | Centre of mass in Y direction |
| $Y_{es}$ | Centre of stiffness in Y direction |
| $Y_i$ | Centroidal distance of wall panel in Y direction |
| $Y'$ | Distance of wall panel from centre of stiffness in Y direction |
| $Y_s$ | Centroidal distance of slab in Y direction |
| $Z$ | Zone factor |
| $P_c$ | Density of concrete |
| $P_m$ | Density of masonry |

| | |
|---|---|
| $a_b$ | Bending stress |
| $a_d$ | Compressive stress due to dead load on wall panel |
| $a_{di}$ | Compressive stress due to dead and live load on wall panel |
| $a_t$ | Tensile stress |
| $T_c$ | Design shear strength of concrete |
| $T_t$ | Permissible tensile stress for masonry |
| $T_u$ | Permissible shear stress in $N/mm^2$ |
| $T_v$ | Nominal shear stress |
| $\Phi$ | Bar diameter |

# CHAPTER 1

# PROJECT DESCRITION AND BACKGROUND

## 1.1 Project Background

After 2015 earthquakes, an intervention program was presented in Bhimphedi community after the village's request to our organization (Amics del Nepal). The earthquake had left 85% of the families without a home. Bhimphedi has a population of 6,321 inhabitants (official census of 2001, the last one carried out in the country), 3,166 men and 3,155 women; 88.8% of the population is devoted to agriculture, in most cases not paid.

The main aim of the intervention was to strengthen and empower the community by providing knowledge and skills, in order to acquire a greater capacity for response to future earthquakes. This development program is intended to act both from the educational, productive and organizational perspectives, meaning that the population's capacity for management and decision-making will be highly improved. Therefore, the actions to be carried out will focus on **training** and **provididing technical assistance** to the people, in order to build earthquake-resistant houses, so that the population, in an active way, will have the know-how and will be able to solve future similar situations.

Several entities have partnered to facilitate this program: **Amics del Nepal NGO** contributes with the knowledge of the village, the population, the social situation, the culture, and provides general coordination; **Base-A NGO** contributes with their experience in cooperation projects, provides technical coordination along with an architect from Amics del Nepal, as well as volunteer coordination, monitoring and evaluation of the program, **CCD-UPC Polytechnic University of Catalonia** (UPC) contributes in the participation of volunteer students, as well as providing an expert advisor for the Program. Professor PhD. Architect Mr Pedro Lorenzo, who has extensive experience in anti-seismical reconstruction and construction in poverty and emergency situations, leads the program implementation and keeps track of all its actions. Both University students and Professor travel to the site with the economic support of CCD - Center for Development Cooperation at UPC University; **Agragaami Krishak Krishi Sahakaari**, a local agricultural cooperative in Bhimphedi, which knows the population and provides social organisation; and

**Rotary Club of Kantipur**, who provides legal, logistical and administrative support to the Awasuka Program.

All participant organizations in this program agree to understand that **BHIMPHEDI AWASUKA** is a **Technical Support Center for Habitat Improvement** created inside the AKKS Cooperative, coordinated by ADN and developed through the AWASUKA Program, by Amics del Nepal, Base-A, CCD-UPC and Rotary Club of Kantipur.

They also comprehend that Habitat -Aawaas- means the environment where a human being lives, which is made by physical factors (soil quality, safe houses, good quality water, healthy cooking…) and intangible relationships (better social organization, better hygiene, waste management…); and that AWASUKA will work to **convey all kinds of knowledge** to **foster habitat improvements** like: smoke-free kitchens, water treatment, hygiene diffusion, but being the main focus: earthquake resistant houses.

- **Program Name**

Awasuka Program: Habitat improvement and antiseismical reconstruction in Bimphedi, Nepal

Aawaas Sudhar Karyakram = Habitat Improvement Program

- **Location**

Country / District / Village: **Nepal / Makawanpur / Bhimphedi**

Address: Janajati Hall, **Bhimphedi Bazaar, Ward 2**

- **Duration**

Starting date: **July 2015** - Ending date: **August 2018**

### 1.2 Partnership and Lobbing

#### 1.2.1 Main Entities

**Amics del Nepal NGO:** General Coordination

Amics del Nepal is a Non-Governmental Organization registered in Barcelona, Catalonia, has been operating in Nepal since 1995 to improve the life-conditions of needy children, youths, women and other underprivileged groups, in the fields of health, education and community development, within a framework of sustainability and respect for the Nepalese culture. It also has the goal to disseminate the cultural and social current situation of Nepal in the Spanish society, through the organization of different awareness activities.

### 1.2.2 Collaborating Entities

- **Base-A NGO**: Technical Coordination and Volunteer Architects Coordination

Base-A is a non-profit association of young architects and students, founded in 2011 and set up in 2016 in Barcelona, Catalonia, which understands architecture as a tool for social transformation. Within the field of cooperation and training, it carries out activities in matters related to building, rehabilitation and urban planning; from a perspective of sustainable and participative development.

- **CCD - UPC University**: Expertise support and students' mobility program

The Center for Cooperation and Development (CCD) at the Polytechnical University of Catalonia (UPC) is a unit of the UPC Polytechnical University of Catalonia that was created in 1992 after its Social Council's initiative. Its mission is to foster active involvement of the UPC in cooperation and development and support the realization of initiatives in this field by all members of the UPC. It also develops training tasks and awareness activities to this problem. As a fundamental part of its mission to serve society, the University has the responsibility to participate actively in the promotion of solidarity and equity between peoples and in promoting a better human and sustainable development in the world, based on those activities that are their own: teaching, research and the transfer of knowledge and technology.

### 1.2.3 Local Partners

- **Agragaami Krishak Sahakari (Local Coordination in Bhimphedi )**

Agragaami KKS is an agricultural cooperative established in 2012 approved by the Government of Nepal within the Ministry of Finance and Revenue Division, affiliated to the Division Office of Cooperatives in Hetauda. AKKS's main functions are: distribution of fertilizers, insecticides, seeds and also to support and develop any kind of social program that will help improve the life conditions of the population. Access to safe housing meets this priority. In the aim of helping farmers, the Cooperative purchases fertilizer wholesale and so the villagers can get a better price.

- **Rotary Club of Kantipur (Logistical, legal and administrative support in Kathmandu)**

Rotary Club of Kantipur was founded and registered in Kathmandu in 2000, affiliated with the Rotary Club International. Its mission is to support social projects to improve the living conditions of the underprivileged people, in the fields of health, education and community development; as well as to foster conections, partnerships and cooperation projects between non-profit organizations and organize awareness activites within the society.

## 1.3 Location and Local Context

- **Country Background**

Nepal is an extremely underdeveloped country, it is considered one of the least developed and poorest countries in the world according to the UN report, despite the paradox of possessing a large untapped potential, for example in hydroelectric power, since it receives the great flow of the southern slope of the Himalayan mountain range. Part of the reason for this situation has to do with the socio-political and geographical environment.

Nepal is a very small country located between two giants: India and Tibet (China), from whom it depends on oil and energy supplies. On the other hand, a convulsive political situation in the country has influenced the economic slowdown and development, due to the recent decade of maoist insurgencies. Shortly after the earthquake, the government aproved the new constitution, a great challenge that had been lingering for a long time due to the difficulties in satisfying all the political forces that represented the longings and hopes of the Nepalese people.

These and other factors have halted Nepal's development, which should have been growing in relation to the untapped potential of its natural environment; thus today, with 27 million inhabitants, Nepal has a poverty line of more than 40%.

- **Village Context (Bhimphedi)**

Bhimphedi is a municipality of 6,000 inhabitants located in the mid-mountain area or "siwali" (between 1500 and 2500m), at an altitude of 1150 meters and about 60 kilometers south of Kathmandu (the capital of Nepal). It is located in the Mahabarat mountain-range, at the bottom of a valley with two rivers, Lamo Khola on the north and Rapati Khola on the south. It is one of the historical communities of the Makwanpur district, in the Narayani area in the Central Nepal region. The capital of the district is Hetauda, located 20 km south of Bhimphedi.

Until 1956 this municipality was an important crossing point between the Kathmandu valley and India. After the construction of the Bhainse-Kathmandu road and its subsequent transfer to Hetauda of the district capital, Bhimphedi began a progressive decline as a commercial hub, while the economy of its inhabitants also fell, as well as the opportunities for development, already small in most of Nepal.

This situation also led the population to exploit the mountain in northeast of Mahabarat to use it as crops, to the detriment of the forest. The old thick forest full of flora and fauna has disappeared to give rise to areas of culture. Deforestation aggravates the problem of landslides, began in 1954 with monsoon rains, which caused some little villages to disappear, including the Ward of Dhorsing, and affected severely road communications to Hetauda. Neither the local authorities nor the central government took initiatives to avoid this degradation of the ecosystem and create a sustainable outlet for the region.

Recently a new road communicating Kathmandu and Bhimphedi was inaugurated: it follows the path of the former walking path. The track is paved in almost all its way and it allows traveling from Kathmandu to Bhimphedi in less than three hours, as long as there is no heavy traffic or landslides. This road is pushing Bhimphedi to get back on track.

In Bhimphedi 80% of the population lives on very basic activities: unpaid agriculture and a very weak trade. Its inhabitants live in precarious conditions and have a fatalistic attitude towards adversity. Moreover, the population has another problem related to geographic and socio-economic situation of this district: the access to quality education. According to official data, 16% of children under 18 do not go to school, mainly because of the long distances separating their households from the school centers.

The local authorities and the central government of Nepal have not undertaken any planned initiatives to improve the economy of the area and to facilitate the education of its inhabitants.

- **The Earthquake in Bhimphedi**

An earthquake of 7.9 degrees on the Richter scale shook Nepal in April 25$^{th}$ 2015. Its epicenter was 150 kilometers west of Kathmandu. This earthquake has been considered the country's worst natural disaster since 1934: it caused more than 10,000 deaths. In the Bhimphedi community, 85% of its buildings were seriously affected. Fortunately, most of the population was outside their homes that day, that's why in Bhimphedi there were neither serious injuries nor deaths.

Bhimphedi was highly affected by the 2015 earthquakes, as shown in the following images. Former Bhimphedi VDC had 9 wards and 1110 registered homes, from which 30% were totally destroyed, 30% were damaged, and 40% remained intact.

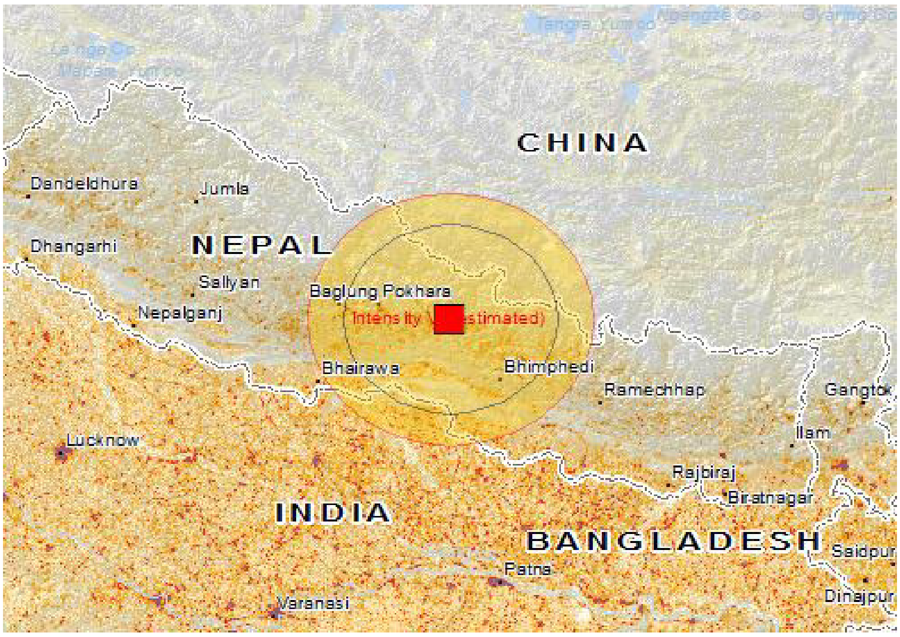



Figure: 1 Epicenter and action radius of the 7'9 degrees earthquake in Richter scale. Bhimphedi is inside the affected area.

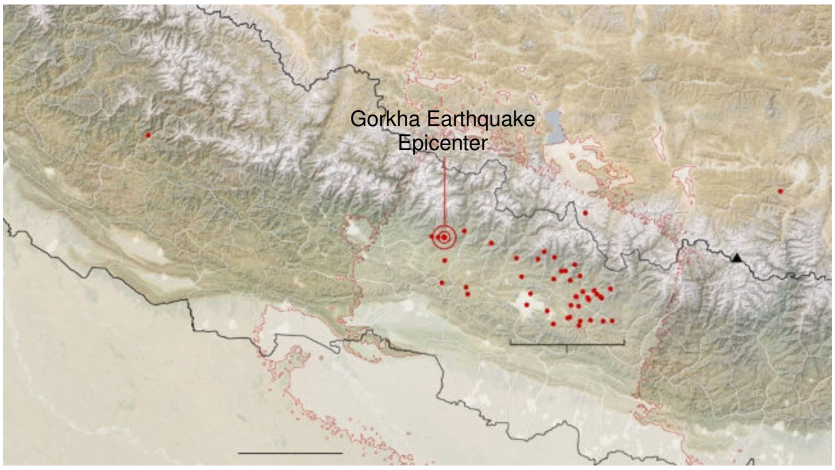



Figure 2: Bhimphedi in the middle of the epicenters of earthquake and aftershocks.

The rural community of **Supin**, one of Bhimphedi VDC's former Wards, was the most affected one during the 2015 earthquakes. That's why the program will start in Suping area and from there it will replicate accordingly.

### 1.4 Project Summary

As previously mentioned, out of 1,100 registered houses in Bhimphedi, 200 were destroyed by the earthquakes, 600 were seriously damaged and only 300 remained intact. As a result of the large number of affected houses, Amics Del Nepal decided to start a program for habitat improvement in this village because these two reasons:

1. Amics del Nepal already knew Bhimphedi due to cooperation programs held in the village. Amics manages Bhimphedi Balmandir Children's Home since 2006 and has also managed several Community Development Programs in this village. This is the reason why, after the earthquake, Bhimphedi community requested technical help to Amics NGO.

2. After the 2015 earthquake Amics del Nepal had many NGOs contacting them to offer their technical help and collaboration on reconstruction in Nepal. These entities were the above mentioned: BASE-A and CCD-UPC. Later on, Agragaami and Rotary Club also joined the team.

After many meetings amongst the different organisations, it was decided to start this program with an identification trip led by the three NGOs: Amics del Nepal, Base-A and CCD-UPC. During the trip many meetings with local private and public institutions took place, both at urban and rural levels. Additionally, more than sixty houses were visited, all in the different wards of Bhimphedi, in order to study their damage degree, related to their construction typology. A detailed form was drafted from each and every visited house. Through their typology analysis some interesting findings on technical improvements were established. The visited homes were referenced and located in Google Maps, coded with a different color depending on its Seismical Damage Degree (SDD). After the trip, the technical team led by Pedro Lorenzo made a comprehensive report which established the basis of the program: contributing to improve the habitat of the population affected by earthquakes, both through their homes and their living conditions, in order to improve their response to new earthquakes. The program name is AWASUKA, after the Nepalese words Aawaas Sudhar Karyakram, meaning: Habitat Improvement Program.

The aim of AWASUKA is to improve two equally important aspects: social organization and building techniques. From the technical field, there will be two different lines of action: support in reconstruction and retrofitting. Different antiseismic technique tests will be developed, always using local materials and improved techniques; and contributing to the training of "mistris", in correcting the inefficient use of traditional techniques. From the social organization, participative design workshops and social trainings will be held, to strengthen the population's organizational capacity, management and decision making. This entails that the action to be taken does not involve a dependence of the population, but rather makes it active, having the knowledge to react positively to new earthquakes.

In Nepal, central government is acting in a logical way regarding reconstruction, but the true reality is translated into a lack of technical support in the remote areas. In rural local government, the organizational level is very poor: it is still based on family subsistence economy. Even though, the creation of local cooperatives is growing and this is strongly benefiting the development and the habitat improvements undertaken by the program.

In Bhimphedi, the agricultural cooperative Agragaami will be in charge of hosting the **Technical Support Center for Habitat Improvement** called **Bhimphedi AWASUKA**, which will be coordinated by a the entities participating in Awasuka Program. This will provide technical support to all the villagers in Bhimphedi, in close collaboration with the government.

### 1.5 Construction Techniques and Methodology

Many research activities have been conducted in the site before starting the practical tests. This has been an enormous amount of work, but with a very small visible impact. The site has been studied from various perspectives: technical bibliography existing in the country, current regulations, available local materials and dimensions; typical tools and traditional techniques, the site's geology, etc…

#### 1.5.1 Confined Concrete Block Design

The confined concrete block masonry technique used in this new model house is different than the ones proposed by the Government Catalogue in its 2nd Volume. The reasons for having chosen concrete block are: its **low cost** compared to wood or brick, its **fast execution** and the **higher need for improvements** in the current local techniques people are using in block construction.

Regarding the layout design, the houses must meet the needs of each and every family. Therefore, the house design is flexible and it provides diferent variations depending on the number of rooms and storeys. Spaces are adaptable. The most demanded houses have one floor, with one or two rooms and one verandah. Two storey houses with four rooms, verandah and optional corridor, are also demanded. Acording to the family needs the inner spaces can be adapted to different functions: usually all spaces are connected, but they can be partitioned if necessary. Upper storeys can either be used as rooms (chota) or for agricultural storage (buikal).

In addition, to contribute to a better thermal insulation, the CGI roof with insulated with bamboo canes. Optionally, the concrete blocks can be filled with vegetable fiber and thus become more insulating.

### 1.5.2 Construction Process

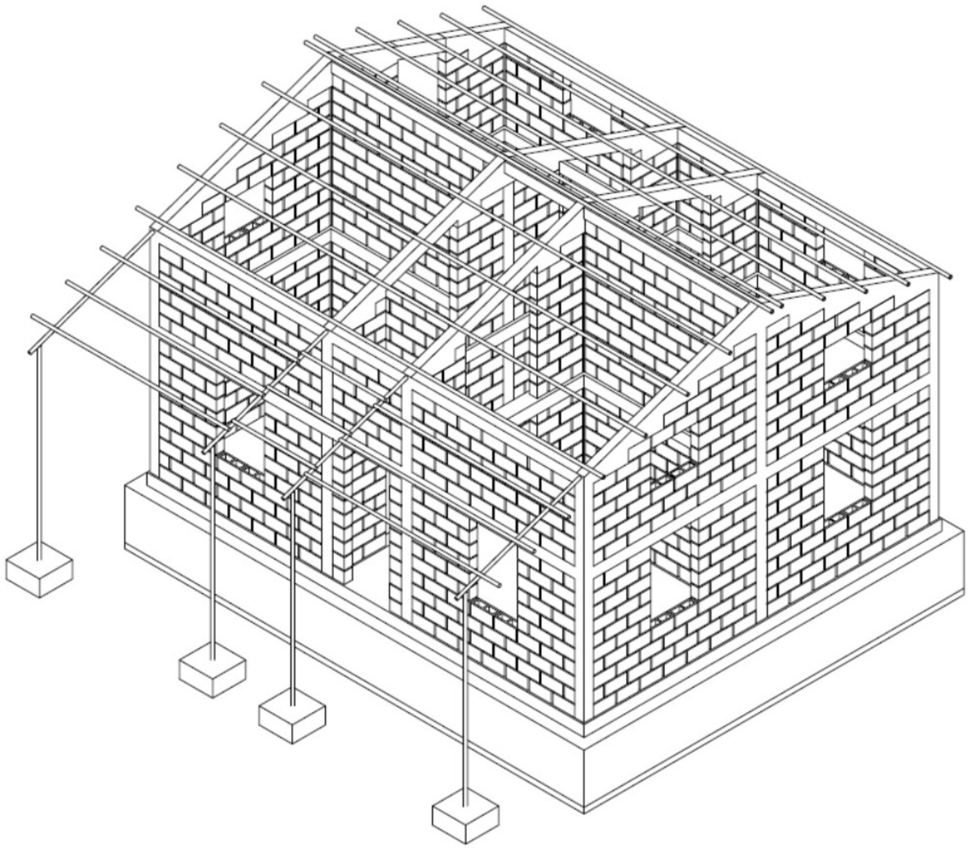


Figure 3: 3D view of confined masonry building

- **Foundations**

Dig the trenches for mat foundations which will be built with stone masonry and concrete. Isolated foundations will be done with concrete. Excavation will always be done by hand, except in the case of finding the rock bed very close to the surface. Vertical rebars for walls and pillars will be embeded in the mat foundations and will have concrete sorrounding them thanks to a plastic pipe.

- **Plinth Beam**

Rebars and stirrups of the plinth beam will be laid on the mat foundations, leaving the vertical rebars go through it. Plinth beam will be concreted.

- **Concrete Block Walls (Ground Floor)**

The block walls will be constructed, puting a horizontal rebar every 2-3 block rows and concreting around the vertical rebars. The confining pillars will not be concreted until the walls reach the tie-beam level.

- **Concreting of Confining Pillars (Ground Floor)**

Once the walls have reached the tie-beam level, the confining pillars will be concreted using formwork on both sides of the walls.

- **Floor Tie-Beam**

After pillar-concreting, rebars and stirrups of the tie-beam will be laid on the walls, ready to be concreted.

- **Concrete Block Walls (First Floor)**

The block walls will be constructed, laying a horizontal rebar every 2-3 block rows and concreting around the vertical rebars. The confining pillars will not be concreted until the walls reach the roof tie-beam level.

- **Concreting of Confining Pillars (First Floor)**

Once the walls have reached the roof tie-beam level, the confining pillars will be concreted using formwork on both sides of the walls.

- **Roof Tie-Beam**

After pillar-concreting, rebars and stirrups of the tie-beam will be laid on the walls, ready to be concreted. This tie-beam will not be horizontal but inclined; hence, the concreting will be done in different stages to ensure the right inclination of the beam.

- **Roof Iron Structure**

Round iron-pipe purlins will be fixed to the roof tie-beams in order to form the roof structure. For the verandah structure, iron-pipe rafters will be fixed to the roof tie-beam in one side and on the verandah's posts on the other. Then, round iron-pipe purlins will be fixed on the rafters. All connections will be made with angles and bolts, to avoid welding. (Welding staff is hard to find in remote areas, that's why it is avoided as much as possible).

- **Roof Finishing CGI Sheet**

The roof finishing in house and verandah will be CGI sheets. They will be fixed on the purlins with the traditional U-bolts. Thermal insulation will be ensured with bamboo rods placed at the purlins' level, before the fixing of the CGI sheets takes place.

### 1.6 Program Viability and Environmental Impact

#### 1.6.1 Socio-Cultural Viability

**• Adaptation to socio-cultural factors in the area**

The program arises from the identification of needs for habitat improvement expressed by the population of Bhimphedi. In addition, the people participate voluntarily in the construction of their houses to learn about anti-seismical techniques. Moreover, awareness talks and practical lectures are held in Awasuka office to convey knowledge and increase motivation.

**• Actions in the most vulnerable population**

The program aims to reach the most needed population. The goal is to improve the community's resilience and encourage the women's involvement, so that they will be aware of the new techniques and thus will be able to spread them later.

**• Collaboration with local authorities**

The local government has given its approval to the program, given the possibility of complementation between the AWASUKA program and the government's government aids. A future collaboration is expected when Awasuka Design is submitted to the government, in order to receive NRA aproval.

#### 1.6.2 Technical Viability

Houses are made using local technologies and adding anti-seismic improvements to traditional techniques, such as diagonal bracings. In all cases the workforce is local people trained in earthquake-resistant improvements; this yields a win-win relationship, as the locals always have improvement suggestions on their side and hence the construction process is improved from both sides.

#### 1.6.3 Environmental Impact and Sustainability

Buildings will be built using local materials, therefore the impact on transportation is minimal and the $CO_2$ emissions will be minimized during the construction process. In addition, most materials are sustainable and, in some cases, reusable: clay, sand, gravel, wood. The only materials coming from Hetauda will be cement, concrete blocks and iron sheets.

As secondary evironmental objectives, several actions and activities are being performed to solve other habitat problems: workshops and awareness programs to raise awareness for water treatment, improvement of the latrine's design and functioning, and dissemination of chimney construction to achieve healthier and safe cooking.

# CHAPTER 2

# CONFINED MASONRY BUILDING DESIGN AND ANALYSIS

## 2.1 Introduction

The basic requirement for seismic design of buildings ie Simplicity, symmetry, material property etc, as given in various IS codes are also applicable for confined masonry. So, all available IS codes such as IS: 4326-2013, IS: 1893-2002, IS 13920-2016, IS 456-2000 are applicable for design of confine masonry buildings. The design procedure of confined masonry building includes following requirements:

- Symmetricity of plan and elevation, appropriate location of bond beams and tie-columns and size of confinement are necessary factors.
- Appropiate selection of material properties for hollow block masonry, cement motar, concrete, reinforcement etc.
- Building load calculations such as dead load, live load and seismic load. Also additional detailed calculation for building weight, shear, lateral load distribution and calculation of equivalent wall stiffness, center of mass and building stiffness, eccentricity, torsional stiffness, and lateral seismic load distribution in each individual walls.
- After computation various checks should be carried out for various building parameters such as wall density, inplane stability (compressive stress, tensile stress and shear stress), overturning, out of plane stability of wall panels.

- Finally, bond beams and tie colums are to be designed

## 2.2 Building Description and components

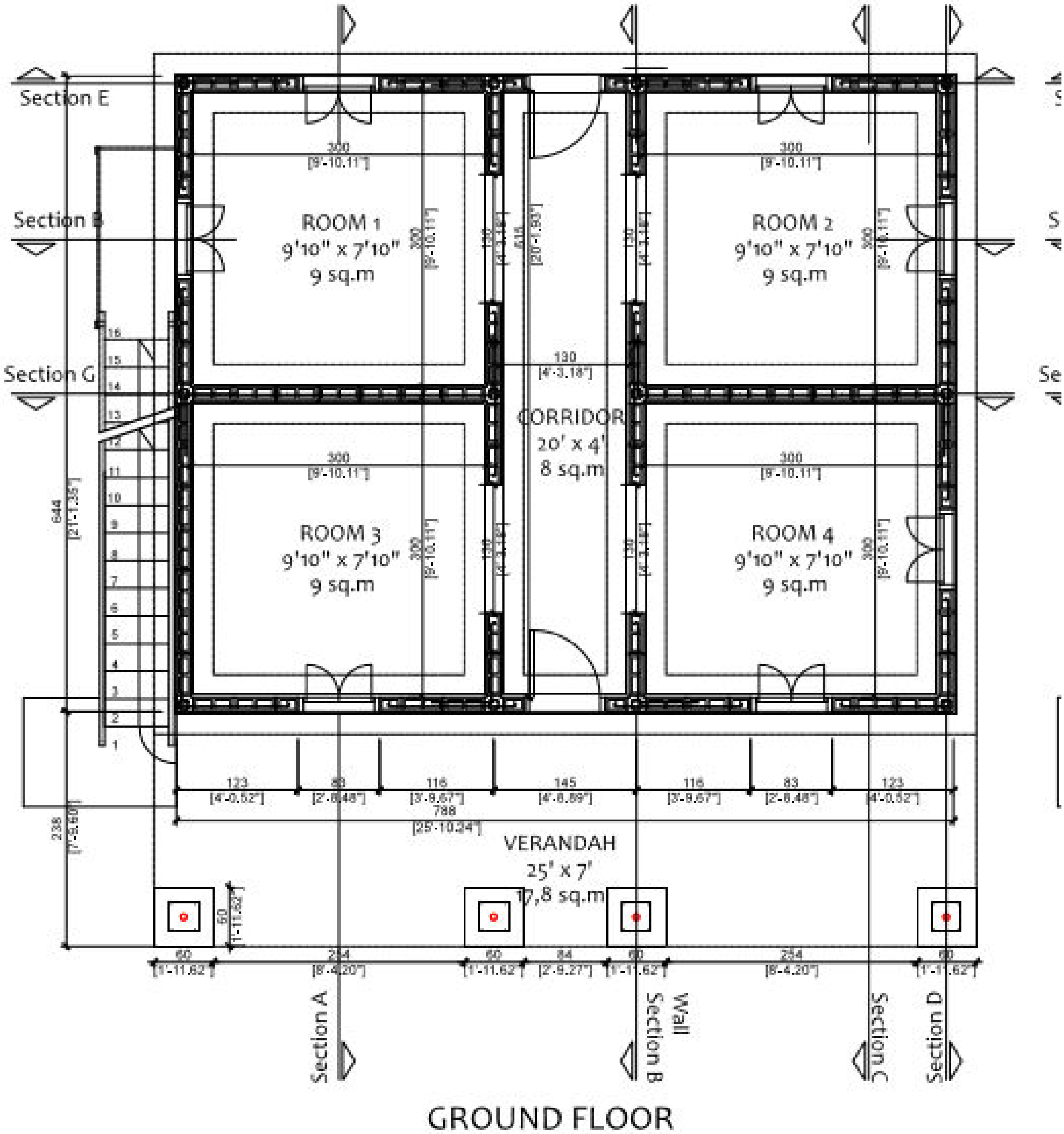



Figure 4: Ground Floor Plan of Building

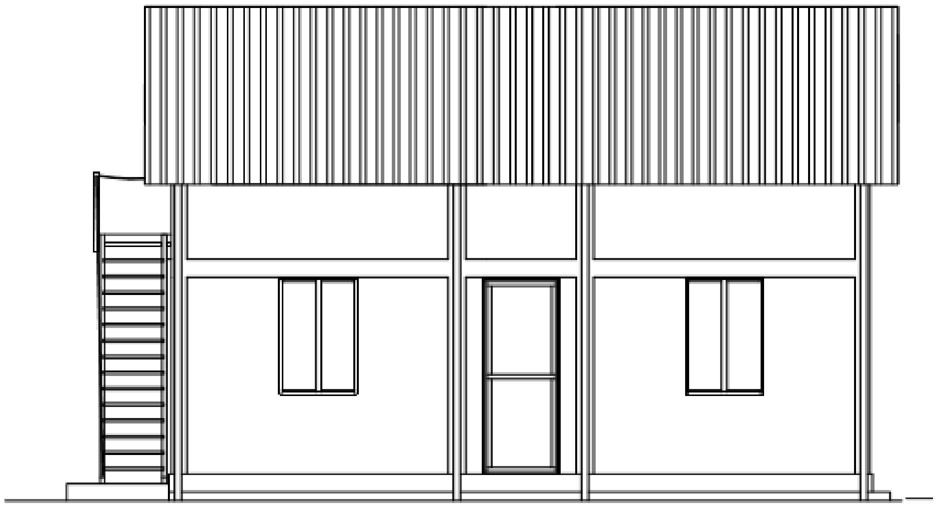



Figure 5: Front Eleveaantion of Building

The Figure 4 and Figure 5 represent plan and elevation of the building respectively. The structural system of builing consists of Confined concrete block wall with vertical and horizontal reinforcement at each 80cm and each side of openings. It also contains reinforced Bond Beams of (15cm X 20cm) at each floor level and Tie column (15cm X 20cm) at each corner and wall connections. At ground floor it consists of wooden floor system with proper connection system. The roofing system consists of hollow pipe connection of wall beam and CGI sheet roofing system.

The building geometry used for construction cab be shown as follows:

Table 1: Building Geometry for Confined Masonry Design

| Sno. | Component | Dimension | Unit |
|---|---|---|---|
| 1 | Plan in X-direction | 7880 | mm |
| 2 | Plan in Y-direction | 6440 | mm |
| 3 | No. of Storey | Ground Floor + Attic | |
| 4 | Storey Height | G.Floor = 2420 / Attic = 1895 (avg) | mm |
| 5 | Bond Beam | 150 x 150 | mm |
| 6 | Tie Column | 150 x 150 | mm |
| 7 | Wooden Floor | 180 | mm |
| 8 | Door Opening | G.Floor = 825 x 2190 / Attic = 825 x 1820 | mm |
| 9 | Window (G. Floor) | 825 x 1340 | mm |
| 10 | Window (Attic) | 825 x 660 | mm |

### 2.3 Assumptions and Basic Data

For design and analysis of confined masonry hollow block building following assumptions are made:

- Building walls are analysed individual panels.
- Both in plane and out of plane behavior are considered to be independent.
- Site location and soil properties determine the foundation system.

As per building materials used for construction the basic material properties used for design and analysis are as follows:

Table 2: Material Properties for Confined Masonry Design

| Sno. | Component | Parameter | Value | Unit |
|---|---|---|---|---|
| 1 | Hollow Block | Compressive Strength | 5 | $N/mm^2$ |
| 2 | Motar | Compressive Strength | 3 | $N/mm^2$ |
| 3 | Block Masonry | Density | 16 | $KN/m^3$ |
| | | Tensile Strength | 0.25 | $N/mm^2$ |
| 4 | Concrete | Density | 25 | $KN/m^3$ |
| | | Grade | 20 | $N/mm^2$ |
| 5 | Steel | Grade | 500 | $N/mm^2$ |
| 6 | Wood | Density | 900 | $Kg/m^3$ |
| 7 | CGI Sheet | Grade | 14 | g |

### 2.4 Design Parameters

All the wall panels in X-direction and Y-direction are named as A,B,C,D,E,F,G, H and 1,2,3,4,5,6,7,8 respectively. The required design parameters required as calculated as plan and elevation with reference with given names. The stability and required properties are checked for one of wall panel A.

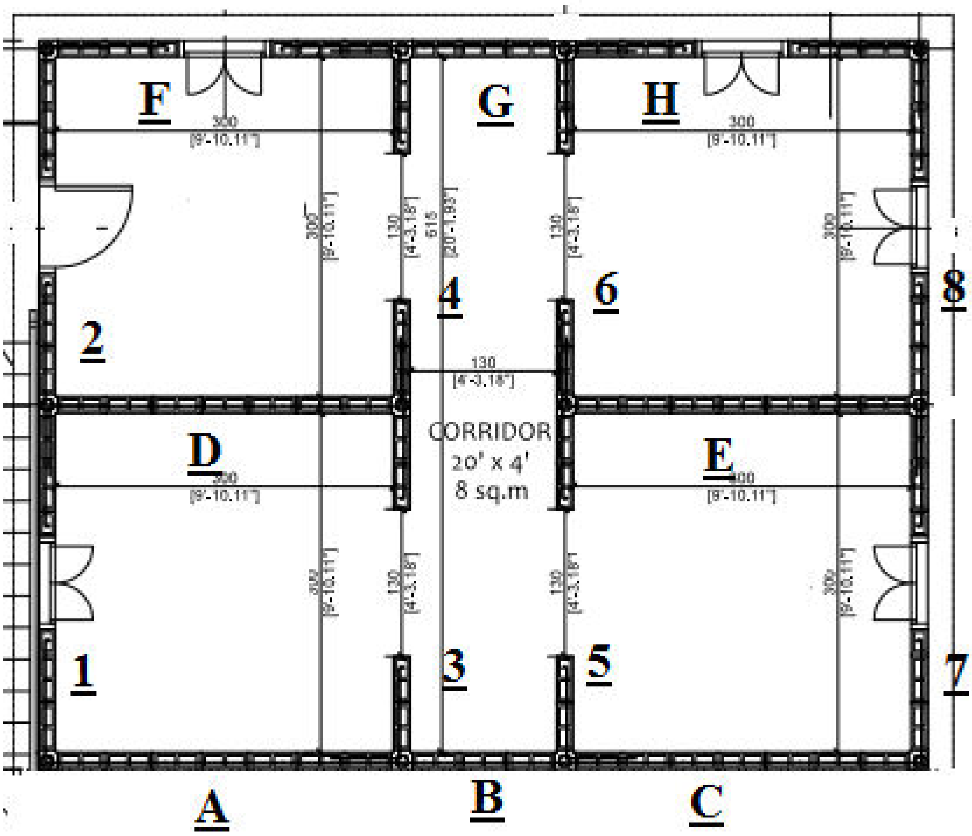



Figure 6: Name for each wall pannel

### 2.4.1 Dead Load

Dead Load is the self weight of building materials. The calculated dead load for various build components are as follows:

Table 3: Dead Load of Building Components

| Sno. | Building Component | Dead Load | Unit |
|---|---|---|---|
| 1 | Wall weight | 191.77 | KN |
| 2 | Bond Beam Weight | 43.49 | KN |
| 3 | Weight of tie column | 22.545 | KN |
| 4 | Wooden Floor | 4.65 | KN |
| 5 | Metal Roof | 11.058 | KN |
| **Total** | | **273.513** | **KN** |

### 2.4.2 Live Load

The live load consists of load on floor load and roof. As per IS code 875 (part 2) live load in various occupancies in the building can be taken as follows:

Table 3: Live Load for various building occupancy

| For Residential Building | | | |
|---|---|---|---|
| **A** | **Floors** | **Load** | **Unit** |
| 1 | Live load for Bed, living Rooms etc. | 2 | $kN/m^2$ |
| 2 | Live load for Kitchen & Dining Room | 2 | $kN/m^2$ |
| 3 | Live load for Toilet Floors | 2 | $kN/m^2$ |
| 4 | Live load for corridor, passage, staircase, balconies floor | 3 | $kN/m^2$ |
| **B** | **Roof** | **Load** | **Unit** |
| | Live Load for Roof (access provided ) | 1.5 | $KN/m^2$ |
| | Live Load for Roof (access not provided ) | 0.75 | $KN/m^2$ |

### 2.4.3 Wall Density

Wall density is a key indicator for the safety of confined masonry buildings subjected to seismic and gravity loads. Wall density (Wd) can be defined as the total cross-sectional area of all walls, Aw (product of wall thickness and wall length), in each direction divided by the plan area, Ap.

| | | |
|---|---|---|
| Wd (%) = | $A_w/A_p$ x 100 | wall area in x,y/Total area |
| Where, | $A_w$ = | Area where wall lies |
| | $A_p$ = | Total Plinth Area |

CM buildings with sufficient wall density performed well during the major earthquakes in contrast to CM buildings with relatively low wall density. Primarily, a minimum 2% wall density is required for CM buildings located in seismic zone II and III, while for building in seismic zones IV and V, the minimum requirements are 4% and 5% respectively, in each principal direction. These wall densities are 33% higher, if hollow concrete blocks are used in CM construction.

The minimum wall density required for confined masonry building located in seismic zone V is 5%, but since hollow concrete blocks are used for construction the wall densities must be increased by 33% ie 5.33%. The calculation of wall density is carried out as follows:

Floor area per floor = 50.7472 $m^2$

Total floor area for 2 floors (this is a two-storey building):

TOTAL FLOOR AREA = 101.4944 $m^2$

In X-direction

| | | |
|---|---|---|
| Ground Floor Wall Area = | 5.4273 | |
| $W_{d,x}$ (%) | 5.35 | >5.33% ,hence ok |
| In Y-direction | | |
| Wall Area in Y-direction | | |
| Ground Floor Wall Area = | 6.168 | |
| $W_{d,y}$ (%) | 6.077183 | >533% ,hence ok |

### 2.4.3 Wall Stiffness and Torsion

#### 2.4.3.1 Equivalent Wall Stiffness

The equivalent stiffness of walls is dependent on boundary conditions, young's modulus of masonry and dimensions of the pier. It can be calculated by spring analogy method for individual piers, with respect to the openings, as shown in Figure 7 and can be expressed as:

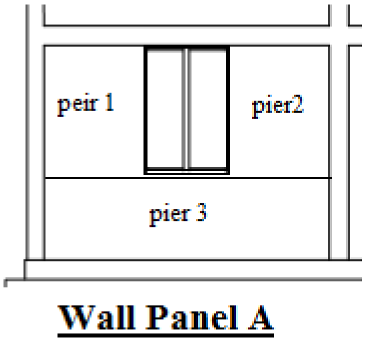



Figure 7: Wall Panels Divided into Piers

Stiffness of a peir,

$K_f = (E_m t_w)/(h_w/l_w)^3+3(h_w/l_w)$

Equivalent stiffness of a wall panel can be calculated using the expression:

$K_w = 1/ (1/K_1+1/K_2+1/K_3)$

Where $K_1$, $K_2$, and $K_3$ are the stiffness of different piers. The stiffness of wall panels in X and Y directions are added separately to obtain building stiffness in both the directions.

The equivalent wall stiffness for panel A is calculated as:

| | | | |
|---|---|---|---|
| Peir 1 (openingleft) | K1= | 0.026839 | Em |
| Peir 2 (openingright) | K2= | 0.026839 | Em |
| Peir 3 (openingbelow) | K3= | 0.127363 | Em |

Equivalent Stiffness wall Panel A

| | | |
|---|---|---|
| Kw = | 0.012140 | Em |

Simillarly, for all panels equivalent wall stiffness are shown in table below:

Table 4: Equivalent Wall Stiffness for Each Wall Panels

| Sno. | Wall Panel | Stiffness (mE) |
|---|---|---|
| 1 | A | 0.01214 |
| 2 | B | 0.00008 |
| 3 | C | 0.01214 |
| 4 | D | 0.03750 |
| 5 | E | 0.03750 |
| 6 | F | 0.01214 |
| 7 | G | 0.00008 |
| 8 | H | 0.01214 |
| 9 | 1 | 0.03750 |
| 10 | 2 | 0.01214 |
| 11 | 3 | 0.01230 |
| 12 | 4 | 0.01230 |
| 13 | 5 | 0.01230 |
| 14 | 6 | 0.01230 |
| 15 | 7 | 0.01214 |
| 16 | 8 | 0.01214 |

| | | |
|---|---|---|
| Stiffness in x-direction (ΣKx)= | 0.12373 | mE |
| Stiffness in y-direction (Σky)= | 0.12313 | mE |

### 2.4.4 Torsion

#### 2.4.4.1 Centre of Stiffness and Centre of Mass

Masonry buildings with horizontal irregularities and lack of symmetry may have considerable eccentricity. It arises when centre of stiffness and centre of mass do not coincide with each other. Eccentricity gives rise to torsion which needs to be considered in seismic analysis of confined masonry buildings.

Centre of stiffness in X and Y directions are given as:

$X_{cs}= \Sigma k_{yi}.X_i/\Sigma k_{yi}$ and

$Y_{cs}= \Sigma k_{xi}.Y_i/\Sigma k_{xi}$

Centre of mass in X and Y directions are given as:

$X_{cm}=\Sigma W_i X_i/\Sigma w_i$ and

$X_{cm}=\Sigma W_i y_i/\Sigma w_i$

Where $X_1$, $X_2$ ........ ,$X_n$ and $Y_1$, $Y_2$, ....$Y_n$ are centroidal distance of wall panels in X and Y direction respectively and $w_1$, $w_2$,...,$w_n$ are weight of individual wall panels. $X_s$ and $Y_s$ are centroidal distance of slab in X and Y direction respectively and $m_s$ is weight of slab.

The center of stiffness and center of mass for the hollow concrete confined masonry building is calculated as follows:

Table 5: Center of Stiffness of the building

| Sno. | Wall Panel | Centroid Distance (mm) | Center of stiffness in X-direction (mm) | Center of stiffness in y-direction (mm) |
|---|---|---|---|---|
| | | | Xcs= Σkyi.Xi/Σkyi (mm) | Ycs= Σkxi.Yi/Σkxi (mm) |
| 1 | A | 1570 | 0 | 0 |
| 2 | B | 3882 | 0 | 0 |
| 3 | C | 6538 | 0 | 0 |
| 4 | D | 1570 | 0 | 475.85 |
| 5 | E | 6538 | 0 | 1981.60 |
| 6 | F | 1570 | 0 | 154.05 |
| 7 | G | 3882 | 0 | 2.57 |
| 8 | H | 6538 | 0 | 641.53 |
| 9 | 1 | 1575 | 0 | 0 |
| 10 | 2 | 3145 | 0 | 0 |
| 11 | 3 | 1575 | 157.35 | 0 |
| 12 | 4 | 3145 | 314.21 | 0 |
| 13 | 5 | 1575 | 157.35 | 0 |
| 14 | 6 | 3145 | 314.21 | 0 |
| 15 | 7 | 1575 | 155.30 | 0 |
| 16 | 8 | 3145 | 310.10 | 0 |
| **Total** | | | **1408.52** | **3255.61** |

Table 6: Center of Mass of the building

| Sno. | Wall Panel | $X_i$ (mm) | $Y_i$ (mm) | Wall Weight ($W_i$) | $Xcm=\Sigma W_iX_i/\Sigma w_i$ | $Xcm=\Sigma W_iy_i/\Sigma w_i$ |
|---|---|---|---|---|---|---|
| 1 | A | 1570 | 0 | 13.24 | 108.42 | 0.00 |
| 2 | B | 3882 | 0 | 3.32 | 67.10 | 0.00 |
| 3 | C | 6538 | 0 | 13.24 | 451.48 | 0.00 |
| 4 | D | 1570 | 3200 | 15.91 | 130.27 | 265.51 |
| 5 | E | 6538 | 3200 | 15.91 | 542.48 | 265.51 |
| 6 | F | 1570 | 6250 | 17.56 | 143.78 | 572.38 |
| 7 | G | 3882 | 6250 | 3.32 | 67.10 | 108.04 |

| | | | | | | |
|---|---|---|---|---|---|---|
| 8 | H | 6538 | 6250 | 17.56 | 598.76 | 572.38 |
| 9 | 1 | 0 | 1575 | 15.91 | 0.00 | 130.68 |
| 10 | 2 | 0 | 3145 | 13.24 | 0.00 | 217.18 |
| 11 | 3 | 3150 | 1575 | 9.02 | 148.11 | 74.05 |
| 12 | 4 | 3150 | 3145 | 9.02 | 148.11 | 147.87 |
| 13 | 5 | 4600 | 1575 | 9.02 | 216.28 | 74.05 |
| 14 | 6 | 4600 | 3145 | 9.02 | 216.28 | 147.87 |
| 15 | 7 | 7750 | 1575 | 13.24 | 535.17 | 108.76 |
| 16 | 8 | 7750 | 3145 | 13.24 | 535.17 | 217.18 |
| | | | **Σwi** | **191.77** | **3908.52** | **2901.48** |

### 2.4.4.2 Torsional Stiffness

Torsion in a building can result into twisting moment and thus torsional stiffness needs to be considered in analysis. It can be expressed as:

$K_t = \Sigma K_{xi} Y_i^2 + \Sigma K_{yi} X_i^2$

Where, $X_i$ and $Y_i$ are distances of wall panels from centre of stiffness in X and Y direction respectively.

The torsional stiffness for the hollow concrete confined masonry building is calculated as follows:

Table 7: Torsional Stiffness of the building

| Sno. | Wall Panel | Torsional Stiffness (mE) |
|---|---|---|
| 1 | A | 0.30 |
| 2 | B | 1.86 |
| 3 | C | 5.26 |
| 4 | D | 1.57 |
| 5 | E | 6.53 |
| 6 | F | 5.14 |
| 7 | G | 6.69 |
| 8 | H | 10.10 |
| 9 | 1 | 0.31 |
| 10 | 2 | 1.22 |
| 11 | 3 | 1.53 |
| 12 | 4 | 2.45 |
| 13 | 5 | 2.91 |
| 14 | 6 | 3.83 |

| | | |
|---|---|---|
| 15 | 7 | 7.70 |
| 16 | 8 | 8.62 |
| **Total ($K_t$)** | | **66.01** |

### 2.4.4.3 Eccentricity

Eccentricity is the difference in centre of mass and centre of stiffness.

In X-direction

$e_x = X_{cm}-X_{cs}$

In Y-direction

$e_y = Y_{cm}-Y_{cs}$

Design eccentricity (edi) can be calculated as (IS: 1893-2002):

Design Eccentricity ($e_{di}$) = $1.5e_x + 0.05b_i$

Design eccentricity is to be calculated in both the directions according to the floor plan dimension perpendicular to the direction of force ($b_i$). The maximum ($e_{di}$) among both the directions shall be considered is designs.

The calculated eccentricity for the confined masonry building can shown as below:

| | |
|---|---|
| $e_x$ | 2500.00 |
| $e_y$ | -354.13 |

The design eccentricity in both directions

| | | |
|---|---|---|
| Design Eccentricity in X-direction | 4143.993 | 2105.995027 |
| Design Eccentricity in Y-direction | -209.199 | -676.132646 |

## 2.5 Design Lateral Force

Nepal lies in the $V^{th}$ zone which is at high risk of vulnerability. So, there is high demand for the earthquake resistant design of the building for saving from these devastating disasters.

Earthquake is a shaking of the earth surface caused by the waves originated underneath and on the surface of earth. Earthquake causes are volcanic eruption, slipping of faults, tectonic activities, explosion etc…..

Structural design with the sound knowledge of structural engineering determines the sizes of members like beam, column, rebar arrangements and others. These structures are subjected to various loads like concentrated loads, uniformly distributed loads, uniformly varying loads, random loads, internal or earthquake load and dynamic forces. The structure transfers its load to the support and ultimately to the ground. While transferring the loads acting on the structures, the members of the structures are subjected to internal forces like axial force, shear force, bending and torsion moments. Structural analysis deals with analyzing these internal forces in the members of the structures.

For computation of design lateral forces due to earthquake in confined masonry building IS 1893 : 1975 code for masonry building can be adopted. The lateral force calculation involves various which steps by which lateral force distribution in each wall pannel is calculated. The seismic parameters for confined masonry design can be using IS 1893 : 1975 as follows:

Table 8: Seismic Parameter for Design of Confined Masonry

| Sno. | Seismic Parameter | Value |
|---|---|---|
| 1 | Zone Factor (Z), V | 0.36 |
| 2 | Response Reduction Factor (R ) | 2,5 |
| 3 | Importance Factor (Residential) | 1 |
| 4 | Soil type medium | II |

### 2.5.1 Building Weight

The building weight includes all dead load/weight of building. In this confined masonry building building weight includes wall load, bond beam, tie column, floor and roof. The building weight is calculated in table 3, from which total building weight is **273.513 KN.**

### 2.5.2 Base Shear

Base shear is the maximum expected lateral force that will occur due to seismic ground acceleration at the base of the structure . The base shear, or earthquake force, is given by the symbol "$V_B$" and Base shear of a building is computed as per IS: 1893 (2002).

Table 9: Calculation of Base Shear

| | | |
|---|---|---|
| Height of Building | 3.93 | m |
| Time period (T) = 0.09H/d^1/2 | | |
| Time period in X-direction | 0.13 | |
| Time period in Y-direction | 0.14 | |
| $A_h$= | ZISa/2Rg | |
| $A_h$ in X-direction | 0.18 | |
| $A_h$ in Y-direction | 0.18 | |
| $V_B$= | W x $A_h$ | |

| | | |
|---|---|---|
| $V_B$ in X-direction | 49.23 | KN |
| $V_B$ in Y-direction | 49.23 | KN |

### 2.5.3 Distribution of Design Lateral Force

The distribution of lateral load for every floor along the height of the building is considered in both X and Y directions, separately. The design lateral force distribution along the height of building can be obtained as :

Design Later Force ($Q_i$) = $W_i h_i^2 / \Sigma W_i h_i^2 \text{ X } V_B$

The designed lateral force in both X and Y direction is calculated as follows:

Table 10: Lateral Force distribution

| | | |
|---|---|---|
| In X-direction | 49.23 | KN |
| In Y-direction | 49.23 | KN |

### 2.5.4 Distribution of Seismic Force into Individual Panels

The storey shear is distributed into individual wall panels in the given direction. The wall panels are subjected to both lateral and torsional loads. The force due to lateral translation is based on storey

shear and stiffness of wall panels, which can be calculated as:

Force due to Lateral translation

In X-direction

$F_{lxi} = (Q_i / \Sigma K_{xi}) \text{ X } K_{xi}$

In Y-direction

$F_{lyi} = (Q_i / \Sigma K_{yi}) \text{ X } K_{yi}$

Similarly, the force due to torsion is based on storey shear, design eccentricity and torsional stiffness of the building and can be calculated as:

Force due to Torsion

In X-direction

$F_{txi} = (Q_i e_{dy} / K_t) Y'_i \Sigma K_{xi}$

In Y-direction

$F_{tyi} = (Q_i e_{dx} / K_t) X'_i \Sigma K_{yi}$

Thus, the total force i.e. algebraic sum of force due to lateral translation and torsion, is considered for evaluating wall panels for its in-plane safety.

Total Force, Pi = Fn + Fti (to be calculated for each wall panels in both X and Y directions)

The distribution of seismic forces in panel A is calculated.

Force due to Lateral translation

For Wall panel A = 4.83 KN

Force due to Torsion

For Wall panel A = 0.21 KN

Total Force acting on Wall Pannel A = 5.04 KN

Simillarly, for all panels total seismic force acting are shown in table below:

Table 11: Distribution of seismic forces in each wall panels

| Sno. | Wall Panel | Total Lateral Force (Pi)KN |
|---|---|---|
| 1 | A | 4.94 |
| 2 | B | 0.24 |
| 3 | C | 5.03 |
| 4 | D | 15.12 |
| 5 | E | 15.12 |
| 6 | F | 5.03 |
| 7 | G | 0.24 |
| 8 | H | 5.03 |
| 9 | 1 | 15.53 |
| 10 | 2 | 5.39 |
| 11 | 3 | 5.45 |
| 12 | 4 | 5.45 |
| 13 | 5 | 5.45 |
| 14 | 6 | 5.45 |
| 15 | 7 | 5.39 |
| 16 | 8 | 5.39 |

**2.6 In Plane Stability of Walls**

In-plane failure in walls occurs when wall tends to bends in its own plane when subjected to excessive horizontal and vertical forces, applied along its plane, producing in-plane bending moments. To ensure in-plane stability of wall panels, various checks of stress are performed as under.

### 2.6.1 Check for Compressive Stress

Compressive strength of wall depends on its constituents i.e. units and mortar. The ultimate strength of confined masonry walls subjected to vertical load can be determined with the following expression:

$P_{comp} = k_s \text{ X } f_m$

where $fm = 0.422 \text{ x } f_b^{0.69} \text{ x } f_{mo}^{0.252}$

$f_b$ = compressive strength of hollow brick, i.e 5

$f_{mo}$ = compressive strength of masonry, i.e 3

implies, $f_m = 1.6898$

$k_s$ is stress reduction factor based on slenderness ratio and eccentricity. Wall panel is considered to be safe in compression if following criteria is fulfilled:

$P_{comp} = 2.6\sigma_{dl}$

$\sigma_{dl}$ is stress generated due to vertical loading (dead + live) on the wall panel. Self weight of wall panel and load from the slab (dead + live) shall be considered while calculating $\sigma_{dl}$.

For panel A check compressive strength is computed.

Length of wall panel = 3210 mm

Breadth of wall panel = 150 mm

Height of wall panel = 2210 mm

Slenderness Ratio, $h_w/t_w$ = 2210/150 = 14.733

The stress reduction factor as per Slenderness Ratio and Eccentricity is given by table 12.

Table 12: Stress Reduction Factor (k1) for Slenderness Ratio and Eccentricity

| **Slenderness Ratio** | **Eccentricity of Loading Divided by the Thickness of the Member** | | | | | |
|---|---|---|---|---|---|---|
| | 0 | 1/24 | 1/12 | 1/6 | 1/4 | 1/3 |
| 6 | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 |
| 8 | 0.95 | 0.15 | 0.94 | 0.93 | 0.92 | 0.91 |
| 10 | 0.89 | 0.88 | 0.87 | 0.85 | 0.83 | 0.81 |
| 12 | 0.84 | 0.83 | 0.81 | 0.78 | 0.75 | 0.72 |
| 14 | 0.78 | 0.76 | 0.74 | 0.70 | 0.66 | 0.66 |
| 16 | 0.73 | 0.71 | 0.68 | 0.63 | 0.58 | 0.53 |
| 18 | 0.67 | 0.64 | 0.61 | 0.55 | 0.49 | 0.43 |
| 20 | 0.62 | 0.59 | 0.55 | 0.48 | 0.41 | 0.34 |
| 22 | 0.56 | 0.52 | 0.48 | 0.40 | 0.32 | 0.24 |

| | | | | | | |
|---|---|---|---|---|---|---|
| 24 | 0.51 | 0.47 | 0.42 | 0.33 | 0.24 | - |
| 26 | 0.45 | 0.40 | 0.35 | 0.25 | - | - |
| 27 | 0.43 | 0.38 | 0.33 | 0.22 | - | - |

As per Table 12- $K_s$ and Eccentricity, take eccentricity = 0

That implies, $K_s = 0.761$

Therefore,

Psafe comp = 1.2859 N/mm2

$\sigma_{dl}$ = Compressive stress due to dead and live load

Self weight of panel = 17.026 KN

Load coming from wooden diaphragm

| | | | |
|---|---|---|---|
| = | 1.854738 | KN | (wooden plank) |
| = | 3.0375 | KN | (5 wooden beam) |
| Live | = | 18 | KN |
| Total | = | 22.89224 | KN |

$\sigma_{dl}$ = (17.206 + 4.6575 +18) /( 0.15 * 3)

= 88.7068 $KN/mm^2$

= 0.08871 $N/mm^2$

Panel is considered to be safe in compression if following criteria is fulfilled:

$P_{comp} = 2.6\sigma_{dl}$

= 0.23064 $N/mm^2$

Psafe comp > 0.23064 $N/mm^2$

Hence wall panel is safe in compression.

**2.6.1.1 Compressive Strength Check from Wall Density Consideration**

In X direction

$W_d$ = 5.35 %

w = 0.265 + 2 + 191.77 / (6.44 X 6.44)

= 4.67 $KN/m^2$

Now, $W_d = (f_g\ w\ n_s)/ P_{comp}$

Therefore, $P_{comp}$(minimum required)

= (2.33 X 0.00467 X 1 )/ 0.0535

= 0.2033 $N/mm^2$ < 1.28594 $N/mm^2$

Hence safe in compression.

In Y direction

$W_d$ = 6.08 %

w = 0.265 + 2 + 191.77 / (6.44 X 6.44)

= 4.67 $KN/m^2$

Now, $W_d$ = $(f_g\ w\ n_s)$/ Pcomp

Therefore, Pcomp(minimum required)

= (2.33 X 0.00467 X 1 )/ 0.0608

= 0.1789 $N/mm^2$ < 1.28594 $N/mm^2$

Hence safe in compression.

### 2.6.2 Check for Tensile Stress

The masonry walls shall be checked for net tensile stress (at)as per following expression, as against the permissible tensile stress. To calculate tensile stress, total stress due to vertical load is subtracted by moment on the panel divided by its sectional modulus. The permissible tensile strength of masontry is considered to be 0.25 N/mm2

For wall panel A ,

$$\sigma_t = M/S - \sigma_{dl}$$

$$M = (P_i \times h_w) / 2 \quad \text{and}$$

$$S = (t_w \times l_w^2) / 2$$

Therefore,

$$\sigma_t = ((4940 \text{ X}2210)/2)/((150 \text{ X } 32102)/6) - 0.08871$$

$$= -0.0675 < 0.25 \quad N/mm^2$$

(Safe in tension)

### 2.6.3 Check for Shear Stress

The permissible shear stress (TU) for the confined masonry walls is given as per following expression, subjected to a maximum of 0.5 N/mm2.

$\tau_u = 0.1 + \sigma_d/6$

While the actual shear stress of CM wall panels are calculated at sill level by the following expression:

Actual shear stress acting on wall = Pi / Aw

For Wall Panel A,

Permissible shear strenth for the confined masonry wall is given as:

ţ$_u$ = $0.1 + \sigma_d/6$

$\sigma_d$ is the coompressive stress due to dead load

For wall panel

Self weight of panel = 17.026 KN

Load coming from wooden diaphragm

| | | | |
|---|---|---|---|
| = | 1.62 | KN | (wooden plank) |
| = | 3.0375 | KN | (5 wooden beam) |
| Live | = | 18 | KN |
| Total | = | 22.6575 | KN |
| Overall | = | 39.6833 | KN |
| σd | = | 39.683/(0.15 X 3.21) | |
| | = | 82.415 | $KN/mm^2$ |
| | = | 0.082 | $N/mm^2$ |

Therefore,

| | | | |
|---|---|---|---|
| ţu | = | 0.1+0.082/6 | |
| | = | 0.11367 | $N/mm^2$ |

Actual shear stress acting on wall = $P_i / A_w$

| | | |
|---|---|---|
| = | (4.94 X 1000) / (150 X 3210) | |
| = | 0.0102 | $N/mm^2$ |

ţ$_u$ > 0.0102 $N/mm^2$ .

Hence, wall panel is safe in shear.

### 2.6.3.1 Shear strength check from wall density consideration

In X direction

$W_d$ = 5.35 %

w = 0.265 + 2 + 191.77 / (6.44 X 6.44)

= 4.67 $KN/m^2$

Now, $W_d$ = $(A_h\ f_s\ \ w\ n_s)/\ \tau_u$

= (0.18 X 1.3 x0.00467 x1)/0.0535

Therefore, $T_u$ min

= 0.02 $N/mm^2$ < 0.114 $N/mm^2$

Hence safe in shear.

In Y direction

$W_d$ = 6.08 %

w = 0.265 + 2 + 191.77 / (6.44 X 6.44)

= 4.67 $KN/m^2$

Now, $W_d$ = $(A_h\ f_s\ \ w\ n_s)/\ \tau_u$

= (0.18 X 1.3 x0.00467 x1)/0.0608

Therefore, $T_u$ min

= 0.017 $N/mm^2$ < 0.114 $N/mm^2$

Hence safe in shear.

### 2.7 Out- of- Plane Stability of Walls

#### 2.7.1 Check for Overturning

The dynamic stability of masonry walls under out-of-plane forces depends on its slenderness ratio, and is also a function of the floor response. It is well known fact that stiff masonry walls amplify ground accelerations, leading to larger motions of the walls. The amplifications depend on the site soil conditions and on the aspect ratio of wall.

The total lateral force ($P_i$) causes overturning moment ($M_o$) in the walls, which is equal to $P_i h_w/2$ at the bottom of the wall, whereas free standing walls shall be checked against overturning under the action of design seismic coefficient allowing the factor of safety of 1.5.

Overturning Moment is given by:

$M_o = P \times h_w/2$

Resisting Moment ($M_r$)

Gravity load of wall panel ($T_L$) = Self Weight of Panel+Load coming from slab (dead+live)

For safety in Overturning

$M_r/M_o > 1.5$

For Panel A,

$M_o = P \times h_w/2 = 9.70$ KN-m

Self Weight 13.24272 KN

Load coming from slab (dead+live) 28.799036 KN

Total = 42.04 KN

From Yield line theory load coming from slab 140.42 KN-m

For safety in Overturning $M_r/M_o>1.5$

= 14.47 Safe in Overturning

Simillarly, for all panels total seismic force acting are shown in table below:

Table 13: Check for Over Turning

| Sno. | Wall Panel | Overturning Moment (KN-m) | Resisting Moment (KN-m) | Safe/Unsafe |
|---|---|---|---|---|
| 1 | A | 9.70 | 140.42 | Safe |
| 2 | B | 0.46 | 107.26 | Safe |
| 3 | C | 9.89 | 140.42 | Safe |
| 4 | D | 29.72 | 149.33 | Safe |
| 5 | E | 29.72 | 149.33 | Safe |
| 6 | F | 9.89 | 154.85 | Safe |
| 7 | G | 0.46 | 107.26 | Safe |
| 8 | H | 9.89 | 154.85 | Safe |
| 9 | 1 | 30.52 | 149.33 | Safe |
| 10 | 2 | 10.59 | 140.42 | Safe |
| 11 | 3 | 10.72 | 126.30 | Safe |
| 12 | 4 | 10.72 | 126.30 | Safe |
| 13 | 5 | 10.72 | 126.30 | Safe |
| 14 | 6 | 10.72 | 126.30 | Safe |
| 15 | 7 | 10.59 | 140.42 | Safe |
| 16 | 8 | 10.59 | 140.42 | Safe |

### 2.7.2 Check for Out-of-plane Stability

Out-of-plane failure in walls occurs when lateral load is acting perpendicular to the surface of wall e.g. lateral load on the wall is acting perpendicular to wall surface.

The lateral seismic load acting of CM wall panels can be calculated as:

Seismic load per unit area of the panel, $F = A_h \rho_m t_w$

Ultimate Bending Moment, $M_u = Fh^2/8$

Bending Stress ($\sigma b$) = $M_u/S$

To check wall for out-of-plane action, actual stress should be less than the tensile stress

For Wall Panel A,

| | | | |
|---|---|---|---|
| Seismic load per unit area of the panel, F | | = | $A_h \ \rho_m \ t_w$ |
| | | = | 0.19 X 16 X 0.15 |
| | | = | 0.456 |
| Ultimate Bending Moment, Mu | | = | $Fh^2/8$ |
| | | = | 0.456 X 2.21 X 2.21/8 |
| | | = | 0.278394 |
| Bending Stress, | $\sigma_b$ | = | $M_u/S$ |
| | | = | (0.278 X 1000)/(1502/6) |
| | | = | 1.110519 |
| Actual Stress, | $\sigma_p$ | = | $\sigma_b$-$\sigma_{dl}$ |
| | | = | 1.1105- 0.08871 |
| | | = | 1.02179 |

(Actual Stress is positive so wall is safe in out of plane stability)

**2.8 Design of Bond Beam**

The basic requirements for design of bond beam in confined masonry building are:

- The bond beam shall be located above the masonry walls and at lintel/sill level, resting over brickwork.
- The minimum width to depth ratio shall be 0.3.
- The minimum depth of bond beam shall not be less than 200 mm.
- The width of bond beam shall be same to that of wall thickness.
- Minimum 4 number of 10 mm diameter deformed bars shall be provided in tie beams.
- In a comer joint of tie beam and tie column, anchorage length of Ld + 10db shall be provided in top as well as bottom bars of tie beam as shown in the whereas in a middle joint, both top and bottom bars of the tie beam shall be continuous through the column.
- Wherever longitudinal bars are spliced, stirrups shall be provided over the entire splice length, at spacing not exceeding 150 mm. The lap length shall not be less than $L_d$ in tension.

- Lap splices are not to be provided within a joint, within a distance of 2deff from joint face and within a quarter length of the member where flexural yielding occurs due to lateral loads. Not more than 50 percent of the bars shall be spliced at one section.

By following above requirements, the bond beams are designed considering lateral load acting on the wall panel.

Bond beam had been designed for total lateral load acting on the wall

$A_{st}$ = (4.94 X1000)/415

$A_{st}$ = 11.9 $mm^2$

$A_{st\ min}$ = $(0.85\ BD)/f_y$

$A_{st\ min}$ = (0.85 X 150 X 150) /415

$A_{st\ min}$ = 46.08 $mm^2$

Therefore, provide 4 bars of 10 mm dia (314 $mm^2$)

Nominal Shear Stress

$\tau_v$ = $V_u$ / (BD)

= 4940 / (150 X150)

= 0.219 $N/mm^2$

$P_t$ = (4 X 3.14 X 10 x 10 /4) X 100/(150 X150)

= 1.39 %

$\tau_c$ = 0.51 $N/mm^2$ (as per IS 456)

Providing 6 mm dia bars for stirrups,

$A_{sv}$ = 2 X 3.14 X 32

= 56.52 $mm^2$

Spacing between stirrups shall be provided, minimum among the following:

$S_v$ = $(0.87 \times f_y \times A_{sv}) / (0.4 \times B)$

= (0.87 X 415 X 56.52) / (0.4 X 150)

= 340 mm

or

$S_v$ = 0.75 D

= 0.75 X 150

= 112.5 mm

or

= 300 mm

Therefore, provide 6 mm stirrups at 100 mm spacing.

However as per IS 13920:2016, the spacing of stirrups over a length of $2d_{eff}$ at either end of a beam shall not exceed deef/4 or 8 times the diameter of the smallest longitudinal bar or not less than 100 mm, whichever is minimum. Therefore, upto a length of 350 ($2d_{eff}$) from either end of the beam, spacing of stirrups shall be 100 mm and at rest of the beam legnth the stirrups shall be spaced at 150 c/c.

**2.9 Design of Tie Column**

The basic requirements for design of tie column in confined masonry building are:

- Tie columns should be located at all corners and wall intersections of structural walls.
- Minimum size of column should be same as that of wall thickness.
- Reinforcement detailing should confirm to ductile detailing provisions.
- 1 Omm dia bars shall be considered as minimum diameter of longitudinal bar
- At mid height of tie-column, spacing of stirrups shall not exceed half the least lateral dimension of the column.

- Special confining reinforcement shall be provided at either ends of the column over a maximum length of larger lateral dimension of the member, I*J6,* and 450 mm.
- The spacing of stirrups in special confining reinforcement shall not exceed 1/4 of minimum member dimension. Butin no case it shall be less than 75 mm or more than 100 mm.
- Around the openings i.e. window, a nominal reinforcement shall be provided.
- Cross ties or a pair of over lapping stirrups shall be provided wherever parallel legs of stirrups are spaced at a distance of more than 300 mm c/c.

Design of Tie Column

Let tie column size be 150 mm X 150 mm

Area of steel in tie columns is calculated by the expression,

$$A_{st} = (1 + 0.25k)\ P_i\ h_w /\ l_w\ f_y$$

For tie column in wall panel

$$A_{st} = (1 + 0.25 \text{ X } 0) \text{ X } 4140 \text{ X } 2120 /( 3210 \text{ X } 415)$$

$$= 37.78 \ \text{mm}^2$$

Therefore, provide 4 bars of 10 mm dia (314 $mm^2$)

As per IS 13920:1993, the spacing of the stirrups shall not exceed half the least lateral dimension of the column. Also the length of special confining reinforcement shall not be less than larger lateral dimension of the member at the section where yielding may occur. 1/6th of the clear span of the member and 450mm.

Therefore, length of special confining reinforcement is larger of 150mm, 1/6 X 3000 =500mm and 450mm.

The spacing of stirrupes in special confinement reinforcement shall not exceed 1/4 of the minimum member dimension but need not be less than mm or more than 100 mm.

Therefore provide stirrupes at a spacing of 75 mm at 500 mm from either end. And provide stirrups at a spacing of 160mm at midspan of the column.

**2.10 Foundation Design**

The foundation system applicable for CM buildings shall be governed by the local site conditions. However, some of the important points to be considered are:

- The foundations shall be laid on hard and well compacted strata.
- The selection of type of foundation shall be as per local site condition, practices and may be designed accordingly. A typical RC strip footing.
- Foundation for CM buildings on clayey soil, under reamed cast-in-situ pile foundations can be adopted.

For the design of foundation a foundation section is checked against soil bearing capacity and shear strength in both directions.

Let, Length of footing (L) = 600 mm = 0.6m

Width of footing (B) = 600 mm = 0.6m

Height of footing (h) = 800 mm = 0.8m

Total Load of building per m 8.97kN/m

Partial Safety factor for dead and imposed load

Factor of safety below ground level $\gamma f = 1.4$

Total Load 12.56kN/m

Weight of foundation 10.18kN/m

Total 22.74kN/m

Average Soil bearing pressure considered = 100 kN/m$^2$

Stiff Mass foundati

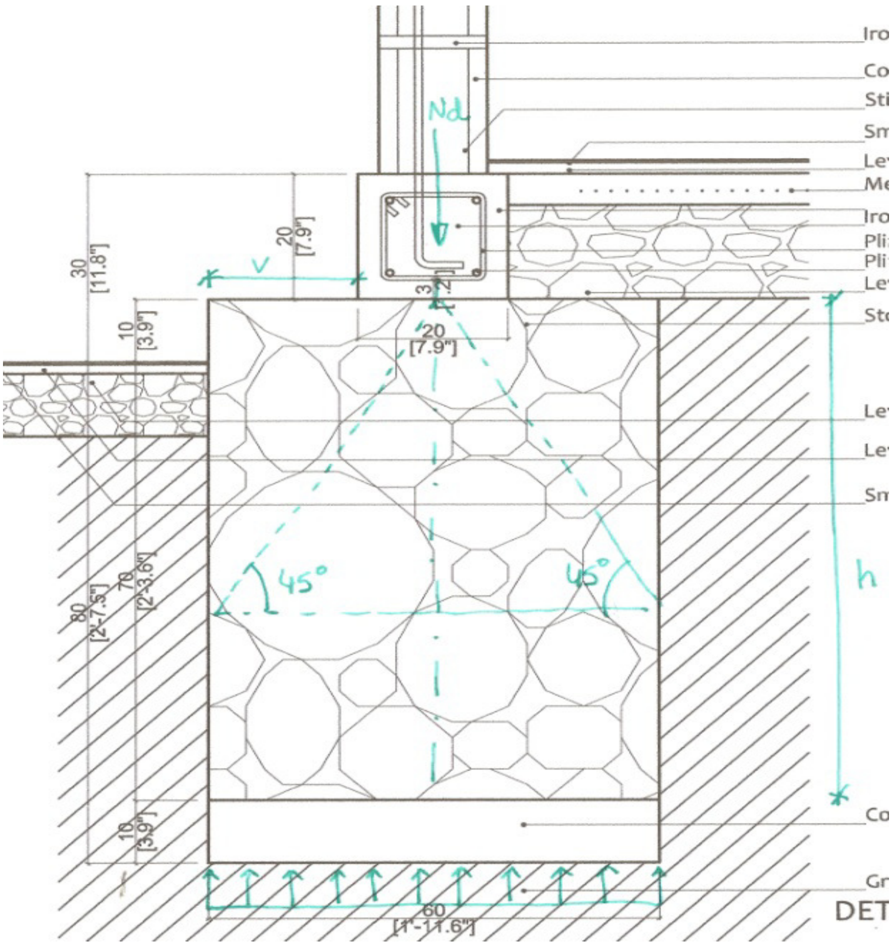
Geometry requierement:

v/h <0.5

v = 0.2m

h= 0.8m

v/h = 0.25 < 0.5



Actual Soil pressure per m:

h > b; so:

$$\sigma = \frac{N_d}{1m \times b} = \frac{22.74kN}{1m \times 0.60m} = 37.9\ kN/m^2 < 100kN/m^2$$

Check for one way shear:

Maximum one way shear occurs at, h distance away from the face of the column.

We have,

l= (B/2 –L/2)

where, B= 600mm

b= 200mm

$V_u = q_u$ X Shaded Area

But, $q_u = 37.9\ KN/m^2 = 37.9 \times 10\text{-}3\ N/mm^2$

$V_u = 37.9 \times 10\text{-}3 \times 600 \times (l – d)$

Assuming, Pt = 0.2%

$T_u = 0.32\ N/mm^2$

$0.32 \times 600 \times d = 37.9 \times 10\text{-}3 \times 600 \times (200\text{-}d)$

d = 221.1 mm (< 800mm)

Hence, the footing is safe in one way shear.

Check for two way shear:

Critical section is at d/2 distance from the face of column.

i.e. 800/2 = 400 mm

which lies outside of the actual footing section.

Hence, the footing is safe in two way shear.

## 2.11 Floor Design

The floor is made of wooden joist that rest on the walls, leaning on ring beams. The joist are joined to the concrete ring beam by a metal angle which are screw in the bolts whose head are placed in the ring beams. The bolts heads are fixed with RCC by pacing them in required position before concreting. The other end of the bolts are screwed with angle fixed in them.

### 2.11.1 Design of wooden frame

Density of timber = 900 $kg/m^3$

| | | | | | | |
|---|---|---|---|---|---|---|
| Dimension, | L | = | 3.6 | m | | |
| | B | = | 3.6 | m | | |
| Wall thickness | | = | 0.15 | m | | |
| Depth of wooden plank | | = | 0.015 | m | | |
| Weight of wooden plank | | = | 900 X 3.6 X 3.6 X 0.026 | | | |
| | | = | 174.96 | kg | = | 1.7496 |
| Live load on the diaphragm | | = | 1.5 | $KN/m^2$ | | |
| Total live load | | = | 1.5 X 3.6 X 3.6 | | = | 19.44 |
| Total no. of beam in one room | | = | 4 | | | |
| Load taken by one beam | | = | (1.7496+19.44)/4 | | = | 5.2974 |
| Dimension of beam section, | | | | | | |
| | b | = | 3 | inch | = | 0.75 |
| | d | = | 5.5 | inch | = | 137.5 |
| Self load of beam | | = | 900 X 3.6 X 0.1 X0.135 | | = | 43.74 |
| | | | | | = | 0.4374 |
| Total load on one beam | | = | (5.29 + 0.4374) | | = | 5.7348 |
| Distributed load on the beam | | = | 1.575 | KN | | |
| Effective length | | = | 3.6 + 0.15/2 + 0.15/2 | | = | 3.75 |

Now, bending moment to be resisted :

Maximum Bending Moment = $wle^2/8$

= 1.575 X 3.75 X 3.75 / 8

= 2.76 KN-m

Now, from table 1 of code IS 883 : 1994

For Deodar wood,

Permissible bending stress for inside location

$f_b$ = 10.2 $N/mm^2$

We have,

Moment (M) = $f_b$ X Z

or, Sec. Modulus (Z) = M / $f_b$

= (2.76 X 106) / 10.2

= 270588.2 ……. (i)

But, $Z = bd^2/6$ …….. (ii)

If, b is provided of 3 inches,

i.e. $b = 0.075$ m ……. (iii)

From (i) , (ii) , (iii)

We get, $d = 130.92$ mm

Now, we have, from clause 7.5.5 of code IS 883 :1994

The minimum width of the beam,

(bmin) > 50mm or l/50 whichever is greater

(bmin) > 50 mm or 72mm whichever greater

(bmin) > 72 mm

Hence, provide $b = 75$ mm

And $d = 137.5$ mm

Check for shear :

Maximum horizontal shear < Permissible Shear

From table 1 of code IS 883 : 1994

For Deodar wood

Permissible horizontal shear stress

T permissible = 0.7 $N/mm^2$

and, from clause 7.5.7.1 of code IS 883: 1994 for rectangular beam,

$H = 3 V /(2 bd)$

where, $V = Wl/2 X (1 - 2d/l)$

$= 1.575 X 3.75 / 2 X (1 - 2 X 0.173 / 3.75)$

$= 2.63$ KN

So, $H = 0.405$ $N/mm^2$

This is less than T permissible. Hence OK

Check for bearing

Bearing reaction = W le / 2 = 1.575 X 3.75 /2

= 2.953 KN

Bearing stress = 2.953 X 1000 / (75 X 137.5)

= 0.302 KN

Now, again from table 1 of IS 883 : 1994.

For Deodar wood,

Permissible bearing stress for compression

Perpendicular to grain and inside location,

$f_{cn}$ = 2.7 $N/mm^2$

Therefore, Bearing stress is under permissible bearing stress. Hence OK

Check for deflection :

From clause 7.5.9.2 of code IS 883 : 1994,

Deflection = $5\ Wl^4 /(384\ EI)$

Therefore Deflection = 14.09 mm

We have, again, from clause 7.5.9.1 of code IS 883 : 1994,

Permissible deflection = 1 / 240 = 15.625 mm

Deflection calculated < Deflection permissible. Hence OK

Hence, provide beam of dimension 3" X 5.5 ".

**2.11.2 Connection design with metal angle strip**

| | | | | | |
|---|---|---|---|---|---|
| AXIAL TENSILE FORCE (ASD) | | T = | 5.3 | k | |
| NUMBER OF BOLTS | | n = | 3 | | |
| BOLT DIAMETER | | f = | 0.75 | | |
| BOLT SPACING | | S = | 3 | in | |
| END DISTANCE OF WOOD | | $E_n$ = | 4 | in | |
| END DISTANCE OF STEEL | | $E_{n,s}$ = | 1.5 | in | |
| LUMBER TYPE | | | | | |
| 0 Douglas Fir-Larch, G=0.5 | | | | | |
| ( 0=Douglas Fir-Larch, 1=Douglas Fir-Larch(N), 2=Hem-Fir(N), 3=Hem-Fir, 4=Spruce-Pine-Fir) | | | | | |
| LUMBER SIZE | 2 | thk. x | 6 | width | |
| STRAP SIZE | 5 | width x | 0.25 | thk. | |
| LOAD DURATION FACTOR ( Tab 2.3.2, NDS 2015) | | $C_\Delta$= | 1.6 | | |
| WET SERVICE FACTOR ( Tab 10.3.3, NDS 2015) | | $C_M$ = | 1 | | |

TEMPERATURE FACTOR ( Tab 10.3.4, NDS 2015) $C_t$ = 1

THE CONNECTION DESIGN IS ADEQUATE.

ANALYSIS

CHECK STEEL STRAP CAPACITIES (AISC 360-10, ASD)

$A_g$ = 1.25 $in^2$, yielding criterion

$F_y$ = 36 ksi

$T_{allow}$ = $0.6\ F_y\ A_g$ = 27 k > T

[Satisfactory] (0.6 from $1/\Omega_t$, Typ.)

$A_n$ = 1.03125 $in^2$, fracture criterion

$F_u$ = 58 ksi

$T_{allow}$ = $0.5\ F_u\ A_n$ = 29.90625 k > T

[Satisfactory]

$A_v$ = 1.328125 $in^2$, block shear

Tallow = $0.3\ F_u\ A_v + 0.5\ F_u\ (0.5\ A_n)$ = 38.0625 k > T

[Satisfactory]

rmin = $t / (12)^{0.5}$ = 0.072169 in

L = Max ($E_n$ , S ) = 4 in

L / rmin = 55.42563 < 300

[Satisfactory] (AISC 360-10 D1)

CHECK EDGE, END, & SPACING DISTANCE REQUIREMENTS (NDS 2015, Table 12.5.1A, Table 12.5.1B, & Table 12.5.1C)

| | | | | |
|---|---|---|---|---|
| $E_g$ = | 2.75 in | > | 1.5 D | [Satisfactory] |
| $E_n$ = | 4 in | > | 3.5 D | [Satisfactory] |
| S = | 3 in | > | 3 D | [Satisfactory] |

CHECK WOOD CAPACITY

$C_\Delta$ = Min ($C_{\Delta 1}$ , $C_{\Delta 2}$ , $C_{\Delta 3}$) = 0.761905 , (geometry factor, NDS 2015, 12.5.1)

where

$C_{\Delta 1}$ = (actual end distance) / (min end distance for full design value) = En / 7D = 0.761905

$C_{\Delta 2}$ = (actual shear area) / (min shear area for full design value) = 1

$C_{\Delta 3}$ = (actual spacing) / (min spacing for full design value) = S / 4D = 1

$$C_g = \left[ \frac{m(1-m^{2n})}{n\left[(1+R_{EA}m^n)(1+m)-1+m^{2n}\right]} \right]\left[\frac{1+R_{EA}}{1-m}\right] = 0.987564$$

, (group action factor, NDS 2015, 10.3.6)

where n = 3

REA = Min [$(E_sA_s/E_mA_m)$ , $(E_mA_m/E_sA_s)$] = 0.616

$E_sA_s$ = 37500000 lbs, (NDS 2015, Table 10.3.6C)

g = 180000 $D^{1.5}$ = 116913.4

tm = 3 in

u = 1+g S/2 [1 / $E_mA_m$ + 1 / EsAs] = 1.012268

EmAm = 23100000 lbs, (NDS 2015, Table 10.3.6C)

m = u - $(u2 - 1)^{0.5}$ = 0.855147

$Z'_{II}$ = n $Z_{II}$ $C_\Delta$ $C_M$ $C_t$ $C_g$ $C_\Delta$= 5.309144 kips > T [Satisfactory]

where $Z_{II}$ = 1470 lbs / bolt, (interpolated from NDS 2015)

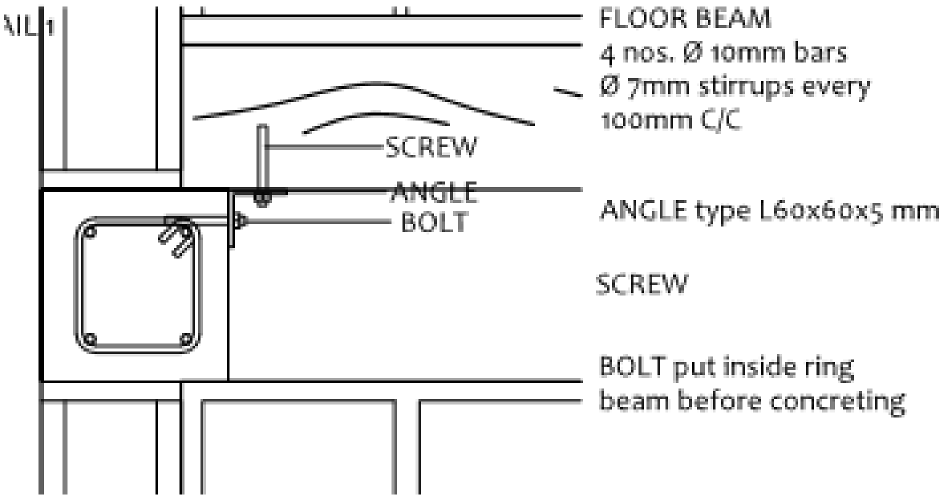



Figure 8: Connection detail of wooden floor beam and RCC beam

## 2.12 Roof Design

The roof system is designed out of tubular profile metal pipes of required diameter as per analysis. The pipe rests on the concrete walls, leaning on the upper ring beam. The roof is finished with CGI sheets, allowing attachment of false ceiling and bamboo insulation. The roof is divided in two parts as:

1) Roof Part
2) Cantilever Part

The Roof Parts and Cantilever Parts are divide in two parts in its structural load basis, since roof part rest on the building walls whereas the cantilever parts lies slightly below the roof walls resting on front wall and load is transferred by vertical poles directly to the ground and are designed using structure design software Etabs 2016 v2.

1) Roof Part

The properties of material used in roof are:

Pipe material = Fe250

Mass per unit volume = 7849.049 kg/m3

Modulus of Elasticity (E) = 210000 Mpa

Nominal Bore = 50mm

Outside Diameter = 60.3 mm

Type = Heavy

Thickness = 4.5 mm

Unit weight = 6.19 kg/m

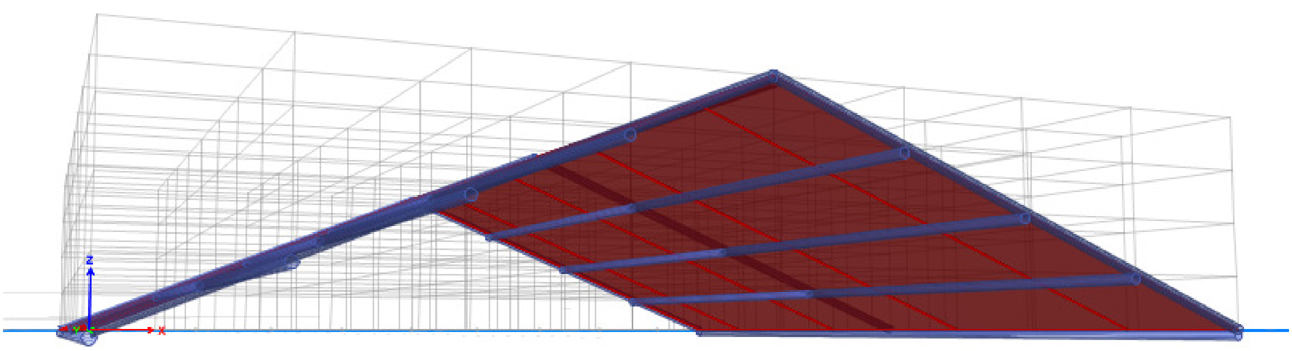
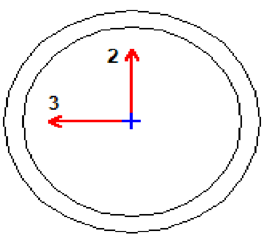



Figure 9: 3D View of Roof Cover

**Pipe design**



Element Details (Part 1 of 2)

| Level | Element | Unique Name | Location (mm) | Combo | Design Type | Element Type | Section |
|---|---|---|---|---|---|---|---|
| Story5 | B1 | 21 | 4025 | DStlS26 | Beam | Special Moment Frame | Pipe |

Element Details (Part 2 of 2)

| Classification | Rolled |
|---|---|
| Class 1 | No |

Seismic Parameters

| MultiResponse | P-Δ Done? | Ignore Seismic Code? | Ignore Special EQ Load? | D/P Plug Welded? |
|---|---|---|---|---|
| Envelopes | No | No | No | Yes |

Design Code Parameters

| γM0 | γM1 | An /Ag | LLRF | PLLF | Stress ratio Limit |
|---|---|---|---|---|---|
| 1.1 | 1.25 | 1 | 1 | 0.75 | 0.95 |

Section Properties

| A ($cm^2$) | Izz ($cm^4$) | rzz (mm) | Ze,zz ($cm^3$) | Av,z ($cm^2$) | Zp,zz ($cm^3$) | Iyz ($cm^4$) | It ($cm^4$) |
|---|---|---|---|---|---|---|---|
| 7.9 | 30.9 | 19.8 | 10.2 | 5 | 14 | 0 | 61.8 |

| J (cm4) | Iyy ($cm^4$) | ryy (mm) | Ze,yy ($cm^3$) | Av,y ($cm^2$) | Zp,yy ($cm^3$) | Iw ($cm^4$) | h (mm) |
|---|---|---|---|---|---|---|---|
| 61.8 | 30.9 | 19.8 | 10.2 | 5 | 14 | | 60.3 |

Material Properties

| J ($cm^4$) | Iyy ($cm^4$) | ryy (mm) |
|---|---|---|
| 61.8 | 30.9 | 19.8 |
| E (MPa) | fy (MPa) | fu (MPa) |
| 210000 | 250 | 410 |

Stress Check Forces and Moments

| Location (mm) | N (kN) | Mzz (kN-m) | Myy (kN-m) | Vy (kN) | Vz (kN) | To (kN-m) |
|---|---|---|---|---|---|---|
| 4025 | 0 | -0.0197 | 0 | -0.0733 | 0 | 0 |

PMM Demand/Capacity (D/C) Ratio 9.3.1.1(az)

| D/C Ratio = Mz / Mndz |
|---|
| 0.007 = 0.007 |

Basic Factors

| Buckling Mode | K Factor | L Factor | L Length (mm) | KL/r |
|---|---|---|---|---|
| Major (z-z) | 1 | 0.5 | 3220 | 162.689 |
| Major Braced | 1 | 0.5 | 3220 | 162.689 |
| Minor (y-y) | 1 | 0.5 | 3220 | 162.689 |
| Minor Braced | 1 | 0.5 | 3220 | 162.689 |
| LTB | 1 | 0.5 | 3220 | 162.689 |

Axial Force Design

| | N Force kN | Td Capacity kN | Nd Capacity kN | Pdy Capacity kN | Pz Capacity kN | Pd Capacity kN |
|---|---|---|---|---|---|---|
| Axial | 0 | 179.285 | 179.285 | 45.7708 | 45.7708 | 45.7708 |

| Tdg kN | Tdn kN | Ncr,T kN | Ncr,TF kN | An /Ag Unitless | N /Nd Unitless |
|---|---|---|---|---|---|
| 179.285 | 232.8697 | 63715.1239 | 61.7729 | 1 | 0 |

Design Parameters for Axial Design

| | Curve | α | fcc (MPa) | λ | Φ | χ | fcd (MPa) |
|---|---|---|---|---|---|---|---|
| Major (z-z) | b | 0.34 | 78.31 | 1.787 | 2.366 | 0.255 | 58.02 |
| MajorB (z-z) | b | 0.34 | 78.31 | 1.787 | 2.366 | 0.255 | 58.02 |
| Minor (y-y) | b | 0.34 | 78.31 | 1.787 | 2.366 | 0.255 | 58.02 |
| MinorB (y-y) | b | 0.34 | 78.31 | 1.787 | 2.366 | 0.255 | 58.02 |
| Torsional TF | b | 0.34 | 78.31 | 1.787 | 2.366 | 0.255 | 58.02 |

Moment Designs

| | M Moment kN-m | Mspan Moment kN-m | Md(yield) Capacity kN-m | Mdv Capacity kN-m | Mnd Capacity kN-m | Md(LTB) Capacity kN-m |
|---|---|---|---|---|---|---|
| Major (z-z) | -0.0197 | -0.0197 | 2.7953 | 2.7953 | 2.7953 | 3.1913 |
| Minor (y-y) | 0 | 0 | 2.7953 | 2.7953 | 2.7953 | |

| | Curve | αLT | λLT | ΦLT | χLT | C1 | Mcr (kN-m) |
|---|---|---|---|---|---|---|---|
| LTB | c | 0.49 | 0.179 | 0.511 | 1 | 1.724 | 95.7423 |

| | Cmy | Cmz | CmLT | kz | ky | KLT | My / Mdy | Mz / Mdz | α1 | α2 |
|---|---|---|---|---|---|---|---|---|---|---|
| Factors | 1 | 0.85 | 0.85 | 1 | 1 | 1 | 0 | -0.007 | 2 | 2 |

Shear Design

| | V Force (kN) | Vd Capacity (kN) | To Capacity (kN-m) | Stress Ratio | Status Check |
|---|---|---|---|---|---|
| Major (y) | 0.0733 | 65.8966 | 0 | 0.001 | OK |
| Minor (z) | 0 | 65.8966 | 0 | 0 | OK |

Shear Design

| | Vp (kN) | kv (Unitless) | ΛW (Unitless) | Tb (MPa) |
|---|---|---|---|---|
| Reduction | 65.8966 | 0 | 0 | 1 |

End Reaction Major Shear Forces

| Left End Reaction (kN) | Load Combo | Right End Reaction (kN) | Load Combo |
|---|---|---|---|
| 0.0367 | DStlS30 | 0.0367 | DStlS30 |

**Deflection Check**

Element Details

| Level | Element | Unique Name | Location (mm) | Combo | Design Type | Element Type | Section | Rolled |
|---|---|---|---|---|---|---|---|---|
| Story2 | B25 | 28 | 0 | DStlD2 | Beam | Special Moment Frame | Pipe1 | No |

DEFLECTION DESIGN (Combo  DStlD2)

| Type | Consider | Deflection mm | Limit mm | Ratio | Status |
|---|---|---|---|---|---|
| Dead Load | Yes | 0.1 | 26.8 | 0.003 | OK |
| Super DL + Live Load | Yes | 1.476E-02 | 26.8 | 0.001 | OK |
| Live Load | Yes | 1.476E-02 | 8.9 | 0.002 | OK |
| Total Load | Yes | 0.1 | 13.4 | 0.007 | OK |
| Total - Camber | Yes | 0.1 | 13.4 | 0.007 | OK |

2) Cantilever Part

The properties of material used in roof are:

Pipe material = Fe250

Mass per unit volume = 7849.049 kg/m3

Modulus of Elasticity (E) = 210000 Mpa

a) Vertical Post

Nominal Bore = 100 mm

Outside Diameter = 114.3 mm

Type = Heavy

Thickness = 5.4 mm

Unit weight = 14.5 kg/m

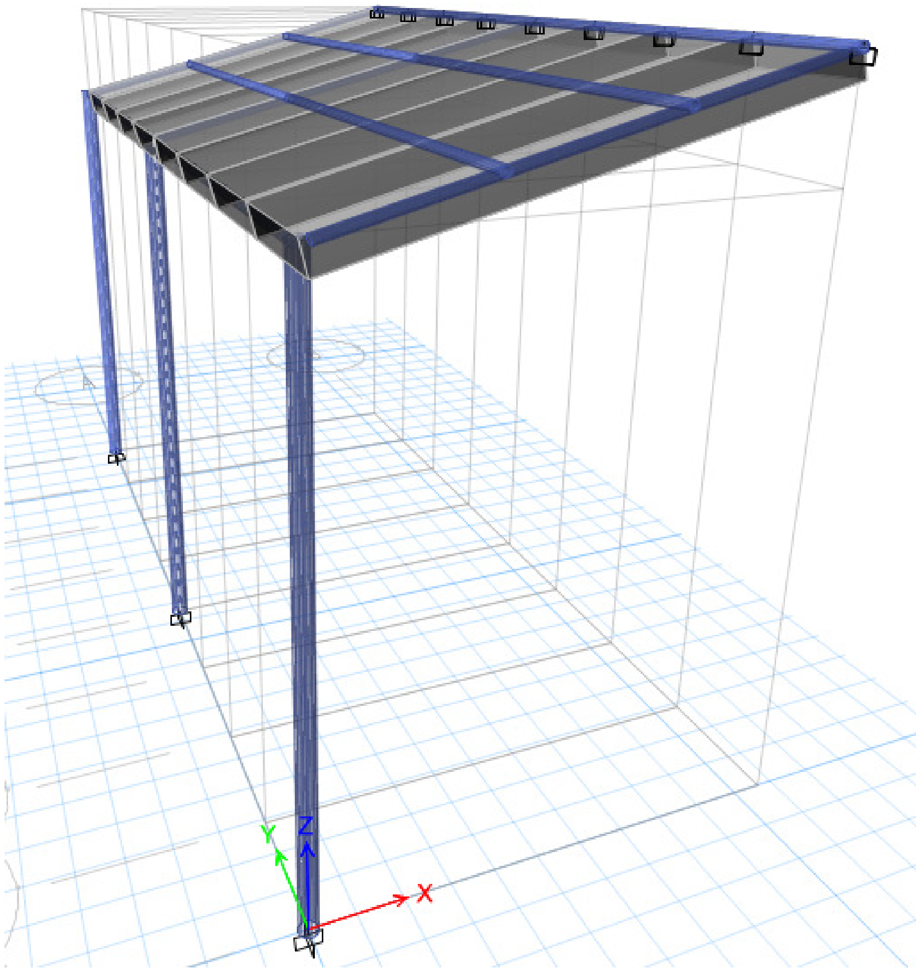
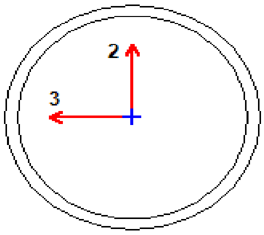



Figure 10: 3D View of Cantilever Cover

## Pipe Design




Element Details (Part 1 of 2)

| Level | Element | Unique Name | Location (mm) | Combo | Design Type | Element Type | Section |
|---|---|---|---|---|---|---|---|
| Story1 | C3 | 3 | 0 | DStlS8 | Column | Special Moment Frame | Pipe 110 |

Element Details (Part 2 of 2)

| | |
|---|---|
| Classification | Rolled |

| Classification | Rolled |
|---|---|
| Class 1 | No |

Design Code Parameters

| ɣM0 | ɣM1 | An /Ag | LLRF | PLLF | Stress ratio Limit |
|---|---|---|---|---|---|
| 1.1 | 1.25 | 1 | 1 | 0.75 | 0.95 |

Section Properties

| A (cm²) | Izz ($cm^4$) | rzz (mm) | Ze,zz (cm³) | Av,z (cm²) | Zp,zz (cm³) | Iyz ($cm^4$) | It ($cm^4$) |
|---|---|---|---|---|---|---|---|
| 18.5 | 274.5 | 38.5 | 48 | 11.8 | 64.1 | 0 | 549.1 |

| J ($cm^4$) | Iyy ($cm^4$) | ryy (mm) | Ze,yy (cm³) | Av,y (cm²) | Zp,yy (cm³) | Iw ($cm^4$) | h (mm) |
|---|---|---|---|---|---|---|---|
| 549.1 | 274.5 | 38.5 | 48 | 11.8 | 64.1 | | 114.3 |

Material Properties

| J ($cm^4$) | Iyy ($cm^4$) | ryy (mm) |
|---|---|---|
| 549.1 | 274.5 | 38.5 |
| E (MPa) | fy (MPa) | fu (MPa) |
| 210000 | 250 | 410 |

Stress Check Message - KL/r > 180 (IS 3.8, Table 3)

Stress Check Forces and Moments

| Location (mm) | N (kN) | Mzz (kN-m) | Myy (kN-m) | Vy (kN) | Vz (kN) | To (kN-m) |
|---|---|---|---|---|---|---|
| 0 | -47.6694 | 0.0799 | 0 | 0.0558 | 0 | 0 |

PMM Demand/Capacity (D/C) Ratio 9.3.2.2(b)

| |
|---|
| D/C Ratio = P / Pdz + 0.6 * Ky * Cmy * (My,span / Mdy; ) + Kz * Cmz * (Mz,span / Mdz; ) |
| 0.633 = 0.631 + 0 + 0.002 |

Basic Factors

| Buckling Mode | K Factor | L Factor | L Length (mm) | KL/r |
|---|---|---|---|---|
| Major (z-z) | 2.043 | 0.984 | 3739.7 | 198.194 |
| Major Braced | 0.78 | 0.984 | 3739.7 | 75.623 |
| Minor (y-y) | 1.831 | 0.984 | 3739.7 | 177.658 |
| Minor Braced | 0.754 | 0.984 | 3739.7 | 73.152 |
| LTB | 1.831 | 0.984 | 3739.7 | 177.658 |

Axial Force Design

| | N Force kN | Td Capacity kN | Nd Capacity kN | Pdy Capacity kN | Pz Capacity kN | Pd Capacity kN |
|---|---|---|---|---|---|---|
| Axial | -47.6694 | 419.8739 | 419.8739 | 91.8905 | 75.5489 | 75.5489 |

| Tdg kN | Tdn kN | Ncr,T kN | Ncr,TF kN | An /Ag Unitless | N /Nd Unitless |
|---|---|---|---|---|---|
| 419.8739 | 545.3658 | 149216.7096 | 97.4782 | 1 | 0.114 |

Design Parameters for Axial Design

| | Curve | α | fcc (MPa) | λ | Φ | χ | fcd (MPa) |
|---|---|---|---|---|---|---|---|
| Major (z-z) | b | 0.34 | 52.76 | 2.177 | 3.205 | 0.18 | 40.89 |
| MajorB (z-z) | b | 0.34 | 362.42 | 0.831 | 0.952 | 0.705 | 160.32 |
| Minor (y-y) | b | 0.34 | 65.67 | 1.951 | 2.701 | 0.219 | 49.74 |
| MinorB (y-y) | b | 0.34 | 387.32 | 0.803 | 0.925 | 0.722 | 49.74 |
| Torsional TF | b | 0.34 | 52.76 | 2.177 | 3.205 | 0.18 | 40.89 |

Moment Designs

| | M Moment kN-m | Mspan Moment kN-m | Md(yield) Capacity kN-m | Mdv Capacity kN-m | Mnd Capacity kN-m | Md(LTB) Capacity kN-m |
|---|---|---|---|---|---|---|
| Major (z-z) | 0.0799 | 0.0799 | 13.1014 | 13.1014 | 13.1014 | 14.5664 |
| Minor (y-y) | 0 | 0 | 13.1014 | 13.1014 | 13.1014 | |

| | Curve | αLT | λLT | ΦLT | χLT | C1 | Mcr (kN-m) |
|---|---|---|---|---|---|---|---|
| LTB | c | 0.49 | 0.152 | 0.5 | 1 | 2.7 | 626.2731 |

| | Cmy | Cmz | CmLT | kz | ky | KLT | My / Mdy | Mz / Mdz | α1 | α2 |
|---|---|---|---|---|---|---|---|---|---|---|
| Factors | 1 | 0.4 | 0.4 | 1.101 | 1.095 | 0.984 | 0 | 0.006 | 2 | 2 |

Shear Design

| | V Force (kN) | Vd Capacity (kN) | To Capacity (kN-m) | Stress Ratio | Status Check |
|---|---|---|---|---|---|
| Major (y) | 0.0558 | 154.3257 | 0 | 3.618E-04 | OK |
| Minor (z) | 0 | 154.3257 | 0 | 0 | OK |

Shear Design

| | Vp (kN) | kv (Unitless) | ΛW (Unitless) | Tb (MPa) |
|---|---|---|---|---|
| Reduction | 154.3257 | 0 | 0 | 1 |

b) Cantilever Pipe design

**<u>Pipe Design</u>**

Element Details (Part 1 of 2)

| Level | Element | Unique Name | Location (mm) | Combo | Design Type | Element Type | Section |
|---|---|---|---|---|---|---|---|
| Story2 | B25 | 28 | 1610 | DStlS8 | Beam | Special Moment Frame | Pipe1 |

Element Details (Part 2 of 2)

| Classification | Rolled |
|---|---|
| Class 1 | No |

Design Code Parameters

| γM0 | γM1 | An /Ag | LLRF | PLLF | Stress ratio Limit |
|---|---|---|---|---|---|
| 1.1 | 1.25 | 1 | 1 | 0.75 | 0.95 |

Section Properties

| A ($cm^2$) | Izz ($cm^4$) | rzz (mm) | Ze,zz ($cm^3$) | Av,z ($cm^2$) | Zp,zz ($cm^3$) | Iyz ($cm^4$) | It ($cm^4$) |
|---|---|---|---|---|---|---|---|
| 7.9 | 30.9 | 19.8 | 10.2 | 5 | 14 | 0 | 61.8 |

| J ($cm^4$) | Iyy ($cm^4$) | ryy (mm) | Ze,yy ($cm^3$) | Av,y ($cm^2$) | Zp,yy ($cm^3$) | Iw ($cm^4$) | h (mm) |
|---|---|---|---|---|---|---|---|
| 61.8 | 30.9 | 19.8 | 10.2 | 5 | 14 | | 60.3 |

Material Properties

| J ($cm^4$) | Iyy ($cm^4$) | ryy (mm) |
|---|---|---|
| 61.8 | 30.9 | 19.8 |
| E (MPa) | fy (MPa) | fu (MPa) |
| 210000 | 345 | 450 |

Stress Check Forces and Moments

| Location (mm) | N (kN) | Mzz (kN-m) | Myy (kN-m) | Vy (kN) | Vz (kN) | To (kN-m) |
|---|---|---|---|---|---|---|
| 1610 | -0.1114 | -0.0039 | 1.8E-05 | -0.0623 | 0.0071 | 0.0025 |

PMM Demand/Capacity (D/C) Ratio 9.3.2.2(a)

| D/C Ratio = P / Pdy + Ky * Cmy * (My,span / Mdy; ) + KLT * (Mz,span / Mdz; ) |
|---|
| 0.009 = 0.002 + 0.001 + 0.006 |

Basic Factors

| Buckling Mode | K Factor | L Factor | L Length (mm) | KL/r |
|---|---|---|---|---|
| Major (z-z) | 1 | 1 | 3220 | 162.689 |
| Major Braced | 1 | 1 | 3220 | 162.689 |
| Minor (y-y) | 1 | 1 | 3220 | 162.689 |
| Minor Braced | 1 | 1 | 3220 | 162.689 |

| Buckling Mode | K Factor | L Factor | L Length (mm) | KL/r |
|---|---|---|---|---|
| LTB | 1 | 1 | 3220 | 162.689 |

Axial Force Design

| | N Force kN | Td Capacity kN | Nd Capacity kN | Pdy Capacity kN | Pz Capacity kN | Pd Capacity kN |
|---|---|---|---|---|---|---|
| Axial | -0.1114 | 247.4133 | 247.4133 | 47.5344 | 47.5344 | 47.5344 |

| Tdg kN | Tdn kN | Ncr,T kN | Ncr,TF kN | An /Ag Unitless | N /Nd Unitless |
|---|---|---|---|---|---|
| 247.4133 | 255.5887 | 63715.1239 | 61.7729 | 1 | 4.503E-04 |

Design Parameters for Axial Design

| | Curve | α | fcc (MPa) | λ | Φ | χ | fcd (MPa) |
|---|---|---|---|---|---|---|---|
| Major (z-z) | b | 0.34 | 78.31 | 2.099 | 3.026 | 0.192 | 60.26 |
| MajorB (z-z) | b | 0.34 | 78.31 | 2.099 | 3.026 | 0.192 | 60.26 |
| Minor (y-y) | b | 0.34 | 78.31 | 2.099 | 3.026 | 0.192 | 60.26 |
| MinorB (y-y) | b | 0.34 | 78.31 | 2.099 | 3.026 | 0.192 | 60.26 |
| Torsional TF | b | 0.34 | 78.31 | 2.099 | 3.026 | 0.192 | 60.26 |

Moment Designs

| | M Moment kN-m | Mspan Moment kN-m | Md(yield) Capacity kN-m | Mdv Capacity kN-m | Mnd Capacity kN-m | Md(LTB) Capacity kN-m |
|---|---|---|---|---|---|---|
| Major (z-z) | -0.0039 | -0.0267 | 3.8575 | 3.8575 | 3.8575 | 4.3312 |
| Minor (y-y) | 1.8E-05 | 0.0061 | 3.8575 | 3.8575 | 3.8575 | |

| | Curve | αLT | λLT | ΦLT | χLT | C1 | Mcr (kN-m) |
|---|---|---|---|---|---|---|---|
| LTB | c | 0.49 | 0.232 | 0.535 | 0.983 | 1.414 | 78.514 |

| | Cmy | Cmz | CmLT | kz | ky | KLT | My / Mdy | Mz / Mdz | α1 | α2 |
|---|---|---|---|---|---|---|---|---|---|---|
| Factors | 0.689 | 0.602 | 0.602 | 1.002 | 1.002 | 1 | 0 | -0.001 | 2 | 2 |

Shear Design

| | V Force (kN) | Vd Capacity (kN) | To Capacity (kN-m) | Stress Ratio | Status Check |
|---|---|---|---|---|---|
| Major (y) | 0.0623 | 90.9373 | 0.0025 | 0.001 | OK |
| Minor (z) | 0.0071 | 90.9373 | 0.0025 | 7.833E-05 | OK |

Shear Design

| | Vp (kN) | kv (Unitless) | ΛW (Unitless) | Tb (MPa) |
|---|---|---|---|---|
| Reduction | 90.9373 | 0 | 0 | 1 |

End Reaction Major Shear Forces

| Left End Reaction (kN) | Load Combo | Right End Reaction (kN) | Load Combo |
|---|---|---|---|
| 0.0107 | DStlS22 | 0.0191 | DStlS22 |

**<u>Deflection Check</u>**

Element Details

| Level | Element | Unique Name | Location (mm) | Combo | Design Type | Element Type | Section | Rolled |
|---|---|---|---|---|---|---|---|---|
| Story2 | B25 | 28 | 0 | DStlD2 | Beam | Special Moment Frame | Pipe1 | No |

DEFLECTION DESIGN (Combo DStlD2)

| Type | Consider | Deflection mm | Limit mm | Ratio | Status |
|---|---|---|---|---|---|
| Dead Load | Yes | 0.1 | 26.8 | 0.003 | OK |
| Super DL + Live Load | Yes | 1.476E-02 | 26.8 | 0.001 | OK |
| Live Load | Yes | 1.476E-02 | 8.9 | 0.002 | OK |
| Total Load | Yes | 0.1 | 13.4 | 0.007 | OK |
| Total - Camber | Yes | 0.1 | 13.4 | 0.007 | OK |

### 2.12.1 Pipe Connection Design

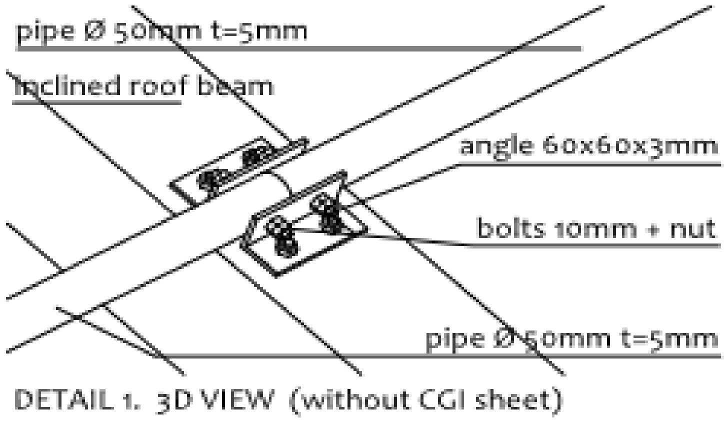



Figure 12: Pipe Connection Design

ISA 60 X 60 X 3 mm was choosen in tension which is connected to both sides of the ISNB80L hollow steel pipe section using M10 bolts of property class 5.6.

The yield and ultimate strength of the pipes section are 250 MPa and 415 MPa.

Provided,

Diameter of the pipe = 89 mm

Diameter of bolt (d) = 10 mm

Diameter of bolt hole ($d_h$) =10+1= 11mm

Here, property class of bolt is 5.6,

$F_{ub} = 5 \text{ X } 100 = 500 \text{ N/mm}^2$

And, $f_{yb}= 0.6 \text{ X } 500 =300 \text{ N/mm}^2$

For steel,

$F_u = 415$ MPa

$F_y = 250$ MPa

From table 5.1 of code IS 808: 1989, page 8A,

For ISA 60 X 60 X3 mm,

$Ag = 3.51 \text{ cm}^2 = 351 \text{ mm}^2$

Design strength of the angle due to yielding,

From clause 6.2 of code IS 800:2007,

$T_{dg} = (A_g f_y) / \gamma_{mo}$

$= (351 \text{ X } 415) /1.1$

$=132.422$ KN

Design of end connection:

Let one shear is in shaft and one is in the thread.

i.e $n_s = 1$ , $n_n = 1$

$A_{sb} = \pi/4\ d2 = \pi/4\ (10)2 = 78.53mm^2$

$A_{nb} = 0.78 \text{ X } A_{sb} = 61.26 \text{ mm}^2$

$$V_{dsb} = \frac{Vnsb \text{ X } (nn\ Anb + ns\ Asb)}{\gamma mb}$$

$$= \frac{500 \text{ X } (78.53 + 61.26)}{\sqrt{3} \text{ X } 1.25}$$

$V_{dsb}$=32.514 KN

Again,

$V_{dpb} = 2.5\ K_b\ d_t\ f_u / \gamma_{mb}$

$K_b$ is smaller of $e/3d_o$, $p/3d_o – 0.25$, $f_{ub}/f_u$ and 1

We have,

e is not less than $1.7\ d_h = 17$

e=20

$K_b$ is smaller of 25/ (3 X 10) , 60/(3 X 10)- 0.25 , 500/410 , and 1

0.833, 0.861, 1.22, 1

$K_b$= 0.833

Here,

$V_{dpb}$ = (2.5 X 0.833 X 10 X 6 X 450) /1.25

= 44.982 KN

For two bolt =65.028 KN

Use 65.028 KN

No. of bolts required = 132.422 / 65.028 = 2.036

Provide 3 bolts in each connection

Design strength due to rupture:

$A_{nc}$ = (60 – 11 – 3/2) X 3

=172.5 $mm^2$

$A_{go}$ = (60 – 3/2) X 3 = 205.5 $mm^2$

w= 60 mm

bs = $w + w_1 – t$

=20 + 60 -3

=77 mm

$L_c$ = 7 X 40 = 280

B = 1.4 – 0.076 X (60/3) X (250/415) X (77 / 280)

B = 1.15

$T_{dn} = (0.9\ A_{nc}\ f_u) / \gamma_{m1} + (B\ A_{go} f_y) / \gamma_{mo}$

= (0.9 X 172.5 X 415 ) / 1.25 + (1.15 X 415 X 205.5) / 1.1

= 140.701 KN

Tdn= 140.701 KN

Design Strength due to the block shear:

(Provide 25 mm edge distance from each end, i.e. breadth of angle

=25+25+10= 60 mm

Avg = (25 + 20) X 3= 135 $mm^2$

Avn = (25 + 20 – 0.5 X 10) X 3 = 105 $mm^2$

Atg = 25 X 3= 60 $mm^2$

Atn = (25 – 0.5 X 11) X 3 = 58.5 $mm^2$

Tdb shall be smaller of the two:

Tdb = (120 X 250)/ ($\sqrt{3}$ X 1.1) + (0.9 X 43.5 X 415) / 1.25

= 37.711 KN

For 2, = 75.422 KN

OR

$T_{db}$ = (0.9 X 105 X 415) / ( $\sqrt{3}$ X 1.25) + (250 X 60 ) /1.1

=38.220 KN

For 2, =76.44KN

Take least, i.e 76.44 KN.

The ultimate load carrying capacity is least of 132.422 KN, 140.701 KN and 76.44 KN.

The ultimate load carrying capacity of 2 ISA 60 X 60 X 3 with M10 bolts is 76.4 KN.

Calculation of strength required:

We have,

Basic wind speed ($V_b$) = 55 m/sec

Take $K_1$=1

$K_2$=0.88

$K_3$=1

Design wind speed ($V_d$) = 0.88 X 55 = 48.4 m/sec

Wind pressure ($P_w$) = 0.6 X $(V_d)^2$ = 1405 N/m$^2$

Wind load = ($C_{pi}$ – $C_{pe}$) X $P_w$

= (-1.2 – 0.2 ) X 1405

= 1967 N/m$^2$

Hence wind pressure for uplift is 1.967 KN/m$^2$.

Now total panel area of one slope of the roof is: 2.568 m X 6.44 m

So Wind load = 1.967 X 2.568 X 6.44

=32.53 KN

Dead Load =11.058 KN

Total Load =43.588 KN < (76.4 KN)

Hence the connection is safe.

### 2.12.2 Base Plate Design

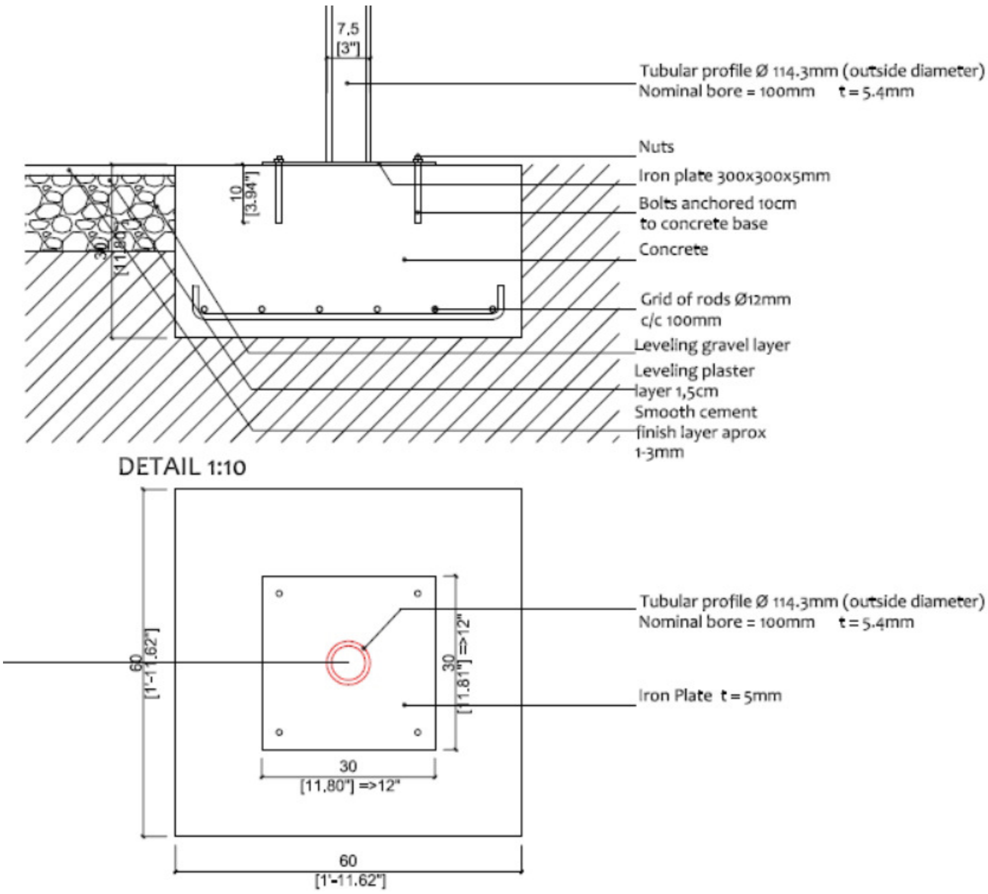



Figure 13: Base Plate Design

| | | | |
|---|---|---|---|
| | | | ASD |
| Axial Load of Compression (Pa) | = | 0.083 | kips, |
| Steel Plate Yield Stress (Fy) | = | 60 | ksi |
| Concrete Strength (fc') | = | 3 | ksi |
| | | | |
| Coulmn Size | => | HSS2.875X0.250 | |
| Nase Plate Size (N) | = | 12 | in |
| B | = | 12 | in |
| Area of Concrete Size ($A^2$) | = | 1156 | $in^2$ |

(Geometrically similar to and concentric with the loaded area.)

Use 12 X 12 in

1/8 thick plate (adopt 5mm plate)

ANALYSIS

CHECK BEARING PRESSURE (AISC 360

J8)

$$P_p / \Omega_c = \frac{f'_c A_1}{\Omega_c} MIN\left[0.85 MAX\left(\sqrt{\frac{A_2}{A_1}} \ , \ 1\right) \ , \ 1.7\right] =$$

293.7 6 kips > Pa

| | | | |
|---|---|---|---|
| Where | A1 = | 144 | in2, actual area of base plate. |
| | Wc = | 2.50 | [Satisfactory] |

DETERMINE VALUES OF m, n, n', X , and l (AISC Manual Page 14-5)

DETERMINE VALUES OF m, n, n', X , and l (AISC Manual Page 14-5)

| | | |
|---|---|---|
| m = 0.5 (N - 0.80 d) = | 4.85 | in |
| n = 0.5 (B - 0.80 d) = | 4.85 | in |
| n' = 0.25 (d b )0.5 = | 0.72 | in |

$$X = MIN\left[\left(\frac{4db}{(d+b)^2}\right)\frac{\Omega_c P_a}{P_p} \ , \ 1\right] = \quad 0.00$$

$$\lambda = MIN\left(\frac{2\sqrt{X}}{1+\sqrt{1-X}} \ , \ 1\right) =$$

.20.02

0.02

Where,

d = 2.88 in, depth of column section

b = 2.88 in, depth of column section

DETERMINE REQUIRED THICKNESS OF BASE PLATE (AISC Manual Page 14-6)

$$t_{min} = l\sqrt{\frac{3.33P_a}{F_yBN}} =$$

0.03 in

$l = MAX(m, n, \lambda n') = 4.85$ in

**CHAPTER 3**

**ECONOMIC ASPECTS and CONCLUDING REMARKS**

### 3.1 Economic Aspects

Majority of the building stocks, comprising different building typologies viz. i.e. Reinforced Concrete framed structure with masonry infill (RC), Unreinforced Masonry (URM), and Reinforced Masonry (RM). Adequate seismic resistance along with minimisation of construcfion cost of building is one of the challenges to be addressed by the structural engineer. The experimental results demonstrate the higher seismic resistance of confined masonry (CM) buildings, as compared for URM and RM. Hence, to balance the strength, safety and economy, CM may be adopted as appropriate solution. However, to clarify the economy in construction, rigorous cost analysis is warranted.

The RC buildings were designed in accordance with the design procedure detailed in this guidelines and relevant Indian standards viz. IS-456:2000, IS-875:2003, IS-1893:2002, and IS- 13920:2003. Similarly, URM, RM and CM buildings were designed as per IS-4326:2013, IS- 1903:2003, IS-456:2000, IS-875:2003, and IS-1893:2002.

### 3.2 Concluding Remarks

The document deals with the understanding the behaviour of confined masonry construction under seismic conditions and recommends the design guidelines for such buildings with explanatory design example. To build-up the confidence in confined masonry buildings, chapter 4 presents the seismic performance comparison of tested full scale single storeyed URM, RM and CM buildings subjected to reversed cyclic later displacements at roof level under quasi-static condition.

To examine economic aspects of CM building, ensemble of typical housing in India, were designed as RC, URM, RM and CM, for the uniform design parameters. The construction costs were computed for different structural elements and comparison of each typology was performed with reference to the construction cost of RC building. The results shows that CM, RM and URM buildings allows for average cost reduction of structure by 30%, 33% and 36% respectively, as compared to the RC framed structure. However, CM buildings offer reasonable saving when compared with the construction cost of RC framed buildings and higher level of safety when compared with URM/RM buildings.

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