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Analysis of Earthquake Resistant Compressed Stabilised Earth Block Buildings in rural Nepal

Common construction errors and their influence on structural resistance

Master's thesis in Structural Engineering and Building Technology

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Abstract

Saturday the 25th of April 2015 the largest earthquake since 1934 struck Nepal with its epicentre in the Gorkha region, naming it The Gorkha Earthquake. In the aftermath of The Gorkha Earthquake the non-governmental organisation Build Up Nepal was founded with the aim to provide an earthquake resistant building technique for houses that could be used in the rural areas in an attempt to help fight poverty and rebuild a safer Nepal. The decision was made to use compressed stabilised earth blocks (CSEB), approved and recommended by the government of Nepal.

Currently Build up Nepal have been using the CSEB for almost three years and the question of how well they actually work in practice have become current. To understand the structural resistance of buildings build with CSEB with impact from different construction errors and weakened material properties would be a great support for Build up Nepal to use in their work.

An Error Bank of commonly occurring errors in the CSEB buildings was compiled based on Build up Nepal's drawings (Build up Nepal 2017*a*), supported by a Picture Bank displaying the different errors on site. The impact from the errors were analysed with litterateur studies and FE-analysis in SAP2000. On the basis of the results from the analysis recommendations on how to treat and avoid the errors were given to Build up Nepal to be integrated in their future work. To further help with avoiding the errors a Checklist was created to be used by Build up Nepal's engineers on site. The Checklist is divided into the different construction phases showing the errors that can occur in the specific phase with pedagogic illustrations displaying the consequences of the errors.

The results from the analysis shows that the CSEB buildings constructed according to approved drawings performs well while the FE-analysis give strong indications that some errors are critical in terms of structural resistance.

In collaboration with Build up Nepal some of the errors showed to be severe, for example openings too close to corners and uneven layers of bricks, can be further studied in more detail. Also means to avoid and retrofit CSEB buildings is important to further improve and support Build up Nepal's work.

Keywords: CSEB, compressed stabilised earth blocks, earthquakes,structural resistance, earthquake design, Nepal, earthquake resistance, FE-analysis.

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Amanda Thudén and Alexandra Toivonen, Gothenburg, June 2018

Contents

1
Introduction

1.1 Background

Saturday the 25^{th} of April 2015 the largest earthquake since 1934 struck Nepal with its epicentre in the Gorkha region, naming it The Gorkha Earthquake (Government of Nepal 2015). The earthquake reached 7.8M on the Richter scale and the large amount of aftershocks that followed causing further devastation was unique. Over 300 aftershocks that reached over 4M were measured as well as four aftershocks that reached over 6M. A large percentage of Nepal's population, both in the rural areas and in the cities, were affected. An estimate of 8790 casualties, 22300 injured and over 8 million people affected was a result of the earthquake and the aftershocks. It was clear that the majority of buildings and infrastructure in the affected regions were not able to resist the impact of the earthquake and the damages and losses were estimated to about US$7 billion with the need of reconstruction estimated to about US$6.7 billion. Of these damages the majority of the total 490 000 houses destroyed were build and occupied by poor people in the rural areas.

In the aftermath of The Gorkha Earthquake the non-governmental organisation Build Up Nepal was founded with the aim to provide an earthquake resistant building method for houses that could be used in the rural areas in an attempt to help fight poverty and rebuild a safer Nepal (Build up Nepal n.d.). The technique had to be based on the local conditions, cultural aspects and take the economical situation of the people in these areas into account. For Build up Nepal's work to be affordable and in that way work as intended some rules, guidelines and terms have to be fulfilled according to the Government of Nepal to be able to take part in there Rural Housing Reconstruction Program (RHRP) (Nepal Rural Housing Reconstruction Program Multi-Donor Trust Fund 2016). The RHRP aims to rebuild a safer Nepal with the help of governmental support; "The program's overall objective will ensure that houses destroyed in the most affected districts of the country will be rebuilt using earthquake-safer building techniques through grants and technical assistance to eligible households from the Government of Nepal".

The decision was made to use compressed stabilised earth blocks (CSEB), a form of hollow interlocking brick (HI). Interlocking CSEB provides an earthquake resistant building technique approved and recommended by the government of Nepal and commonly used in other earthquake-prone countries such as Iran and India (Auroville n.d.). The method consist of mixing local soil with cement and water. The mixture is then compressed in an machine that forms the interlocking bricks under compression. The bricks are set to cure for 28 days before they can be used in construction. The benefits from using CSEB are that it is a material that can be produced locally by mostly local components, it is an earthquake resistant building technique and it has lower investment cost than for example fire bricks. Disadvantages of CSEB is that the material properties of the bricks may vary due to varying soil quality and, in relation to the work done by Build up Nepal, due to the fact that the production and construction is done by non-professionals which might lower the resistance of the brick (Maïni 2005). Despite CSEB being an approved earthquake resistant building technique in Nepal, still no national building code for CSEB exists. This means that for every new design of a CSEB building drawings and structural analysis needs to be send in for approval which increase the workload and therefor naturally the cost (Government of Nepal, Ministry of Urban Development n.d., Build up Nepal n.d.).

Build Up Nepal is an organisation that in large is dependant on the help from volunteers, such as a Master's thesis projects supporting the organisation with knowledge and time. For some time Engineers without Boarders Sweden and Build up Nepal have carried out joint research projects where Chalmers University of Technology have been involved (Build up Nepal n.d.).

Currently Build Up Nepal have been using the CSEB for almost three years and the question of how well they actually work in practice have become current (Build up Nepal n.d.). To understand the structural resistance of buildings build with CSEB with impact from different construction errors and varying material properties from production would be a great support for Build up Nepal to use in their work to focus their effort and avoid errors being made.

1.2 Aim and objectives

The aim of this study was to make a high level overview of the structural resistance of one of the approved designs, design number two in the catalogue for approved designs by Build up Nepal (2017*a*), of CSEB buildings built in rural Nepal. The study includes analysis of the structural resistance when built according to the approved building designs with material properties according to the Nepal National Building Code, Indian Standards, previous studies such as the Master's Thesis by Herman Mellergård and Axel Steinert (2016), Auroville (Maïni 2005) and Euro Code 6 (European Committee for Standardization 2005). Analysis including commonly occurring construction errors were then performed to show their relative impact. Furthermore, as a way for Build up Nepal to emphasise the importance to follow drawings and directives when constructing a CSEB building and to have a system to avoid errors being made, pedagogic visualisations of the impact of construction errors on the structural resistance in addition with a phase-wise organised list of the errors was compiled in a Checklist, see Appendix C.

Objectives:

- Identification of requirements for CSEB-buildings in the Nepal
- Identification of materials properties from the current building codes and other indicative documents concerning CSEB
- Studies of Build up Nepal's approved designs of CSEB-buildings
- Studies of the presence and impact of earthquakes in Nepal
- Studies of how CSEB-buildings are build in practice by Build up Nepal
- Identification and documentation of the errors commonly occurring when constructing with CSEB in a Error Bank
- Identification and documentation of the variety of material properties of CSEB occurring due to faulty production
- Literature studies on the impact from the detected common construction errors
- FE-analysis on design two from and according to the catalogue of approved designs by Build up Nepal (2017*a*)
- FE-analysis of selected detected common construction errors included in the model
- Creation of a Checklist for Build up Nepal to use in their work

1.3 Methodology

The project can be divided into three main stages, (1) preparation for field studies, (2) gathering of data on site in Nepal and (3) processing of the data and identification of the results.

In the first stage of the project a literature study was carried out to gain knowledge about CSEB and CSEB buildings. Additionally, the literature study was focused on how to analyse structures with regard to seismic loading as well as gain understanding of the seismic situation in Nepal. This provided a good framework to start from when working on site in Nepal. To gain a better understanding a comparison to other regions, such as India and Iran, where CSEB are also used as a earthquake resistant building method, was made as well as studies of the different codes for seismic design of structures.

The second stage of the project took place on site in Nepal. Field trips to different villages where Build up Nepal are active were carried out to gain an understanding about the production of CSEB, the construction of buildings made with CSEB as well as what the most common errors are when building with CSEB in Nepal. Both observations, interviews and a study of existing material such as pictures and documentation were used to gain this knowledge.

In the third stage of the process, an ideal model was initially created in the FE-program SAP2000 modelled according to drawings (Build up Nepal 2017*a*). Relevant loads including wind loads and seismic loads were applied. Further, models based on the first model were created including the errors detected. In addition to the FE-analysis a literature study was made on the impact of different errors on the earthquake resistance specifically.

The results, displacements and local stresses, from the FE-analysis were compared to the ideal model, to the requirements set in the Nepal National Building Code and other relevant building codes and to the capacities of the CSEB building. A conclusion was made of the impact on the structural resistance by the different errors.

1.4 Limitations

There were a few limitations to this study. First and foremost, the context of these buildings are in a developing country and one has to appreciate that this has some influences on the national building codes and regulations. The current Nepalese Building Codes are sparsely defined and explained and often reference to the Indian Standards. An other aspect to take into account is that there are no inspections of finished constructions in the rural areas of Nepal to control that everything is built according to approved drawings.

An other limitation aspect is to the building material itself. CSEB has not been studied in the same extent as more commonly used building materials such as concrete, steel and timber. Some assumptions and simplifications have thereby been conducted on how the material preform together with mortar as a masonry unit. Comparisons to how masonry made off fire bricks and lightweight aggregate blocks was carried out. The quality assurance of the bricks is usually lacking when they are locally produced by unskilled labour. Observations and interviews was the main methods used to gather information about the most likely and common errors, that means that the construction errors either have to be visual or that the engineers and architects at Build up Nepal and the locals has to have a good memory about the building process or awareness of the errors previously made. The common errors are more or less unique to the context but for simplification some categorisation and generalisations about the errors was made. There were also assumptions about how to model the errors in an FE-analysis software since no regulations nor building codes exists on this specific topic.

The FE-analysis in this study has been carried out as linear elastic analysis due to the simplicity of modelling and time commitment to analyse each construction error. Nevertheless, this provides another limitation to the results due to the fact that no crack propagation nor failure of structures could be identified. The results were limited to indicated when the masonry initiate cracks and where.

2

Theory

2.1 Earthquakes

To understand and be able to handle the effects of earthquakes on structures it is important to understand the underlying theory of earthquakes. Section 2.1 presents the basics of earthquakes which is a topic considered in most literature within the subject. Literature studied regarding the basics of earthquake theory in this report are Scholz & Shedlock (2017), Armouti (2015), Bangash (2011), Tomazevic (1999), Alvarez, Hurtado & Bedoya-Ruíz (2012), Chandak (2012) and Okamuraa, P.Bhandarya, Moria, Marasinia & Hazarikab (2015).

2.1.1 Introduction

Earthquakes are one of the most destructive natural phenomena and they regularly strikes our planet. The occurrence of most earthquakes are not possible to detect without advance seismologic measuring instruments but in earthquake prone zones major earthquakes strike regularly and causes severe material damage, personal injuries and loss of life. One of the largest difficulties with earthquakes is that they are impossible to reliably predict and therefore escape from. In the event of an earthquake, one of the largest consequences causing personal injuries are man-made constructions falling down. The importance of well constructed buildings becomes clear as well as the emphasis that has to be put into developing the field of structural engineering to increase quality and possibilities of earthquake resistant building techniques with economical consideration.

The five main events that can cause earthquakes are earthquake from tectonic movements, volcanic activities, cave collapses, natural and man-made explosions and from filling reservoirs. The majority, about 90 %, of all earthquakes come from tectonic movement and is therefore the most studied event.

2.1.2 Earthquakes from tectonic plate movement

The earth consists of an outer crust, varying in thickness from a few kilometres to a few tens of kilometres, that is divided into several plates. The outer 40 to 70 kilometres of the earths shell together with the crust is often referred to as the lithosphere. The plates of the outer shell are constantly moving in regard to each other, based on the theory that the rigid lithosphere drift on the rheological asthenosphere underlying the lithosphere. Where two plates meet faults or fault planes can occur.

The plates can move away from each other, towards each other pushing one plate down and the other one up or the two plates can slide along each other. Friction prevents the two plates to move independently at the boundary, building up energy in the plates. When the limit of the frictions capacity is reached, the energy is released in a phenomenon called elastic rebound. The energy released in the elastic rebound sets of seismic strain waves in all directions causing ground motion and an earthquake is initiated.

There are different kinds of strain waves released in the event of an earthquake, see Figure 2.1. These waves are classified into two types of waves, surface waves and body waves. Within the group body waves there are P-waves, fast primary waves, and S-waves, slow shear waves. Within the group surface waves there are R-waves, Rayleigh waves, and L-waves, Love waves. The P-waves and S-waves move at different speed and the difference in time when they reach a point can be used as a measurement of distance from the hypocentre.

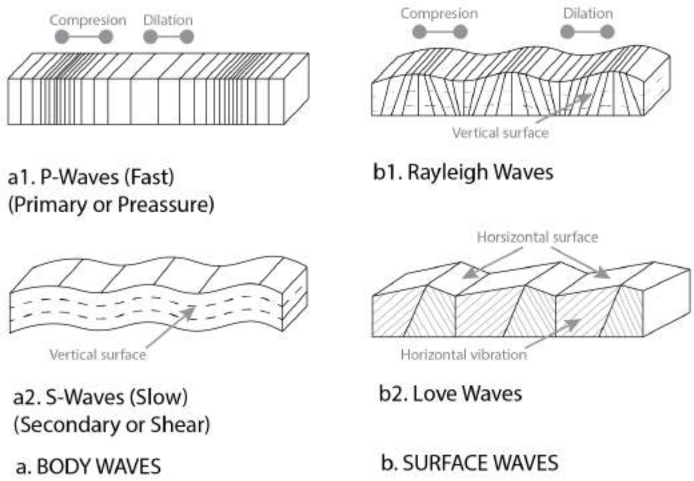


Figure 2.1: Illustration of strain waves based on picture by (Armouti 2015)

The effect earthquakes have on buildings does not only depend on the mechanism and characteristic of the earthquake at its source. When the rebound occur and seismic waves radiate out from the hypocentre area, the waves have to travel through numerous layers of soil and bedrock before reaching the surface and inducing vibration in structures. Dependant on the type of soil and bedrock as well as the length of travelling the waves reflect and refract in different ways. The waves also change their amplitude and frequency of oscillation along the way.

2.1.3 Earthquake characterisation

To quantify the size and effect of an earthquake the energy released at the source, the hypocentre, is determined as the magnitude of an earthquake. The size of an earthquake at other locations is measured by the intensity at that specific site. Today hundreds of seismometers around the world constantly record the seismic activity providing information about earthquakes. In Figure 2.2 important components to understand and describe an earthquake is shown.

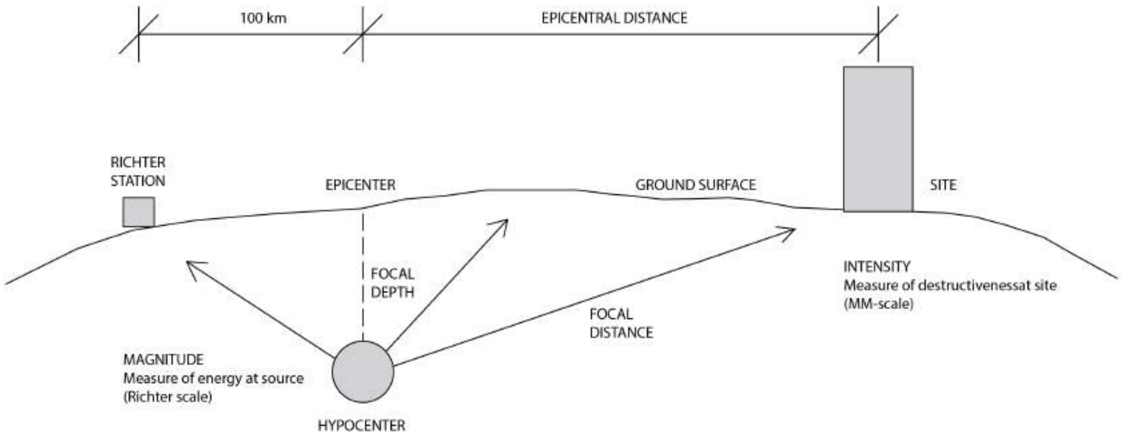


Figure 2.2: Earthquake parameters based on illustration by Armouti (2015)

2.1.3.1 Magnitude

A commonly used word when talking about earthquakes is magnitude. The magnitude of an earthquake is a measurement of the energy released at the source of the earthquake and gives a picture of its strength. The magnitude was first defined by Charles Richter in 1935 and is therefore measured on the so called Richter Scale. The magnitude was then defined by the equation; $M = log(I/S)$, where I is the amplitude measured by a standardised seismometer at, what should be, exactly 100 km away from the epicentre and S is the amplitude of a standard earthquake. In practice the magnitude of an earthquake is given by recalculating the largest measured ground motion read by a seismometer to what a seismometer 100 km away from the epicentre would show.

The Richter scale is a logarithmic scale that goes from one to nine. An earthquake with a one or two on the Richter scale is barely noticeable whereas an earthquake with a seven or higher causes massive structural damages that often lead to extensive personal injuries and losses of life. The ground motion and the impact of an earthquake is strongest closest to the epicentre and decreases with distance. To what speed and with what characteristic intensity the earthquake decreases is dependent on multiple geological parameters. The value of the magnitude in combination with the distance from the epicentre can therefore only be seen as a good indicator of the impact of an earthquake at a certain location.

2.1.3.2 Intensity

To describe the local destruction caused by an earthquake at the surface of the earth at different locations, intensity scales are commonly used. The intensity is a subjective value based on individuals experience and impression of the impact of an earthquake at a certain distance from the epicentre collected through surveys.

The earthquake intensity was developed to be a physical quality, before seismic instruments were available to provide a quantitative measurement of ground motion. The intensity is often used in official context and in post-disaster assessments to give a picture of the destruction at different locations.

There are a number of different scales describing the intensity of an earthquake. The most commonly used scale for intensity is the Modified Mercalli (MM) scale. The MM-scale is divided into 12 levels where a one means that individuals only felt the earthquake under extremely favourable circumstances, and a 12 means that individuals experienced total destruction.

Many equations exist that have been developed in attempts to provide a mathematical relationship between intensity scales and other earthquake parameters such as magnitude, distance to the epicentre and peak ground acceleration, velocity and displacement. Such a relationship could provide atomised mapping of the intensity after an earthquake and in that way simplify emergency distribution and other emergency actions such as shut-off of gas supply.

2.1.4 Quantitative measurements

The above mentioned measurements, magnitude and intensity, of an earthquake gives a picture of the severeness of the earthquake but can not be used by structural engineers in design nor analysis. To be able to make structural analysis and designs, taking earthquakes into account, a quantitative measurement is needed. Accelerogram is a record of the acceleration of the ground versus time and provides such a measurement. The accelerogram is often used to derive a response spectra used in design to calculate a buildings theoretical response to an earthquake. A response spectra plots the displacement, velocity or acceleration of an earthquake at a certain position against the frequency. Since seismic ground motions are complex the ground acceleration is often measured in North-South, East-West and vertical direction. The recorded accelerogram for the Gorkha Earthquake can be seen in Figure 2.3 (a).

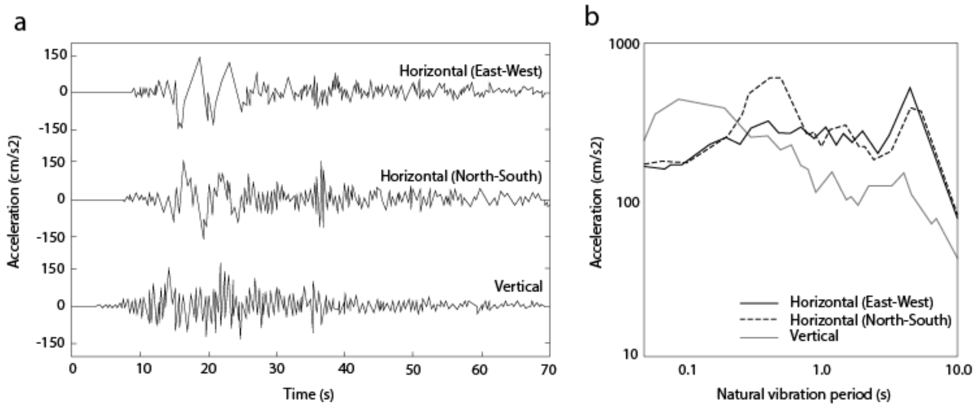


Figure 2.3: (a) Illustration based on recorded accelerograms of the Gorkha Earthquake at KATNP and (b) Illustration of 5 %-damped spectral accelerations of the recorded accelerograms at KATNP. Based graphs by Chiaroa et al. (2015)

Important parameters such as peak ground acceleration (PGA), length of continuous pulses and total duration can be read directly from the accelerogram and other important parameters can be obtained by mathematical analysis of the accelerogram. Such parameters are frequency content, peak ground displacement, peak ground velocity and power spectral density. The response spectra can also be constructed from the accelerogram and is used in most national building codes for seismic design. The response spectra for an earthquake plots the response in the form of either displacement, velocity or acceleration to the frequency. In Figure 2.3 (b) a response spectra of the Gorkha Earthquake can be seen.

2.2 Earthquakes in Nepal

According to the United Nations Development Program, Nepal is the 11^{th} most earthquake-prone country in the world, which has been shown through history (Government of Nepal 2015) and can be explained by the geology of the country (Okamuraa et al. 2015). The Himalaya mountain chain, going straight through the entire length of Nepal, has developed due to the collision of the Indian plate and the Eurasian plate, sometimes referred to as the Tibetan plate, as can be seen in Figure 2.4. The Indian plate moves north, pushing down underneath the Eurasian plate. This movement increases the height of the Himalaya with approximately two centimetres per year but also leads to major faults distributed along the boarder which regularly generates earthquakes.

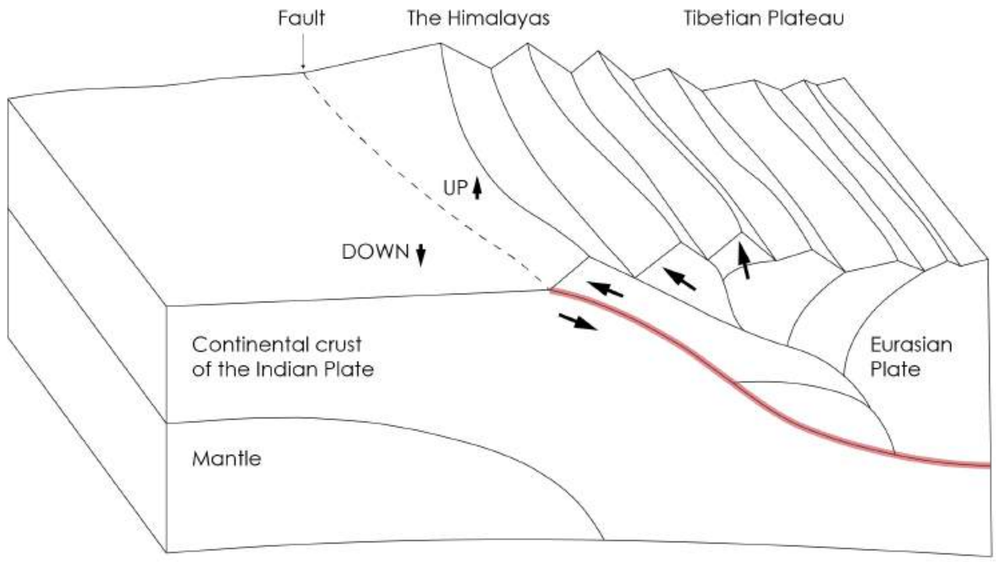


Figure 2.4: Tectonic plate movement in Nepal based on illustration by McLain & Wang (n.d.)

2.2.1 History of earthquakes

There are records of earthquakes in the Nepal Himalayas as far back as to the 13^{th} century, which is said to have killed one third of the inhabitants of the Kathmandu Valley, but without reliable details about the destruction caused (Government of Nepal 2015). From the records it can be seen that about every 100 years a major earthquake has occurred in Nepal. The most recent earthquake, The Gorkha Earthquake, occurred in 2015 which was 81 years after the previously recorded major earthquake in 1934, that lead to more than 10 000 deaths in the Kathmandu Valley. In the last 35 years another three large earthquakes has occurred in the Nepal Himalayas (Okamuraa et al. 2015). In 1980 an earthquake with the magnitude of 6.5 struck the western regions of Nepal. In 1988 an earthquake with the same magnitude struck the eastern Nepal and in 2011 a 6.9 magnitude earthquake struck the border between India and Nepal. Almost every year between 1993 to 2003 Nepal has been struck by less severe (<M6) earthquakes and almost every month minor earthquakes can be measured.

2.2.2 The Gorkha Earthquake

Looking at the history of earthquakes in Nepal, but also in the entire region of the Himalayas, it can be seen that major earthquakes has occurred in the East and the West parts of the Himalayas leaving a seismic gap in the central parts of the Nepal Himalayas (Okamuraa et al. 2015). This made researchers anticipate the Gorkha Earthquake to occur about 80-100 years after the 1934 earthquake in the area where it struck. Due to the history, with majority of seismic activity in the Eastern and Western Himalayas as well as the recent and current seismic activity, the researchers believe that yet another major earthquake will occur in the region in the next 10-30 years.

At 11:56am, local Nepalese time, on the 25^{th} of April 2015 the Gorkha Earthquake struck Nepal with a magnitude of 7.6 recorded by Nepal's Seismological Centre (Government of Nepal 2015). The earthquake had its epicentre in Barpak in the district of Gorkha as can be seen in Figure 2.5. Barpak is located about 76 kilometres Northwest of Kathmandu which means that the earthquake struck with highest impact in one of the most densely populated areas in Nepal. Following the initial 7.6 magnitude earthquake, four major aftershocks occurred with magnitudes over 6 as well as over 300 earthquakes with magnitude over 4 was recorded.

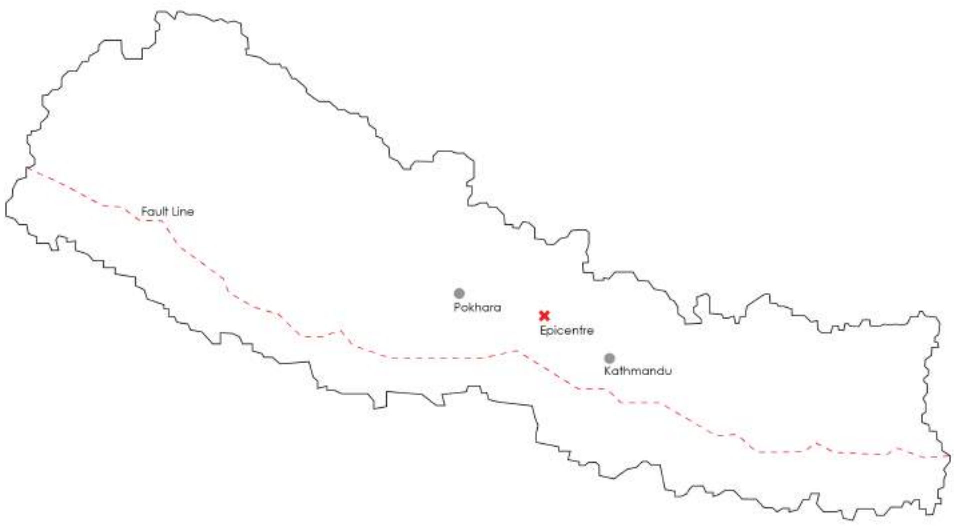


Figure 2.5: Map of Nepal with fault line and epicentre based on illustration by Pesta & Mandhana (n.d.)

2.2.2.1 Post disaster

The destruction in Nepal was vast and all parts of the country were affected by the earthquake in some way while about one fourth of the country was severely hit (Government of Nepal 2015). The categorisation of affected areas made by the Post Disaster Needs Assessment team, PDNA team (Government of Nepal 2015), can be seen in Figure 2.6. Damages included destruction of residential and governmental buildings, heritage sites, schools, hospitals, infrastructure, water supply systems, agricultural land, hydro-power plants etc. The most affected rural areas were partly swept away by landslides and triggered avalanches. The Eastern and Western rural areas, also partly devastated by the earthquake, became further isolated due to damages of the rural roads which were their only connection to more developed areas.

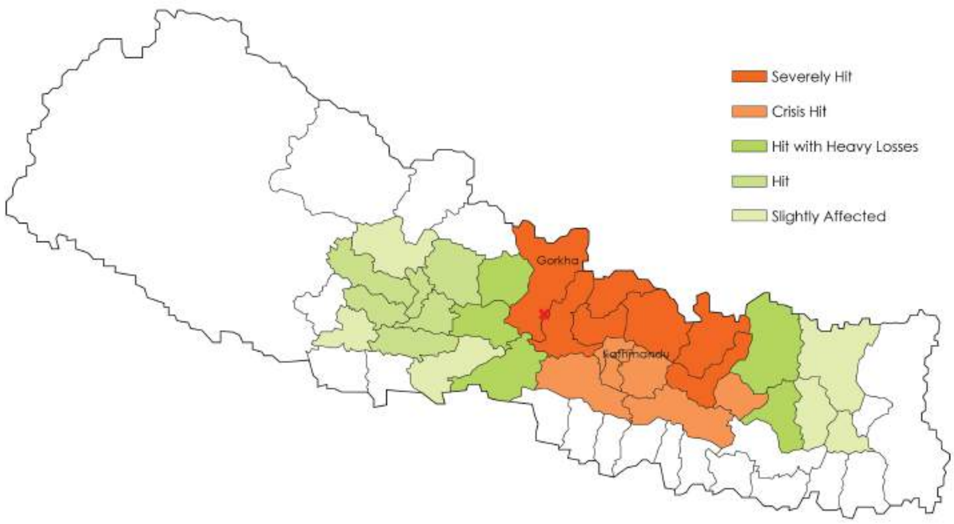


Figure 2.6: Earthquake-affected districts after the Gorkha Earthquake based on illustration by Government of Nepal (2015)

The earthquake damaged more than half a million houses around Nepal, exposing the earthquake vulnerability that prevailed in the country at that time. These damaged houses were not earthquake resistant and were most likely not built according to the building codes. Until this day there are large parts of the country where no building permit is needed to build, which means that the building codes, enforced to ensure that safe houses are build, does not necessarily need to be followed (Government of Nepal 1994*a*). In the PDNA (Government of Nepal 2015) it was detected that rural and poor areas were more affected by the earthquake than the cities due to their lower quality buildings.

PDNA was a joint exercise between different contributors, foreign and domestic, led by the National Planning Commission (NPC) to make an early assessment of the damages as well as the needs after the Gorkha Earthquakes (Government of Nepal 2015). The PDNA team concluded that "Housing" was the single largest field affected by the earthquake and constituted approximately 60 % of the total cost of disaster.

2.2.2.2 Post disaster actions

The PDNA team also made an estimation of the financial needs after the earthquake, divided into different sectors (Government of Nepal 2015). Also here the single largest field is housing of about 50 % .

After completion of the PDNA the Nepalese Government addressed the need for reconstructing a safer Nepal by implementing the National Housing Reconstruction Program (Nepal Rural Housing Reconstruction Program Multi-Donor Trust Fund 2016). The program aims to support the rebuilding of all the damaged houses and serve as a coordinating framework for standardised housing reconstruction, independent of the source of funding.

In a more directed effort to support the fourteen most affected rural areas with housing reconstruction the Rural Housing Reconstruction Program, RHRP, was founded. Technical support would be provided, supported by grants to those using approved earthquake resistant housing techniques to ensure a long-term disaster resilience.

The implementation procedure of the RHRP is structured around five consecutive steps. In the first step the Central Bureau of Statistics (CBS) assess the fourteen beneficiary districts generating a beneficiary/damage database that serves as the basis for the program. Secondly, eligible households within each district are chosen for the program following the beneficiaries enrolment in the program by legally binding agreements. Reconstruction can start based on, for the RHRP fundamental, owner-driven principal. Beneficiaries are supported technically as well as financially in a three-steps-grant process upon certification of earthquake resistant techniques guided by Nepal's National Building Code (NBC).

After completion of the reconstruction the beneficiaries receive the so called Building Construction Completion Certificate and the final portion of the grant.

2.3 Dynamic response of structures

In the event of an earthquake, seismic waves radiate from the epicentre causing vibrations in buildings in the surrounding area. Additional to static loads, dynamic loads are therefore affecting structures and consequently the dynamic behaviour has to be included in analysis of structures in earthquake-prone zones.

The basics of structural dynamic is described by authors in several published books, for example by Bangash (2011), Chopra (1995), Armouti (2015) and Tomazevic (1999).

2.3.1 Structural requirements for dynamic behaviour

When looking at a structures capacity to withstand the effect of an earthquake additional parameters than the ones used in vertical load analysis becomes interesting (Armouti 2015). If only the elastic strength demand of a structure would be considered in regard to earthquake loading the costs would be extremely high. To reduce the material demands that the elastic response puts on a structure, additional structural requirements arise. Such requirements are put on the ductility, energy dissipation and self-centering capacity of a structure.

2.3.1.1 Ductility

Ductility is an important parameter of a structure when it comes to earthquake loading (Armouti 2015). The ductility of a structure is a measurement of its capacity to undergo large deformations without any significant reduction of strength. The ductility of a structure is often referred to in terms of high or low ductility. Materials with low ductility are often referred to as materials with a brittle behaviour.

2.3.1.2 Energy dissipation capacity

Another highly important parameter in the context of earthquake resistance is the energy dissipation capacity. This is the capacity of a structure to dissipate a part of the absorbed energy. Since a vast amount of energy is inflicted on a structure during an earthquake excitation this parameter will have significant influence when it comes to the building's capacity to withstand an earthquake. Compared to the energy absorption, which is all energy imparted on the structure including the elastic energy, the energy dissipation is the energy dissipated in the structure in any form, e.g. in the form of cracking and yielding. The elastic energy that is absorbed but not dissipated in a structure is converted back into kinetic energy and will make the structure swing back during vibration. The ductility and energy dissipation capacity are not related. A completely elastic system exhibit no energy dissipation capacity (Armouti 2015).

Both ductility and energy dissipation are advised to take into consideration in the Nepal National Building Code, NBC 105 (Government of Nepal 1994*d*).

2.3.1.3 Self-centering capacity

The self-centring capacity of a structure is also a parameter of interest (Armouti 2015). If a inelastic structure returns to its original position after earthquake excitation within the inelastic range the structure possesses self-centring capacity. The permanent plastic deformation remaining after the excitation time is called the plastic drift.

2.3.2 Typical failure modes of unreinforced masonry structures

To understand the failure modes of the sparsely studied CSEB-buildings, typical failure modes of masonry buildings made of firebricks will be discussed followed by a comparison of the two building materials.

2.3.2.1 Damage and failure of load bearing walls

For masonry buildings to resist seismic loads the structural configuration is most commonly shear walls, solid or perforated by door and window openings (Tomazevic 1999). Reported by Arya, Boen & Ishiyama (2013) the primary cause for damages are the tensile stresses and shear stresses in the walls. The most common failures in load bearing walls under seismic loading can be divided into three different categories; out-of-plane seismic movement, in-plane seismic movement and too poor anchorage of the structural elements, see Figure 2.7.

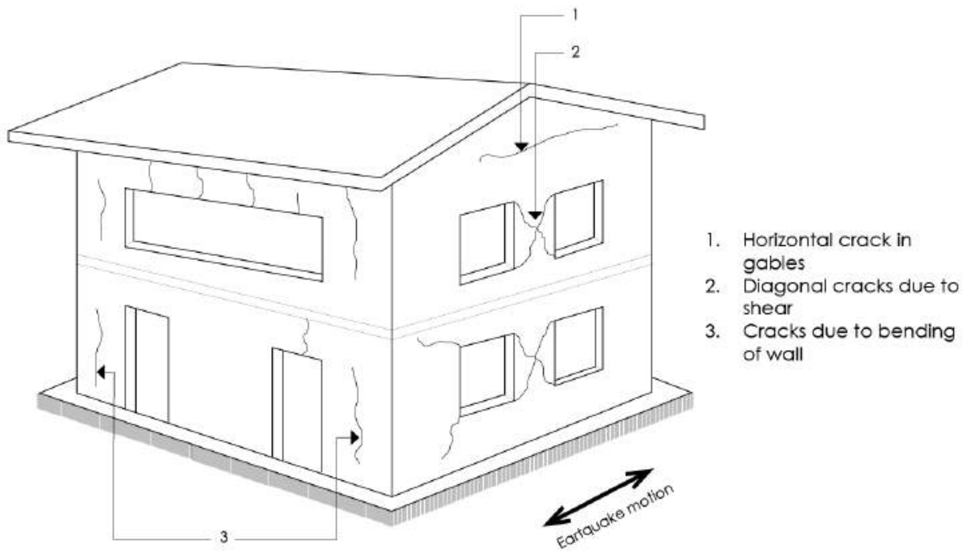


Figure 2.7: Cracking of masonry building in bearing walls due to bending and shear motion based on (Arya et al. 2013).

2.3.2.2 Out-of-plane failure of shear walls

The structural walls which are oriented perpendicularly to the seismic movement undergoes out-of-plane bending that gives cause to vertical cracks in the middle of the wall or at the wall ends (Tomazevic 1999, Arya et al. 2013), see Figure 2.7. The longer the wall segment and the longer the openings are the more severe are the damages that possibly can lead to partial or complete collapse of the wall. However, as long as the wall spans are not too extreme, the prominent failure mode will not be out-of-plane bending of the structural walls.

2.3.2.3 In-plane failure of shear walls

For the walls oriented in the direction of the seismic movement in-plane forces will occur according to Paulay & Priestley (1992). In general, large walls with no openings will not have a problem with the in-plane forces and no real instability takes place. The walls will manly rock on their basis. However, if the uplifting displacements are too large, failure might occur gradually by spilling bricks on the tension end of the wall. This failure mode can be seen on the right hand in Figure 2.8.

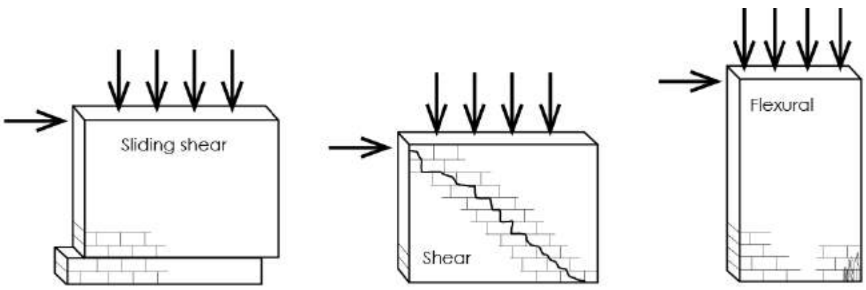


Figure 2.8: The typical failure modes for masonry walls subjected to in-plane seismic movements based on (Tomazevic 1999).

Mainly three different failure mechanisms occur during in-plane seismic loading as seen in Figure 2.8. Which mechanism that will occur highly depend on the geometry of the wall and the quality of the materials according to Tomazevic (1999). Sliding shear failure will typically happen when the wall is subjected to low vertical load and is built with low quality mortar. If the vertical load is at a normal level, shear failure or bending failure are more prominent to occur. As summarised by Anderson & Brzev (2009) the walls that undergoes in-plane seismic loads need, together with roof and floor diaphragms, to transfer the lateral seismic loads to the foundation and down into the ground.

Both Arya et al. (2013) and Tomazevic (1999) agree that shear of an unreinforced masonry wall will most often initiate a failure mode that is characterised by diagonal cracks, see Figure 2.7 and Figure 2.8. These cracks occur mainly due to diagonal tension in the wall. This failure mode will either crack through the mortar between the bricks or completely diagonally through the masonry. These cracks are commonly initiated at corners of openings or in the middle of a wall segment. This failure mechanism can cause partial or complete collapse of a building.

If, however, the wall has a higher shear resistance and a high moment to shear ratio, flexural failure mode are more prominent to occur Tomazevic (1999) argues. This can be seen in crushing of the compressed zones at the ends of the wall, seen in Figure 2.8. During an earthquake a structure will often withstand seismic movement along both axes at the same time. This means that both bending and shear effects act simultaneously as failure modes (Arya et al. 2013).

Between two floors with openings vertically aligned, a critical area exists in the deep beam between the openings when the wall undergoes lateral in-plane forces, see Figure 2.9. Diagonal cracking of the spandrels will occur before cracking of the piers, unless the piers are very narrow. This can be taken care of if full distribution of shear takes place between all piers. By introducing a rigid slab or a reinforced cement concrete (RCC) band between the floors, full distribution can be achieved (Arya et al. 2013).

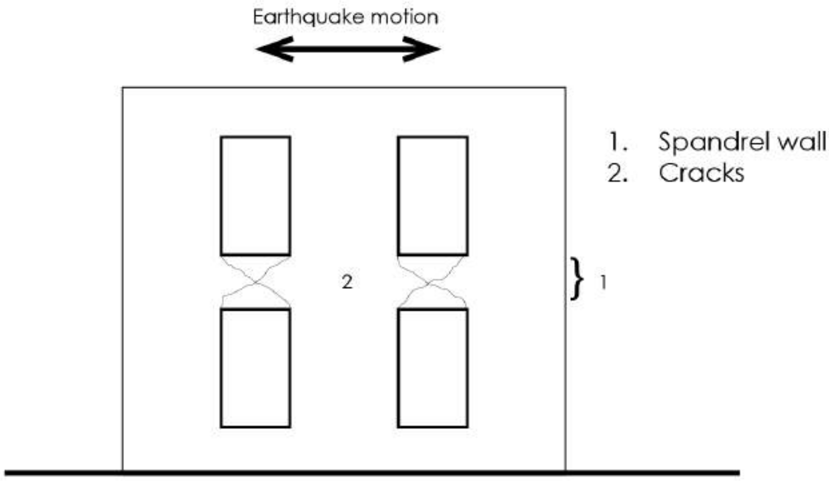


Figure 2.9: Cracking of masonry building at spandrel wall between vertically aligned openings based on (Arya et al. 2013).

2.3.2.4 Failure of gable walls

The gable walls of a building are critical spots if they are not reinforced, as they can become unstable during seismic loading. Senaldi, Magenes & Ingham (2012) report that this is due to the low gravity loads and poor anchorage to the surrounding roof trusses. Moreover they argue that the increased accelerations at the top of the structure are the prominent reasons for collapsing unreinforced gable walls due to overturning. In addition to this Arya et al. (2013) report that the gable ends have to withstand pushing actions from the purlins of the roof. Horizontal bending tension cracks usually develops in the gables during earthquake actions, see Figure 2.7. Together with the imposed loads of the roof purlins this often cause the failure mechanism of the gable walls.

2.3.2.5 Failure of roofs and floors

When looking at the impact seismic load has on different kind of roof structures two main different failure mechanisms occur according to Arya et al. (2013). The seismic movement of the roof may damage the underlying walls and cause partial or complete collapse of these or cause partial or complete collapse of the roof structure itself, see Figure 2.10 and Figure 2.11.

Common characteristics about roof structures that are prominent to cause damage to underlying walls are massive, flat roofs that are improperly supported on the underlying walls, see Figure 2.10. The failure mechanism is commonly initiated by formation of tension cracks and separation from the walls. Furthermore, if the connection between the wall and the foundation is insufficient the wall might crack at the foundation connection and slide (Arya et al. 2013).

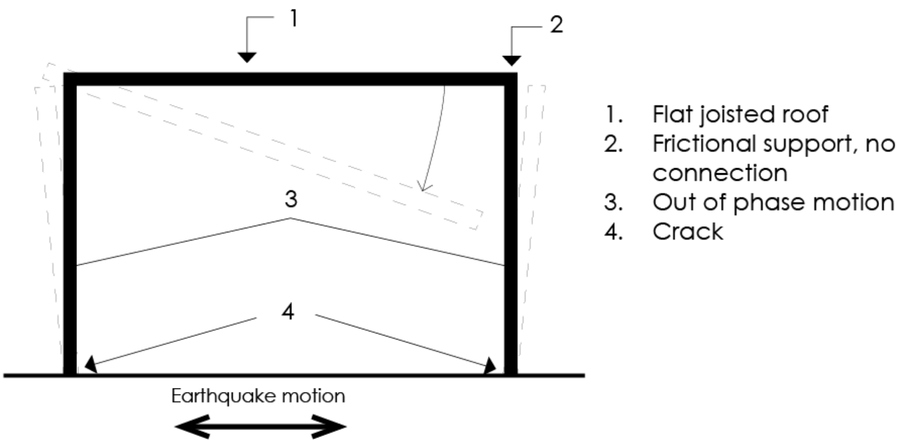


Figure 2.10: Failure of roof construction due to weak connection between massive flat roof and wall based on (Arya et al. 2013).

Typical failure mechanisms, according to Arya et al. (2013), for sloping roof are that the roofing members dislodge due to the increase of inertia forces from the seismic loading. This is especially common when slates, clay and tiles are used as roofing materials, see Figure 2.11. If the connection between the supporting wall and the roof is insufficient, separation of roof trusses from the support may occur. Also, complete roof collapse usually happens due to failure of the supporting structure.

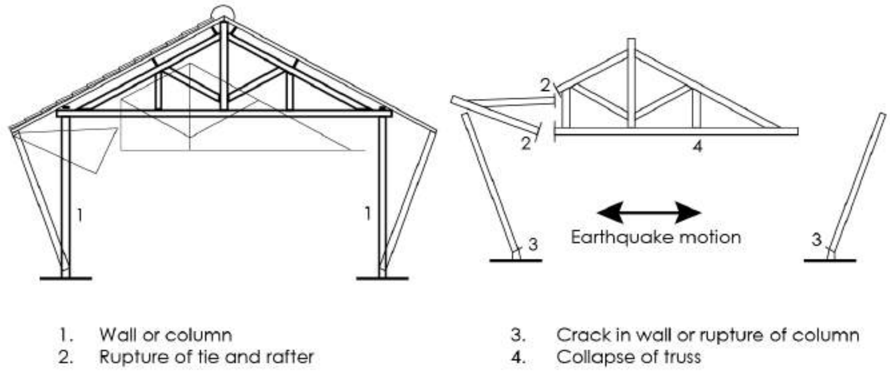


Figure 2.11: Failure of roof construction due to dislodging of roof structure based on (Arya et al. 2013).

2.3.3 Behaviour of reinforced masonry

Masonry is usually characterised as brittle building material and since ductility is a highly desirable characteristic of a structure under seismic loading, as described in Section 2.3.1.1, measures to achieve ductility must be taken (Kaushik, Rai & Jain 2007). A good way according to Reitherman & Perry (2009), to obtain a ductile behaviour of a masonry structure is to integrate steel reinforcement into the structure.

Reinforcement can be added in different ways to a masonry structure. This section will focus on the methodology that reinforcement is added in Build up Nepal's designs of CSEB buildings. That means RCC bands for the horizontal reinforcement and grouted cavity masonry for the vertical reinforcement, see Figure 2.12 (Tomazevic 1999, Build up Nepal 2017*a*).

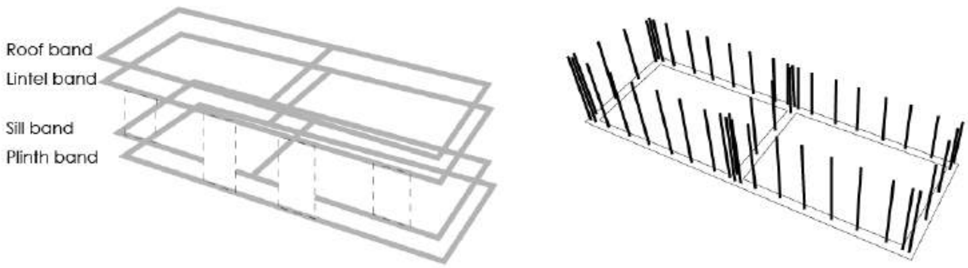


Figure 2.12: Reinforcement layout in masonry structures, horizontal RCC bands and vertical ties based on (Shrestha 2012).

2.3.3.1 Box behaviour

By adding reinforcement, the masonry structure will have a more monolithic response under seismic loading Reitherman & Perry (2009), Shrestha (2012) agree. Reinforcement need to be integrated in a grid of horizontal and vertical reinforcement to the structure to attain this behaviour, see Figure 2.13.

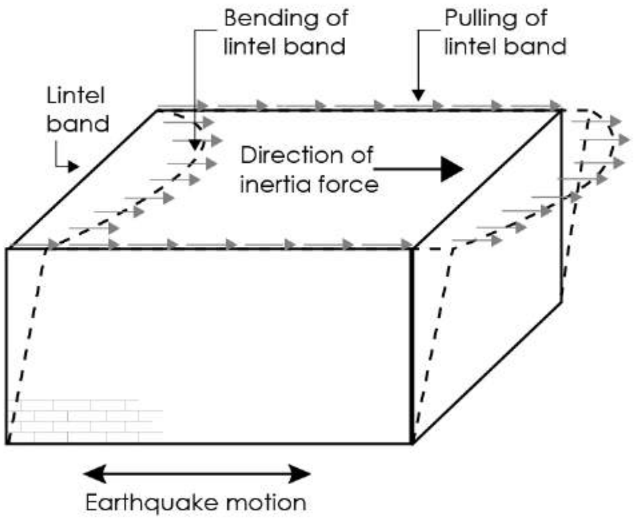


Figure 2.13: By adding reinforcement to the structure a box behaviour can be achieved based on (Murty 2002*a*).

2.3.3.2 Horizontal reinforcement - RCC bands

By introducing horizontal reinforcement in form of RCC bands mainly two beneficial attributes are added to the structure in terms of seismic resistance according to Arya et al. (2013) and Murty (2002*c*). Firstly the horizontal bending strength to resist out-of-plane bending is increased. Secondly perpendicular walls are tied together. This means that walls that are loaded in their weak direction can gain stability from the perpendicular walls loaded in their strong direction. Reported by Murty (2002*a*) in partition walls the RCC bands can help to prevent temperature cracks and shrinkage as well.

RCC bands should be provided continuously through all load bearing walls, see Figure 2.12. According to Build up Nepal (2017*a*) and Arya et al. (2013) the bands that should be provided for a one storey building are plinth, sill, lintel and roof RCC bands. Furthermore, if the structure is built on soft soil a RCC band should be provided at the bottom of the foundation as well. If the structure have masonry gable walls a triangular RCC band, a so called gable band, should be provided around the wall end.

As mentioned above the walls in a masonry structure will transfer the seismic loads to each other. This means, according to Murty (2002*c*), that corners of walls and RCC bands are critical parts of the structure. Due to this, openings near corner connections are not favourable since they inhibit the flow of forces between the walls.

2.3.3.3 Vertical reinforcement

Even though horizontal RCC bands are integrated in a masonry structure the building is still weakened by the openings in the load bearing walls presented by Murty

(2002*b*) and Arya et al. (2013). A masonry wall can be divided into three subdomains between the RCC bands, sill masonry, wall pier and spandrel, see Figure 2.14. When the structure undergoes seismic loading the narrower wall piers might disconnect from the masonry above and below causing crushing in its corner regions, see Figure 2.15. This failure mechanism is prominent to occur when the wall piers are slender and the dead load of the structure above is relatively small. Elsewise, the piers are more likely to fail in diagonal shear cracking.

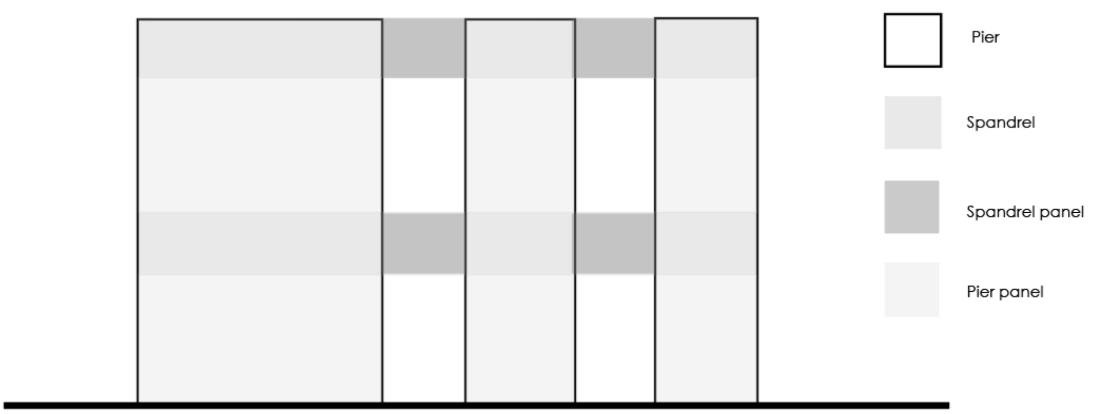


Figure 2.14: Illustration showing piers and spandrels of a masonry wall.

According to Murty (2002*b*) vertical reinforcement is integrated into the structure to avoid these failure mechanisms. By anchoring the steel bars in the foundation and roof band a more favourable behaviour of the structure will occur under seismic loading. Instead of the wall piers disconnecting from the masonry regions above and below, causing crushing in the corner regions of the piers, they will undergo bending. The vertical reinforcement will furthermore prevent the wall from sliding as well as collapsing due to out-of-plane bending, see Figure 2.15.

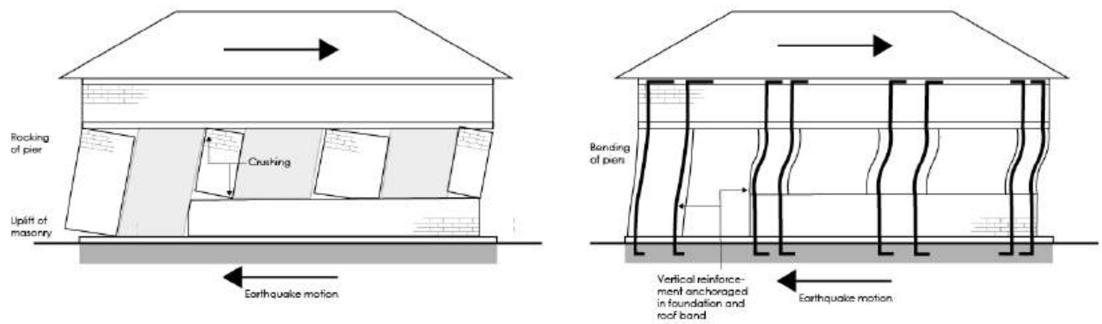


Figure 2.15: Schematic illustration of how vertical reinforcement change the behaviour of the wall piers in masonry structures under seismic loading based on (Murty 2002*b*).

2.3.4 Structural layout

When a masonry structure is subjected to earthquake loading more than just the materials used and their qualities affect the earthquake resistance reported by Tomazevic (1999), Arya et al. (2013) and Maïni (2005). It has been shown that an important feature of the structure is to have regularity and symmetry, preferably along two axes, in the structural layout, both in plan and elevation, see Figure 2.16 and 2.17. In regular and simple masonry structures both gravity and seismic loads are able to be redistributed in an undisturbed way from element to element. When regular shapes are not possible to provide an improvement to the structures' earthquake resistance is to divide the building in several regular parts, see Figure 2.16.

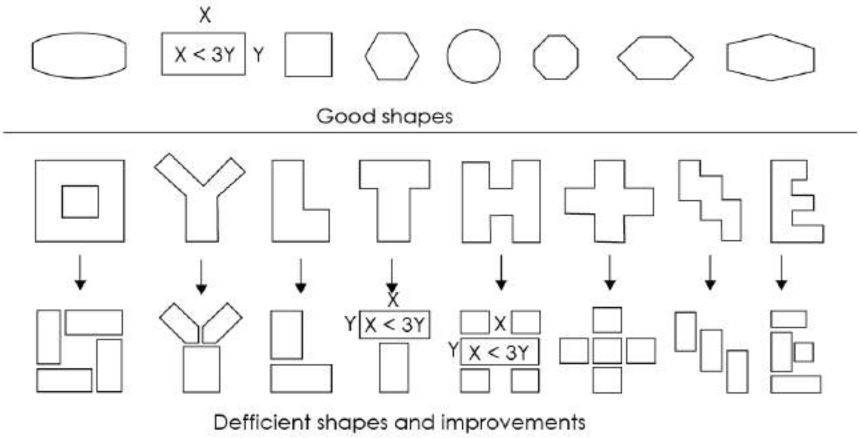


Figure 2.16: Good structural layouts of floor plans from an earthquake resistance point of view based on (Maïni 2005).

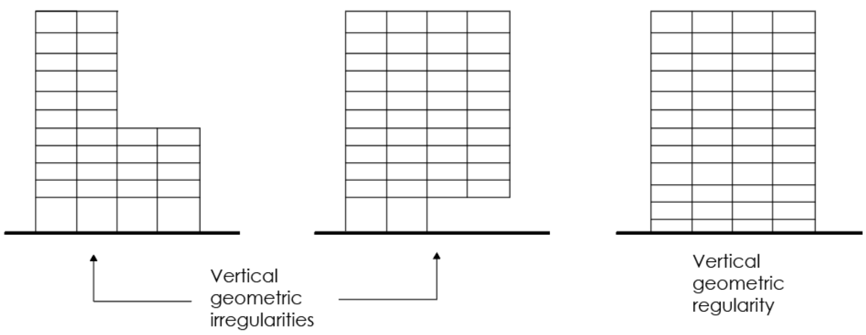


Figure 2.17: Illustrations of irregularities and regularity in elevation based on (Anderson & Brzev 2009)

Agreed by Maïni (2005), Tomazevic (1999) and Anderson & Brzev (2009) by making sure the structure also have a regularity in elevation, as well as in plan, the centre of gravity will coincide with the centre of rigidity of the vertical masses, see Figure 2.17 and 2.18. By ensuring that the centre of gravity and centre of rigidity is at the same location torsion of the building can be avoided. If the structural system lack regularity in plan and elevation stress concentrations might occur in the zones of non-uniformity leading to heavy damages and collapse.

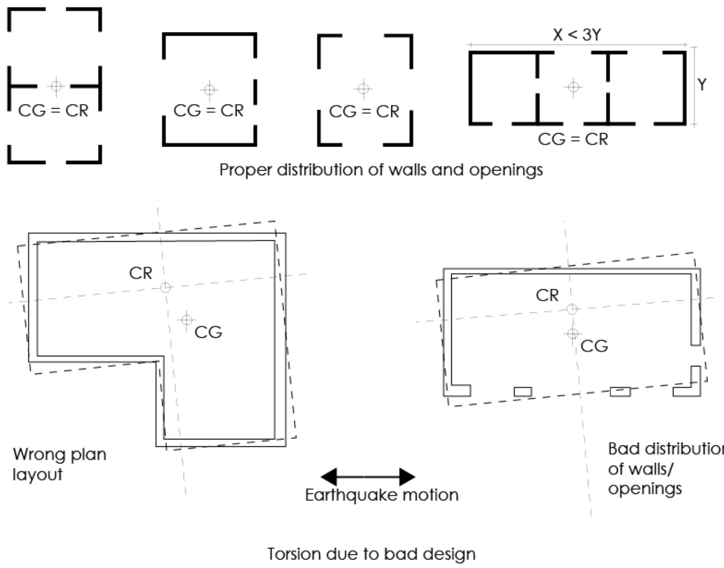


Figure 2.18: The centre of gravity in plan should coincide with the vertical masses' centre of rigidity based on (Maïni 2005).

2.3.5 Comparison - CSEB and masonry

In this study the comparison of CSEB and other more standard types of masonry, described and handled in for example EN 1998-1-1 (European Committee for Standardization 2005), will be made. This is due to the lack of existing studies on specifically CSEB.

The similarities between a standard masonry structure and a CSEB structure is mainly the composition of different units, masonry and mortar, and the behaviour under loading. The similarity between a reinforced masonry structure and a reinforced CSEB structure, such as the ones designed by Build up Nepal (Build up Nepal 2017*a*), can also be made in terms of the grouted reinforcement within the bricks. The more standard types of bricks and the CSEB might compose of different material properties which has been neglected in this comparison. The largest difference between standard masonry structures and CSEB structures in the interlocking key of the CSEB presented in Maïni (2005).

Furthermore, the lifespan of the CSEB used by Build up Nepal is unknown since the brick is relatively new on the market, while firebricks on the other hand is known for its long life span. Since this is not the focus of this study the dissimilarity has been neglected.

2.4 Compressed Stabilised Earth Blocks

2.4.1 Background

As reported by Rigassi (1985) can humans' use of soil as a building material be traced back thousands of years by the use of soil bricks in adobe buildings, where they moulded and sun dried earth bricks. One can say that the CSEB is a modern development of the old adobe blocks. The CSEB are composed of a mixture of soil, cement and water. The soil is taken from the site and the cement in the mixture is used as a stabiliser for the blocks. The mixture is compressed by an arm press and then let to cure. This makes the CSEB a local and low cost material since cost can be cut in raw materials, production and transportation of the bricks. The CSEB are also less polluting and energy consuming than for example firebricks (Maïni 2005). Today the leading institute doing research on CSEB is the Auroville Earth Institute. They have since 1995 researched systems of hollow interlocking reinforced masonry with CSEB. Thereby the following section covering CSEB as a building material will be based on Auroville Earth Institute's research.

2.4.2 Production

For the production of the CSEB different kind of arm presses can be used and regardless of which compression machine is decided to be used the same line of production is recommended to be set up. Maïni (2005) recommends a production using the Auram 3000 press and an adequate line of production is to use a linear organisation or block yard organisation, but also a circular one can be suitable. This line of production is divided into six different stages: preparation, measuring, mixing, pressing, initial curing and first stacking and final curing and stacking, see Figure 2.19. This line of production requires approximately 11 to 13 people working when using one Auram press 3000. By using this line of production Maïni (2005) approximates that 850 blocks can be produced a day. An ordinary approved house design from Build up Nepal (2017*a*) requires approximately 2500 bricks.

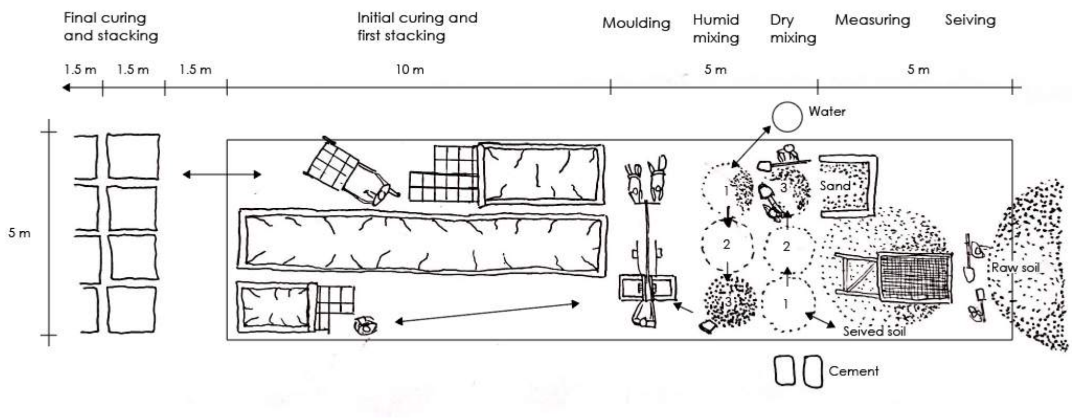


Figure 2.19: Schematic illustration of linear production based on (Maïni 2005)

In the initial stage of production the soil need to be checked to ensure a good quality of the end product. It is preferable to use a more sandy soil than one with a higher clay content, see Table 2.1. To ensure a good soil quality the top layer of soil is removed and checked so that the root and vein content is not too high (Maïni 2005). In rural areas when laboratory tests is not an option to ensure the soil composition Adam & Agib (2001) recommend to conduct sedimentation test on site. The sedimentation test is, in short, conducted by filling a cylindrical glass jar with one-third of clean water and one-third of dry soil (with maximum grain size of 6 mm) together with one teaspoon of salt. The mixture is then shaken to be well mixed. After approximately 45 minutes a clear layering of the soil composition can be identified, see Figure 2.20. When a good quality of soil has been ensured sieving can begin to remove too big pebbles (200-20 mm) from the soil content, see Table 2.2 (Maïni 2005).

Table 2.1: The recommended soil composition for compressed stabilised earth blocks (Maïni 2005)

Soil components	Gravel	Sand	Silt	Clay
Percentage of mixture	15 %	50 %	15 %	20 %

Table 2.2: The grain size classification, pebbles should be removed from the soil to be mixed in CSEB (Maïni 2005)

Pebbles	Gravel	Sand	Silt	Clay
200 - 20 mm	20 - 2 mm	2 - 0.06 mm	0.06 - 0.002 mm	0.002 - 0 mm

Figure 2.20: Illustration of sedimentation test based on (Adam & Agib 2001).

The soil will be measured and mixed with 10 % of cement as a stabiliser. Normally silt and clay would be the binder in soil, however they are not stable when the soil is wet so cement is added into the mixture to stabilise the bricks. After the dry mixing water is added to create a paste to form the bricks from. The appropriate amount of paste is added to the Auram Press' mould and bricks are created from compressive force. The bricks are then placed to initially cure in the open, protected from direct sunlight, for 24 hours and then moved to dry under plastic sheets during the initial curing process. After the initial curing process the bricks will be stacked protected from direct sunlight. During both curing processes the blocks need to be watered to prohibit the bricks to dry out too quickly, see Figure 2.19 (Maïni 2005, Build up Nepal 2017*a*). The bricks need to be tested after 28 days of curing to see if they fulfil a minimum compressive strength of 3.5 MPa (Ministry of Urban Development - Department of Urban Development and Building Construction 2017).

2.4.3 Construction

The typical way to construct a one storey, symmetric residential house, approved by the Nepalese Government, will be presented in the following section. Even though varieties and slightly different approaches might occur, this section explains the most common steps of building with CSEB in Nepal.

The initial step during construction of CSEB buildings is to excavate and cast the foundation. Different methods may be used to build the foundation but the most typical one is to cast a RCC band followed by a stone masonry wall foundation. Before casting the foundation a minimum depth of 900 mm of excavation for the foundation should be prepared, depending on the type of soil and number of storeys of the building, and the earth should be rammed according to Nepal National Building Code NBC 203:1994. In the cast foundation vertical and horizontal reinforcement need to be anchored, see Figure 2.21 (Build up Nepal 2017*a*).

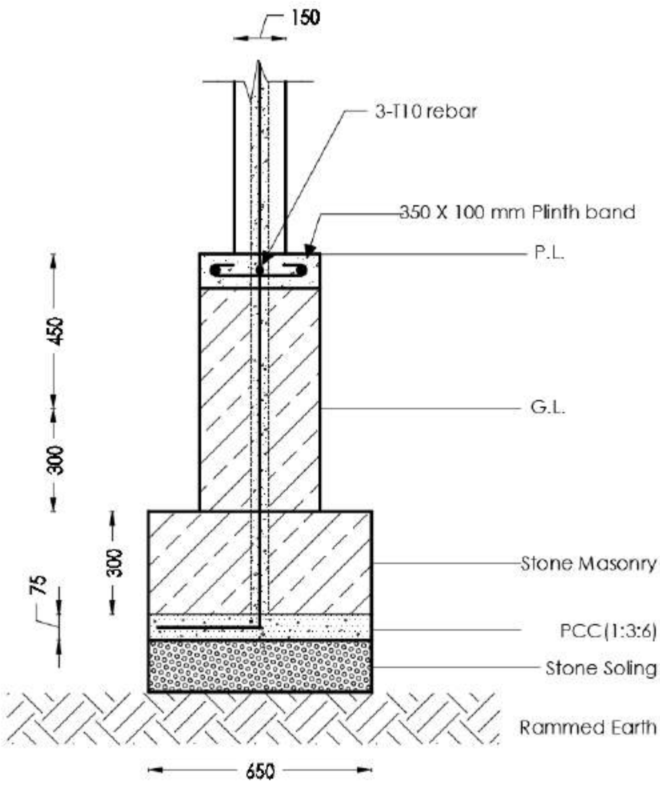


Figure 2.21: Section of foundation detail of a one storey, two rooms and kitchen CSEB-house (not in scale) based on drawing from (Build up Nepal 2017*a*).

A stone foundation stabilised with cement mortar is built on top of the RCC strip followed by a RCC plinth band. Horizontal reinforcement is placed in the plinth band to help withstand lateral forces and create a box behaviour of the building. After the plinth band has cured the layers of CSEB are placed with a layer of maximum 5 mm of mortar in between the bricks (Maïni 2005). Additional RCC bands will be cast at sill and lintel level as well as on top of the wall and around the gable walls, see Figure2.22 (Build up Nepal 2017*a*).

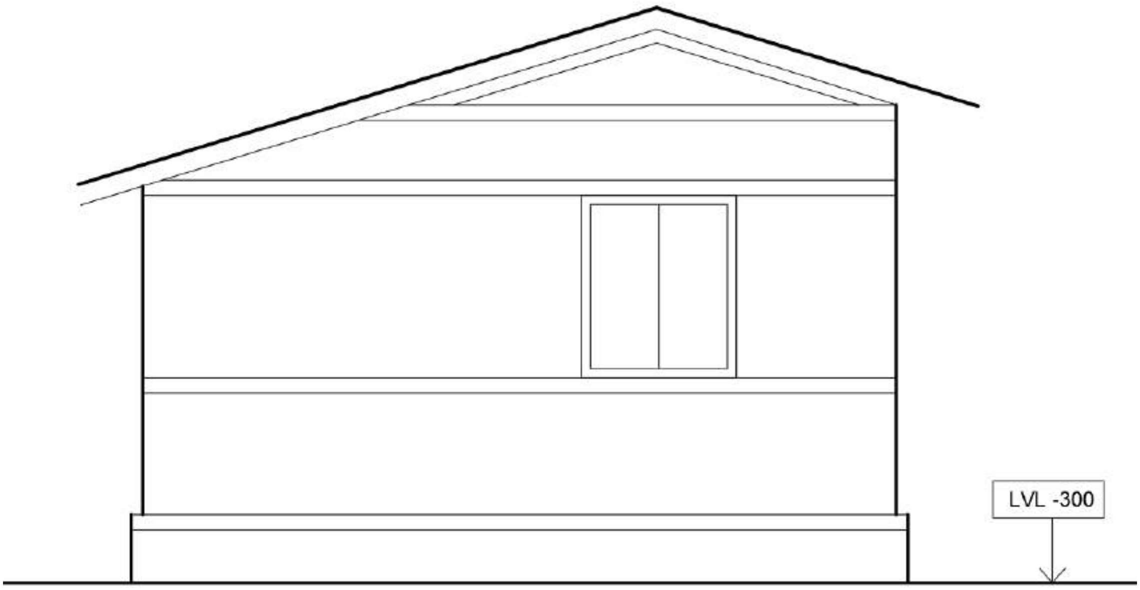


Figure 2.22: Facade elevation of a one storey, two rooms and kitchen CSEB-house (not in scale) based on drawings from (Build up Nepal 2017*a*).

Vertical reinforcement need to be every 1.2 meters as well as in the corners and around openings to strengthen the box behaviour of the building, see Figure 2.22. The vertical reinforcement should be, as mentioned previously, anchored in the RCC band below the foundation and go through the whole of the wall section and be anchored at the top RCC roof band. In the RCC roof band the steel profiles to anchor the roof girders should be cast as well. The girders should then be welded to the steel profiles followed by the purlins being welded to the girders. Lastly the roof cover will be screwed to the purlins (Build up Nepal 2017*a*).

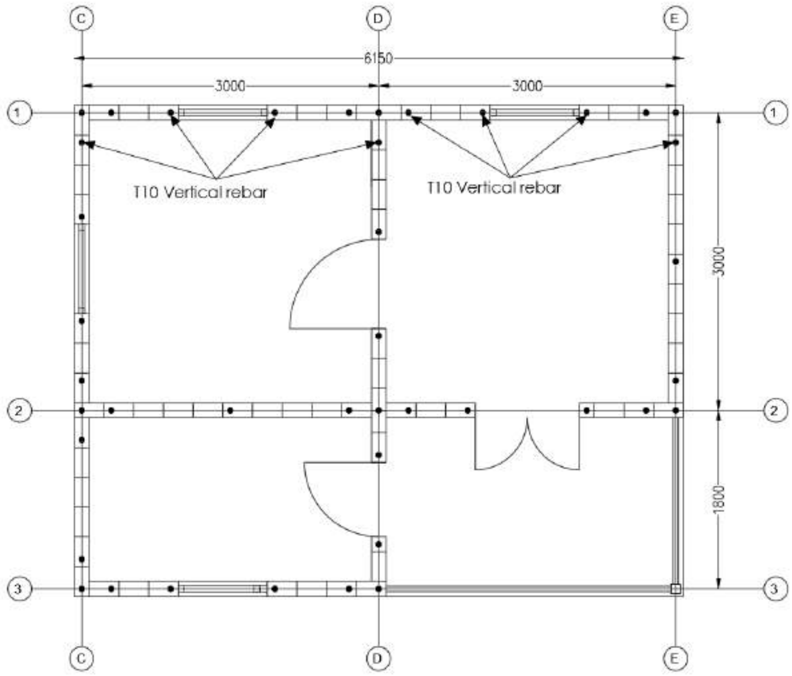


Figure 2.23: Plan drawing of vertical reinforcement of a one storey, two rooms an kitchen CSEB-house (not in scale) based on drawing from (Build up Nepal 2017*a*).

2.4.4 Earthquake design with CSEB

As of March 2017 CSEB became an approved earthquake resistant building method by the Nepalese Government (Ministry of Urban Development - Department of Urban Development and Building Construction 2017). As mentioned previously there is currently no separate building code for using CSEB. Instead each drawing of an CSEB house design need to be sent in to the Nepalese Government and get approval as an earthquake resistant building. Build up Nepal has 12 approved designs of CSEB residential houses and schools (Build up Nepal 2017*a*).

To ensure that the system of CSEB are earthquake resistant the blocks and masonry must fulfil some requirements according to the research done by Maïni (2005). First and foremost the blocks need to satisfy a strict consistency in height. Only a deviance of 1 mm is an allowed tolerance for the blocks. Furthermore, the blocks should be hollow to allow for vertical reinforcement and additional horizontal reinforcement to be cast in RCC bands or tie bands. This is done to ensure a monolithic box behaviour of the building. The biggest shear due to an earthquake will occur in the length of the wall. To ensure that the wall will withstand these loads the interlocking key of the bricks must interlock well in longitudinal and transverse direction to the length of the wall. To achieve full shear strength of the masonry the courses must have good bond between them. To use the full capacity of the blocks the entire area of the blocks as well as the interlocking keys need to transmit the loads. This is achieved by having a good seating of the blocks on top of each other in every course. Additionally the blocks should not be dry stacked but be placed with mortar in between the courses to achieve a homogeneous masonry. The mortar used between the blocks should be based on sand and cement. The thickness of the mortar should be 5 mm thick and the mortar should be of a quite fluid consistence and easily workable to achieve this.

2.5 Earthquake load

2.5.1 Level of earthquake load in national building codes

According to the Structural Engineers Association of California (1999) the requirements for the level of seismic loads provided in national building codes should be that structures resist minor earthquakes without any damage, resist moderate earthquakes with no structural damage but possibly some damages to non-structural elements and that structures withstand the severest experienced or forecast earthquake in the zone without collapse but possibly with damages to structural and non-structural elements.

Structures are not designed to withstand a certain earthquake magnitude but certain intensities at specific sites. This is handled by the zoning factor in national building codes, for example in NBC 105 (Government of Nepal 1994*d*). According to C.V.R. Murty (n.d.) the seismic zones in India, found in IS 1893 (Earthquake Engineering Sectional Committee 2002), are based on recorded intensities in different areas from past earthquakes. In 2002 the seismic zoning map of India was revised and today four different seismic zones exist, II, III, IV and V. They represent a maximum intensity on the MM-scale, see Section 2.1.3.2, of VI or less, VII, $VIII$ respectively IX.

NBC 105 (Government of Nepal 1994*d*) is based on the Indian Standards and therefore also the Indian Standards way of dividing the country into seismic zones.

2.5.2 Earthquake design approaches

According to the studied national building code that handles seismic load there are a few different methods provided. In this section the most common methods will be presented and explained, and the Nepal National Building Code about seismic design, NBC 105 (Government of Nepal 1994*d*), will be explained more detailed in Section 2.5.3.

2.5.2.1 Seismic coefficient method

The seismic coefficient method is a common method used to handle seismic loaded structures in a simple manner by translating the horizontal dynamic load into a horizontal static load (Hamada 2014). The method applies a design horizontal seismic coefficient, calculated with different parameters depending on building code, to the seismic weight of the structure to obtain the horizontal seismic shear force acting on the base of the structure caused by the earthquake (Government of Nepal 1994*d*, Earthquake Engineering Sectional Committee 2002). This shear force is normally referred to as the base shear or the inflicted inertia force. The main part of application of the seismic coefficient method lays in how to obtain the coefficient and dividing the base shear between levels of the building.

Following NBC 105 (Government of Nepal 1994*d*) the base shear is calculated according to Equation 2.1 and following IS 1893 (Earthquake Engineering Sectional Committee 2002) it is calculated according to Equation 2.3.

$$V_B = C_D \cdot W \qquad [kN] \tag{2.1}$$

$$C_D = C \cdot Z \cdot I \cdot K \qquad [-] \tag{2.2}$$

Where:

- C_D is the design horizontal seismic coefficient
- W is the seismic weight of the structure
- C is the basic seismic coefficient for the fundamental translational period in the direction under consideration
- Z is the seismic zoning factor
- I is the importance factor for the building
- K is the structural performance factor

$$V_B = A_H \cdot W \qquad [N] \tag{2.3}$$

$$A_H = \frac{Z \cdot I \cdot S_h}{2 \cdot R \cdot g} \qquad [-] \tag{2.4}$$

Where additionally:

- A_H is the design horizontal seismic coefficient
- R is the response reduction factor
- S_h/g is the average response acceleration factor

The base shear in addition to other loads are generally applied to the structure modelled in a FE-program for analysis of the result.

In earthquake resistance design the inertia force caused by the earthquake acceleration in the vertical direction is also considered in some cases. The self weight of the structure is then multiplied by the design vertical seismic coefficient, often considerably smaller than the design horizontal seismic coefficient (Hamada 2014).

2.5.2.2 Modal analysis

Modal analysis is generally required for buildings between 40 and 90 meters in height and buildings which do not follow the requirements defined for the use of simplified methods. Modal analysis is based on an averaged design response acceleration spectra and a multiplying factor (Bangash 2011). In Nepal National Building code, NBC 105, the method is called Modal Response Spectrum Method (Government of Nepal 1994*d*).

With the use of Modal Response Spectrum Analysis, multi-degree of freedom (MDOF) systems can be modelled in a FE-program and a total response can be obtained.

Initially the mode shapes and frequencies are extracted and the spectral forces and displacements are found for each mode. By the use of a combination method, such as the square root of the sum of the squares (SRSS) or the complete quadratic combination (CQC), the combined forces are found. To get the design forces used in verification the combined forces are divided by a response modification factor.

The number of modes that needs to be included for a given structure is specified in the building code although a general requirement is that the number of modes should be sufficient to obtain 90 % of the mass participating in each direction, considering the total mass of the structure (Armouti 2015, Government of Nepal 1994*d*).

2.5.2.3 Detailed dynamic analysis

For buildings taller than 90 meters, with unusual configuration or of special importance, modal analysis is no longer appropriate and instead a detailed dynamic analysis is required by most building codes. By the use of actual earthquake accelerogram and a time-wise integration the dynamic, non-linear response of structures can be obtained. Detailed dynamic analysis is generally carried out by the help of modern FE-programs and constitutes the most rigorous method to handle seismic load. Consequently detailed dynamic analysis also provides the most accurate and reliable result (Bangash 2011).

2.5.3 Nepalese Building Code

Until 1994 Nepal did not have its own national regulations or documents setting out requirements or guidelines for achieving buildings with satisfactory strength. In 1988 a request of technical assistance came from the Ministry of Housing and Physical Planning, MHPP, to the United Nations Centre for Human Settlements, UNCHS, which is an executing agency of the United Nations Development Programme. A programme of Policy and Technical Support was initiated and in 1991 the MHPP and UNCHS requested proposals for a National Building Code from international organisations. The Nepal National Building Codes was prepared during 1993 and finalised in 1994, largely based on the Indian Standards (Government of Nepal 1994*d*).

Regarding Build up Nepal's and other organisations work in rural Nepal with CSEB, the most relevant documents within the Nepal National Building Codes provide requirements for building with masonry and guidelines on how to build with regards to earthquake load. NBC 202, Mandatory Rules of Thumb Load Bearing Masonry (Government of Nepal 1994*c*), provide requirements for building with reinforced masonry. NBC 105, Seismic Design for Buildings in Nepal (Government of Nepal 1994*d*), and NBC 204, Guidelines for Earthquake Resistant Building Construction: Earthen Buildings (Government of Nepal 1994*a*), provides guidelines for design of earthquake resistant buildings in general respectively design of earthquake resistant buildings with earthen materials.

There is a lack of relevant national documents providing guidelines or requirements on building specifically with CSEB. Instead documents such as Auroville Earth

Institute's Earthquake Resistant Buildings with Hollow Interlocking Blocks (Maïni 2005) are used and each individual design of a CSEB-building has to be approved by the Government of Nepal to legally be classified as a earthquake resistant building similar to the designs in the Design Catalogue for Residential Buildings and Schools (Build up Nepal 2017*a*) created by UK Aid, Build up Nepal and Practical Aid.

2.5.3.1 Seismic design of buildings in Nepal

Nepal National Building Code, NBC 105 Seismic design of buildings in Nepal, from 1994 offers two methods to handle seismic loading. For buildings up to 40 meters in height the Seismic Coefficient Method should be used and for all other buildings The Modal Response Spectra Method should be used (Government of Nepal 1994*d*).

2.5.3.2 Parameters for seismic design

The seismic weight, W_i, considered in seismic analysis is the sum of the dead load and the seismic live load. The seismic load should be considered at each level where the seismic live load is a percentage of the design live load tabulated in the code (Government of Nepal 1994*d*).

The periods of vibration, T_i, should be obtained by reliable recorded data or by calculation according to appropriate equation provided in the code (Government of Nepal 1994*d*).

Other necessary coefficients and spectra are found in the code, obtained by relevant equations, tables or graphs. The coefficients and spectra are the design horizontal seismic coefficient C_d, design spectrum $C_d(T_i)$, basic seismic coefficient C, basic response spectrum $C(T_i)$, site subsoil category, importance factor I, structural performance factor K, design eccentricity e_d and seismic zoning factor Z (Government of Nepal 1994*d*).

2.5.3.3 Seismic design forces

When designing for seismic action all elements and components in the structure should be designed for seismic forces, F_p, in any direction. F_p is determined by Equation 2.5 where P is the structure response factor and K_p is the component seismic performance factor. The structure response factor is determined by Equation 2.6 and reflects how the ground motion is distributed and amplified by the supporting structure of the component in question. The component seismic performance factor is obtained from tabulated values and relates to the importance and performance demands of the component in question during and after the earthquake (Government of Nepal 1994*d*).

$$F_p = C_p \cdot P \cdot K_p \cdot W_p \qquad [N] \tag{2.5}$$

$$P = 1 + \frac{h_i}{H} \tag{2.6}$$

2.5.3.4 Seismic Coefficient Method

When adopting the Seismic Coefficient Method, described in Section 2.5.2.1, according to NBC 105 the horizontal seismic base shear force and the corresponding horizontal seismic forces in the directions considered should be calculated. The horizontal seismic base shear, V_B, is calculated according to Equation 2.1 and the horizontal seismic forces applied on the different levels are calculated according to Equation 2.7 (Government of Nepal 1994*d*).

$$F_i = \frac{V \cdot W_i \cdot h_i}{\sum W_i \cdot h_i} \qquad [N] \tag{2.7}$$

2.5.3.5 Modal Response Method

Initially when adopting the Modal Response Method, described in Section 2.5.2.2, the design spectrum, $C_d(T_i)$, has do be decided according to the Equation 2.8. $C_d(T_i)$ is used to obtain the relative response of each mode contributing by multiplying the factor with the mode response (Government of Nepal 1994*d*).

$$C_d(T_i) = C(T_i) \cdot Z \cdot I \cdot K \tag{2.8}$$

The combination method chosen should take the effect of closely spaced modes into consideration if their frequencies are less than 15% apart. The resulting modal effect should additionally be scaled by the modal combination factor S obtained by Equation 2.9 (Government of Nepal 1994*d*).

$$S = \frac{0.9 \cdot C_d \cdot W_t}{\sum comb.\ modal\ base\ shears\ in\ direction\ under\ consideration > 1.0} \tag{2.9}$$

Torsional effects might also be taken into consideration by the use of a static method. If the eccentricity, e_c, is greater than 0.3, three-dimensional analysis has to be performed for evaluation of the torsional effects (Government of Nepal 1994*d*).

2.5.3.6 Performance demands of building under seismic load

By the use of one of the methods included in NBC 105, forces or design spectrum are obtained that can be used to calculate deformations. The design lateral forces should be taken as the deformations multiplied by a factor of $5/K$. The inter-story deflection may not exceed 0.010 times the story height or 60 mm whichever is greater (Government of Nepal 1994*d*).

Complementing the NBC 105 is the Indian Standard, IS 1893:2002. The acceptable story drift for any story according to the IS 1893 is maximum 0.4 % of the story height when calculated with the minimum design lateral force (Earthquake Engineering Sectional Committee 2002).

2.5.3.7 Guidelines for Earthquake Resistant Building Construction: EB

Within the National Building Code Development Project in 1993, guidelines were prepared in a series of documents to raise the seismic safety in the country. NBC 204, Guidelines for Earthquake Resistant Building Construction: EB, was one of those documents. In NBC 204 basic guidelines for design and construction of buildings with earthen materials are provided. The recommendations are considered mandatory for all public earthen buildings and all residential earthen buildings build in areas where building permit processes exist. For residential earthen buildings build in rural areas the recommendations are considered advisory (Government of Nepal 1994*a*).

2.6 FE-modelling

In this section the approaches of modelling CSEB buildings will be discussed. Since the research of modelling CSEB buildings specifically has not come far the similarity between masonry buildings and CSEB buildings will be considered, see Section 2.3.5.

2.6.1 FE-modelling of masonry building

The interest in developing numerical approaches to analyse the response of masonry structures under loading has during the last years increased and led to a development within the field (Angelillo 2014). Today there are a number of sophisticated modern numerical tools to model and analyse masonry structures. The most used approach to simulate the structural behaviour of masonry today consist of non-linear models implemented in appropriate FE-programs.

The modelling of masonry compared to other materials such as concrete and steel is relatively complex due to the material complexity and structural composition. Masonry respond strongly non-linear to loading and is always build in combination with other materials such as steel, mortar or concrete making it a heterogeneous material. Therefore masonry often require a model with 2D or 3D elements and a non-linear approach gives the most realistic results whereas a linear elastic approach gives approximate results. The largest difficulty when modelling masonry is the definition and use of appropriate material constitutive laws.

Due to the complexity of the material composition of masonry several techniques have been suggested based on the two approaches, micro-modelling and macro-modelling.

2.6.1.1 Macro modelling

With a macro modelling approach the constitutive laws are phenomenological, meaning that they are based on experimental values from tests where the difference in material properties between the masonry and mortar is not distinguished but used as combined values.

A macro approach has its advantage in the computational efficiency but is most often unable to record the detailed damage evolution occurring in the material.

2.6.1.2 Micro modelling

With a micro modelling approach the materials are modelled individually with their own constitutive laws obtained from experimental tests performed on the materials separately. The micro approach provides a more detailed record of the response but requires significantly more computational effort.

2.6.1.3 Micro-Macro modelling

Yet another approach exist where the individual constitutive laws for the masonry and the mortar is used and in a homogenisation procedure combined to a macro-model for the masonry. This provides a method, that in a realistic way uses constitutive laws based on both materials mechanical properties, but where the computational efficiency of macro-modelling can be utilised.

3 Method

3.1 Error Bank

As a initial step towards an analyse of the actual resistance of Build up Nepal's approved CSEB-buildings, see the catalogue for approved designs by Build up Nepal (2017*a*), the errors that occur during production and construction had to be compiled. A list of the errors, sorted according to the location of occurrence, can be seen in Table 4.3 in a so called Error Bank.

The Error Bank have been used as a basis to analyse the actual resistance of the CSEB-buildings and can constitute a foundation for Build up Nepal to further develop and use in their work to avoid errors being made.

The Error Bank has been compiled by interviewing Build up Nepal's employed engineers and architects in addition to site visits to a few selected villages where Build up Nepal are active. Pictures has been collected from Build up Nepal's already existing picture bank as well as from pictures taken during site visits. A comparative study of Build up Nepal's existing picture bank and approved drawings was also undertaken to identify different construction errors. Pictures describing the errors has been compiled and can be found in Appendix B.

As a second step in the analysis of the impact on the resistance from the errors in the CSEB-buildings a literature study was carried out. Complementing the literature study, a FE-analysis was performed on strategically selected errors further described in Section 3.2.1. The results from the FE-analysis can be seen in Section 4.3.

The errors concerning the foundation have been excluded from this study but can be seen listed in the Error Bank in Table 4.3.

3.2 Finite element analysis

A linear-elastic approach was chosen for the FE-analysis and SAP2000 v20.0.0 was chosen as the software to be used. SAP2000 was chosen due to the simplicity to model entire buildings, the speed of analysis and since Build up Nepal's structural engineers have used SAP2000 in their previous structural analysis. This simplifies the possibility for comparison as well as the possibility to make several faster analysis to get an overall estimation of the impact of different errors.

Initially a FE-model was created based on a micro-macro approach as described in Section 2.6.1.3, modelled according to drawings by Build up Nepal (2017*a*). This model will be referred to as the ideal model in this report. Further the strategically selected errors were modelled based on the ideal model.

3.2.1 FE-model

The ideal model was modelled according to design 2 in the catalogue of approved designs by Build up Nepal (2017*a*), as can be seen in Figure 3.1, with the height of 3.472 m. This design was chosen since several of the errors compiled in the Error Bank has been seen in this design and because it is one of the most popular designs being built.

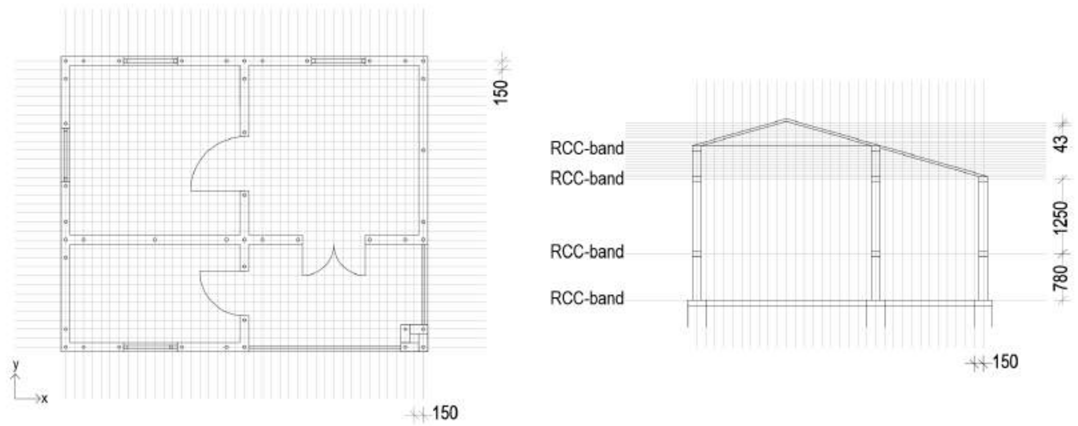


Figure 3.1: Design as base for SAP model. Design from Build up Nepal (2017*a*). Not scaled.

3.2.1.1 Material properties

The model was created with six types of sections, five frame sections and one area section. The sections were based on four material models, concrete, masonry (CSEB/mortar), reinforcement steel and structural steel. The material properties can be seen in Appendix D and were based on information from the catalogue of approved designs by Build up Nepal (2017*a*).

According to SS EN 1996-1-1 (Murverk och Puts, SIS/TK 180 2005), the Swedish version of EN 1996-1-1:2005 (European Committee for Standardization 2005), it is important to take into account that masonry is an assembly of both masonry units and mortar. Therefore both material's structural properties should be taken into consideration when verifying the load bearing capacity of the structural elements. Experimental data for achieving a weighted characteristic compressive strength for CSEB masonry does not exist in any national building code. In IS 1905:1987 (Earthquake Engineering Sectional Committee 1989) there are tabulated values for weighted compressive strength of masonry structures as well as suggested values for the longitudinal and vertical tensile strength. In IS 1905:1987 there is no method provided to adjust the modulus of elasticity according to the compressive strength and therefore the decision was made to follow the guidelines in SS EN 1996-1-1, explained by Molnár & Gustavsson (2016), to achieve the combined characteristic compressive and tensile strengths of the CSEB-masonry as well as adjusted modulus of elasticity. The material properties for CSEB were taken from previous structural analysis made by Build up Nepal (Build up Nepal 2017*b*) as well as from the Master's Thesis, Compressed Stabilised Earth Blocks in Nepal, by Herman Mellergård and Axel Steinert (2016), as can be seen in Appendix D. The type of mortar was taken from the catalogue of approved designs by Build up Nepal (2017*a*).

According to Molnár & Gustavsson (2016) the weighted value for masonry and mortar can be taken from tabulated data presented in EKS 10 (Boverket 2016). Due to the resemblance of CSEB and lightweight aggregate blocks (lättklinkerblock in EKS 10) they were chosen to represent the CSEB. According to the catalogue of approved designs by Build up Nepal (2017*a*) the type of mortar should be 1:5 sand-cement-mortar. Therefore, for the weighing of the masonry the mortar type M2.5 was choose due to the same percentage of material content. This gives a characteristic compressive strength of 3.4 MPa as can be seen in Appendix D.

Following the weighing of the compressive strength of the CSEB and mortar a new value for the elastic modulus was obtained, see Appendix H, and used in the FE-model, see Appendix D.

Equation 3.1 is presented in SS EN 1996-1-1 (Murverk och Puts, SIS/TK 180 2005) and constitutes the correlation between the modulus of elasticity and compressive strength of masonry where K_E is a constant depending on masonry unit groups. Since the modulus of elasticity and compressive strength for the CSEB did not match any value of K_E, tabulated in SS EN 1996-1-1, a new value for K_E was initially obtained by the use of the material properties for CSEB. K_E was then used to get the modulus of elasticity for the masonry, $E_{masonry}$, relating to the weighted compressive strength of 3.4 MPa.

$$E_{masonry} = K_E \cdot f_k \qquad [kN] \tag{3.1}$$

For the tensile strength of the CSEB the values presented in both Maïni (2005) and Shrestha (2012) varies between 0.5-1 MPa. For the FE-model the tensile strength was weighted with the mortar analogous to the weighing of the compressive strength by tabulated values from EKS 10 (Boverket 2016). The calculations can be seen in Appendix H and the resulting characteristic tensile strengths or characteristic bending tensile strengths are $f_{xk1} = 0.15$ MPa respectively $f_{xk2} = 0.3$ MPa depending on the type of failure mode, see Figure 3.2.

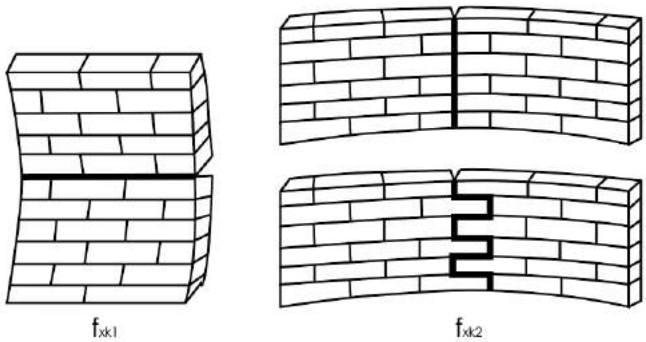


Figure 3.2: Illustration showing the different failure modes due to bending of masonry, based on (Molnár & Gustavsson 2016)

3.2.1.2 Sections

The area section for CSEB walls was defined as a thin shell with a membrane and bending thickness of 0.15 m and consisting of the material model Masonry, see Appendix D.

The RCC bands were defined as a frame section with the depth of 0.1 m and width of 0.15 m made by the concrete material model. A limitation of SAP2000 is that frame sections has to be modelled with a minimum of four reinforcement bars. According to Build up Nepal (2017*a*) the RCC bands should be constructed with two reinforcement bars with a diameter of 10 mm according to Figure 3.3. To compensate for this, four reinforcement bars were modelled with a diameter of 7.071 mm giving an equal total reinforcement area. The confinement bars were modelled with a diameter of 8 mm and a spacing of 150 mm.

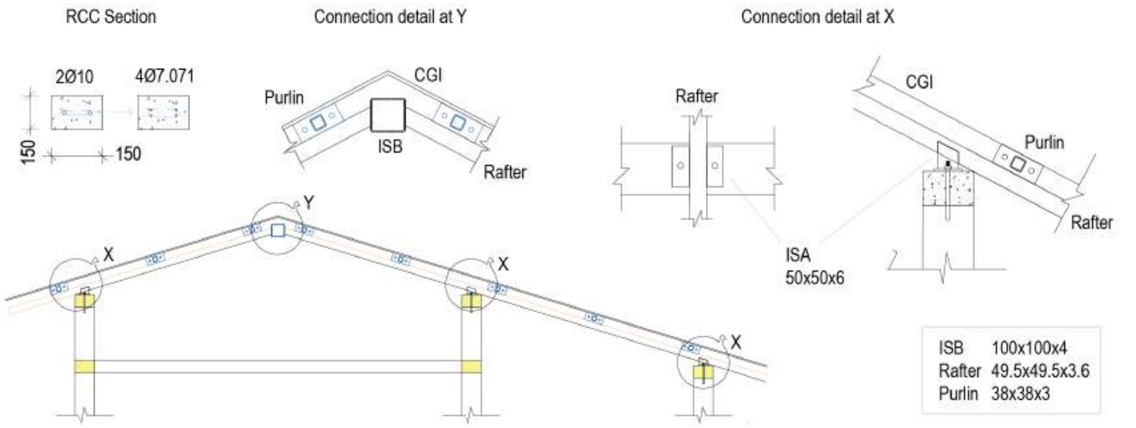


Figure 3.3: Details as base for SAP model. Design from Build up Nepal (2017*a*). Not scaled.

The top roof beam, rafters and purlins were modelled as steel frame sections made by the structural steel model. The dimensions were according to the catalogue of approved designs by Build up Nepal (2017*a*) as can be seen in Figure 3.3.

The CSEB have holes with a diameter of 50 mm and a spacing of 150 mm that are suppose to be grouted with or without reinforcement according to Build up Nepal (2017*a*). The groutings were viewed as unreinforced or reinforced slim concrete columns in the FE model. The reinforced concrete columns were modelled as concrete frame sections made by the concrete material model. The same limitation as described in relation to the modelling of RCC bands applies to the reinforced concrete column as well. In this case one 10 mm diameter reinforcement bar was replaced with four 2.5 mm diameter reinforcement bars. The confinement bars were modelled as 6 mm diameter bars with a spacing of 500 mm.

The unreinforced grouting was modelled using a free form sectional design made by the material model concrete. This since concrete columns can not be modelled without reinforcement.

The CSEB walls were modelled between the RCC bands and meshed with the dimension of the CSEB, 0.15x0.10 mm, as can be seen in Figure 3.1. The grouted infill sections were constrained to the shell sections to enable a uniform movement. The roof structure consisting of a roof beam, rafters and purlins were modelled at a distance from the gable RCC band to avoid uniform movement with the wall and represent the connections to the wall according to Figure 3.3. The connections between the purlins, rafters, roof beam and connections to walls were modelled as to move and rotate uniformly within each connection. Furthermore, the model was supported with hinged supports along the base of all walls.

3.2.1.3 Verification of FE-model

The model was verified with simple beam analysis comparing analytical calculation, see Appendix E, with beams modelled in SAP2000 in the same way as the CSEB-building was modelled. The beams were modelled with a length of 2 m, a depth of 0.1 m and a height of 0.15 m relating to a CSEB-wall where the bricks are 0.15 m in depth. The beams were modelled as shell elements with Masonry properties and meshed to 20x2x1 finite elements. The beams were simply supported. The reinforced beam, verifying the sections of the CSEB-wall where the blocks were grouted, was additionally modelled with a 10 mm diameter reinforcement bar in the middle of the beam.

The deflections from the FE analysis and from analytical analysis was compared to verify the behaviour of the FE model. The deflections extracted from the FE model and concluded from the analytical analysis can be seen in Table 3.1. The small decrease in deflection with the reinforced beam comes from the reinforcement bar not being infinitely small and therefore slightly contributing to the stiffness of the beam.

Since a linear analysis was performed, an analysis of an uncracked beam, all of the load has to be carried by the beam, hence only analytical calculations were performed in the first stage to verify the model.

Table 3.1: Verification

Model	Analytic calculation	FE model
Unreinforced	105.82 mm	105.578 mm
Reinforced	105.82 mm	105.054 mm

This verifies that the model is modelled accurately since the analytical and numerical analysis gave similar results in both cases. The reinforcement should not give impact on the deflection in a linear analysis since it is first when the beam crack that the reinforcements tensile capacity can contribute.

3.2.2 Loads

Nepal National Building Codes are based on the Indian Standards, see Section 2.5.3. The Indian Standards was generally followed for calculations of loads in this study due to the more detailed descriptions of approaches.

Dead load, load from CGI on roof, wind load in x- and y-direction for walls and roof and imposed loads were calculated by analytical calculation as can be seen in Appendix I. The applied earthquake loads are explained in Section 3.2.2.1

3.2.2.1 Earthquake loads

To calculate the earthquake load IS 1893:200 The base shear inflicted on the building due to earthquake excitation was calculated in x- and y-direction and was divided according to the mass between twenty parts of the building, over 3 levels relating to sill, lintel and roof band as can be seen in Figure 3.4. The mass was divided to get an as realistic result as possible using a linear, static FE-analysis. The analytical analysis to obtain the earthquake loads can be seen in Appendix I.

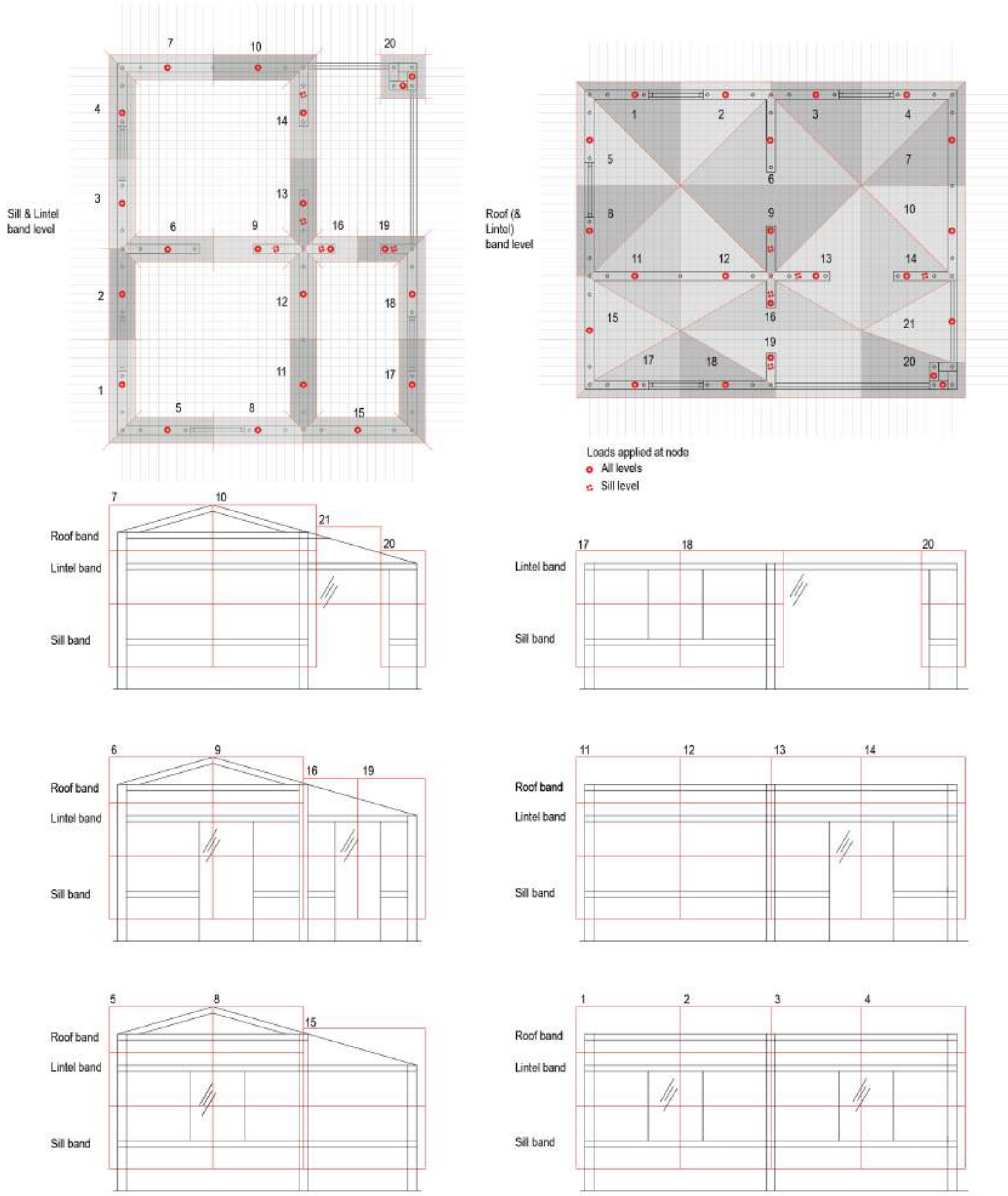


Figure 3.4: Divide of total mass for earthquake load

The total seismic weight of the building is 228.328 kN. This is the weight used to calculate the base shear, according to Equation 2.3. The fifty-five earthquake loads applied to the model and presented in Appendix I were obtained by Equation 3.2 from IS 1893 (Earthquake Engineering Sectional Committee 2002), corresponds to Equation 2.7 from NBC 105 (Government of Nepal 1994*d*).

$$Q = \frac{V_B \cdot W_i \cdot h_i^2}{\sum W_j \cdot h_j^2} \qquad [N] \tag{3.2}$$

Where:

V_B is the total base shear [N]

W_i is the weight of a floor [N]

h_i is the height from base to floor level [m]

j is the number of floors

The dead weight obtained from the reaction force from dead load in the SAP2000 model was, as a comparison, 232.071 kN.

The fifty-five point loads in x- and y-direction were applied to the model at sill, lintel and roof band level at the points marked with red in Figure 3.4.

3.2.2.2 Load combinations

Load cases that were taken into account according to NBC 105:1994 (Government of Nepal 1994*d*) can be seen in Table 3.2 1-12. Both load cased for Working Stress Method and Limit State Method were considered. These are the states considered in the Nepalese National Building Codes like serviceability limit state (SLS) and ultimate limit state (ULS) are considered in the Eurocodes.

Load cases 1-12 will in the analysis be referred to with an *a* or *b* after, where an *a* refers to the earthquake load having a positive sign in the load combination and *b* to the earthquake load having a negative sign in the load combination.

According to NBC 105:1994 (Government of Nepal 1994*d*) wind loads should not be included in load combinations with seismic load. Analysis taking wind loads into consideration, see Table 3.2 13-14, have additionally been performed according to IS 1905:1987 (Earthquake Engineering Sectional Committee 1989) and the commentary document of IS 1905:1987 by Dr. Durgesh C. Rai (n.d.), see Appendix I. The wind loads from calculations showed to be outwards on the walls and uplifting on the roof.

Table 3.2: Load combinations

nr.	**DL**	**WL**$_x$	**WL**$_y$	**EL**$_x$	**EL**$_y$	**IL**
Working Stress Method						
1	1			±1		1
2	0.7			±1		
3	1			±1		
4	1				±1	1
5	0.7				±1	
6	1				±1	
Limit State Method						
7	1			±1.25		1.3
8	0.9			±1.25		
9	1			±1.25		
10	1				±1.25	1.3
11	0.9				±1.25	
12	1				±1.25	
Allowable Stress Design Method						
13	1					1
14	1	1				1
15	1		1			1
16	1	1				
17	1		1			

3.2.2.3 Verification of application of earthquake loads

There are a few approaches generally accepted for the application of earthquake loads depending on structure and outer circumstances.
For serviceable limit state the accepted drift is 0.004 according to IS 1893:2002 (Earthquake Engineering Sectional Committee 2002) when the total base shear is divided between the floor levels and then distributed between the lateral resisting elements. Since the resulting stresses also were intended to be used in the analysis and a few point loads could be assumed to give stress concentrations at their application points, a verification study on the impact of different approaches of application of seismic loads was conducted. Three different approaches where tested and the displacements and deformations were compared. The different approaches tested were applying the equivalent base shear as; point loads where shear walls exists to resist movement (illustrated in Figure 3.5), as equivalent line loads over the roof RCC band (illustrated in Figure 3.7) and finally divided into point loads applied in mass centre of sub-domains contributing to the building's seismic weight, as explained in Section 3.2.2.1.

In the first verification analysis where the loads were applied parallel to the shear walls, see Figure 3.5, the maximum displacement due to seismic load where measure to be 0.35 mm at point 3, see Figure 3.10, and the displacement at the top of the gabble wall was 0.21 mm, which can be seen in Table 3.4 and 3.3 respectively. The deformations of the building due to the application approach for earthquake loads was considered not to be realistic since it is known that that the seismic force will affect the whole buildings deformations and stresses and not just at the location of the shear walls, see Figure 3.6 for a deformation graph.

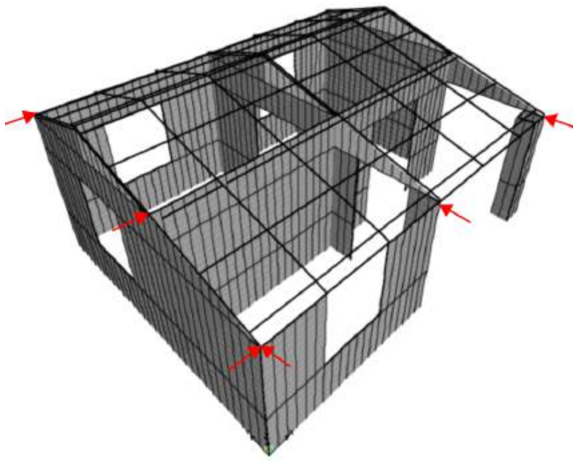


Figure 3.5: An illustration of how the loads were applied in verification model 1

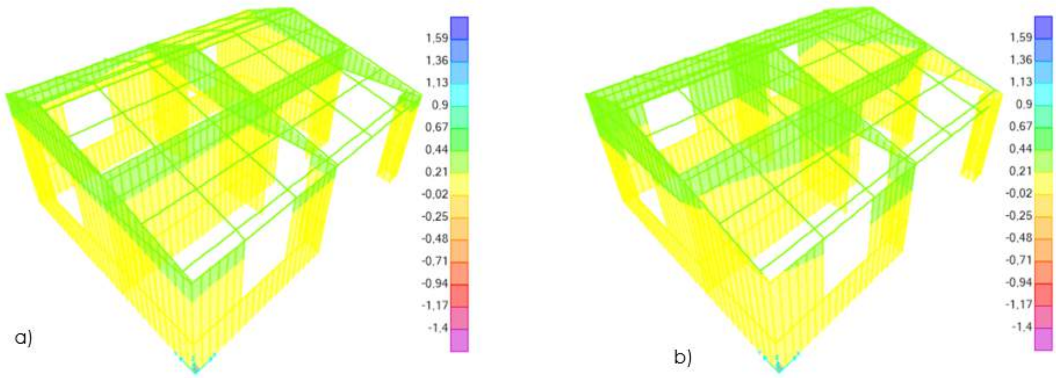


Figure 3.6: The displacements found in SAP2000 in verification model 1 with a scale factor of 200 and unit [N/mm^2] **a)** The displacements from the seismic load in x-direction **b)** the displacements from the seismic load in y-direction

For the second analysis the earthquake load was applied as equivalent line loads over the roof RCC band, see Figure 3.7. The maximum displacement was measured to be 1.59 mm at the top of the gable wall, point 1 in Figure 3.10, according to Table 3.3 and 3.4. The deformations of the building due to the applied earthquake load were not considered sufficiently reliable due to the load concentrations at the RCC band, see Figure 3.8.

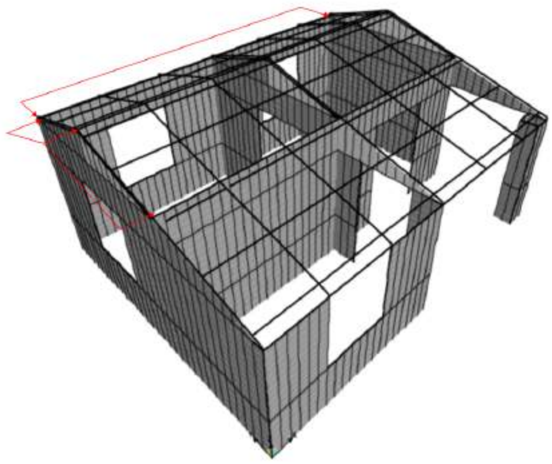


Figure 3.7: An illustration of how the loads where applied in verification model 2

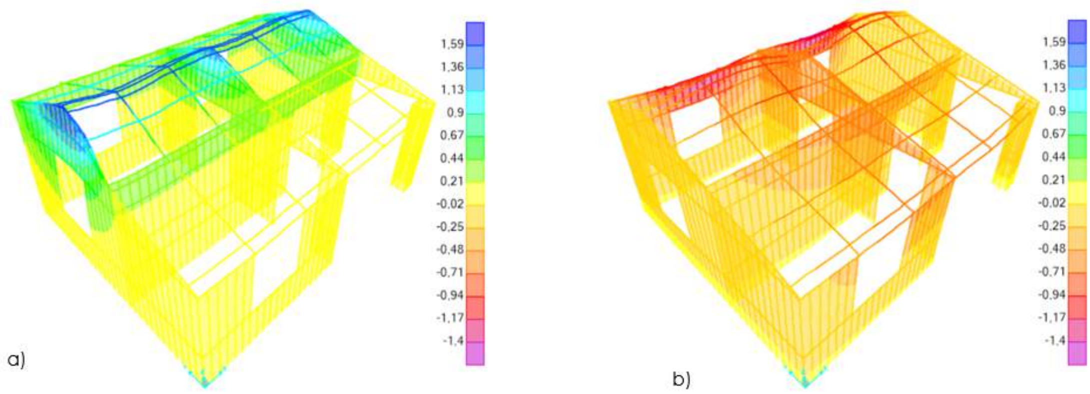


Figure 3.8: The displacements found in SAP2000 in Verification model 2 with a scale factor of 200 and unit [N/mm^2] **a)** The displacements from the seismic load in x-direction **b)** the displacements from the seismic load in y-direction

In the FE-model referred to as the ideal model the earthquake load was divided between different areas of the building over three levels relating to the RCC bands, as explained in Section 3.2.2.1. The maximum displacement was then measured to be 0.54 mm at point 5, see Figure 3.10, and the displacement on top of the gable wall was found to be 0.48 mm, as can be seen in Table 3.4 and 3.3 respectively. This application approach was expected to give the most realistic results since in reality each molecule that contribute to the building's mass will accelerate due to the seismic movement and induce the earthquake load. The displacements can be seen in Figure 3.9.

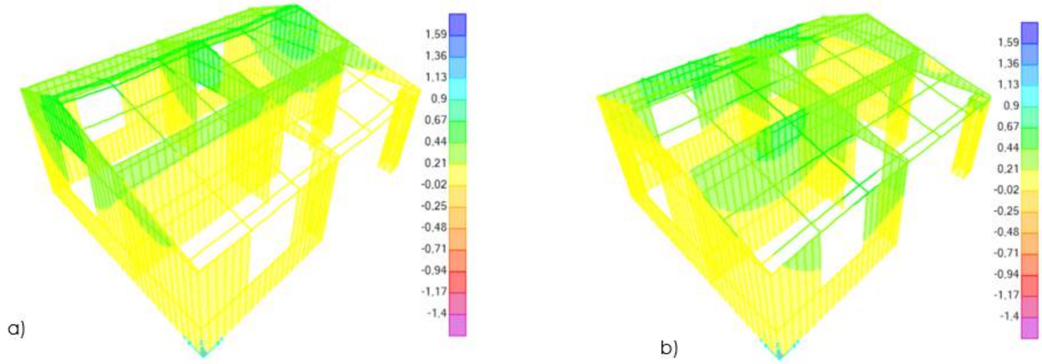


Figure 3.9: The displacements found in SAP2000 in the final model with a scale factor of 200 and unit [N/mm^2] **a)** The displacements from the seismic load in x-direction **b)** the displacements from the seismic load in y-direction

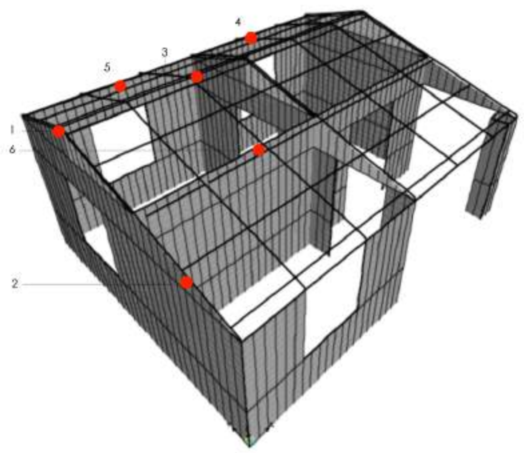


Figure 3.10: The location of the displacements measured in Table 3.3 and 3.4

Table 3.3: Measured displacements at the top of the gable wall in verification models

Analysis	Location	Load	δ_x	Load	δ_y
Resilient	1	EQ_x	1.6 mm	EQ_y	0.95 mm
Model 1	1	EQ_x	0.21 mm	EQ_y	0.22 mm
Model 2	1	EQ_x	1.59 mm	EQ_y	-0.18 mm
Model 3	1	EQ_x	0.48 mm	EQ_y	0.20 mm

Table 3.4: Measured maximum displacements in verification models

Analysis	Location	Load	Max δ_x	Location	Load	Max δ_y
Model 1	2	EQ_x	0.32 mm	3	EQ_y	0.35 mm
Model 2	1	EQ_x	1.59 mm	4	EQ_y	-1.40 mm
Model 3	5	EQ_x	0.54 mm	5	EQ_y	0.52 mm

3.2.3 Analysis of result

Initially the ideal model was analysed in terms of horizontal displacement and local stresses. The maximum displacement was extracted from point 1 as shown in Figure 3.10, the top point of one of the gable walls. The maximum displacement of the whole structure was also extracted. These displacements were considered based on all load cases shown in Table 3.2. The displacements of the ideal model were used as the reference for the analysis of the impact of errors. The choice of applications of earthquake load does not comply with IS 1893:2002 (Earthquake Engineering Sectional Committee 2002) and therefore the allowed drift, 0.004, might not longer be relevant but was still used as a reference value.

The stresses that were considered were the stresses extracted from the FE-program as can be seen described in Table 3.5. Depending on the error modelled, different local stresses at different locations were considered and compared to the ideal model.

Table 3.5: Stresses for analysis

Stress	**Description**
S11Top/Bot	Internal stress from bending in the direction of horisontal axis (local coordinates), at the top/bottom of specific point of area element location
S22Top/Bot	Internal stress from bending in the direction of vertical axis (local coordinates), at the top/bottom of specific point of area element location
S12Top/Bot	Internal shear stress, at the top/bottom of specific point of area element location
SMaxTop/Bot	Area maximum principle stress, at the top/bottom of specific point of area element location
SMinTop/Bot	Area minimum principle stress, at the top/bottom of specific point of area element location

For the analysis of the structure regarding compression, the extracted stresses were the principal stresses (SMaxTop, SMinTop, SMaxBot and SMinBot). They were compared with the compression strength of the masonry according to Table D.1 factored with the partial factor according to EN 1996-1-1:2005 (European Committee for Standardization 2005), as can be seen in Appendix I.

Analysis of the tensile stresses in the structure were based on the internal stresses in the local coordinates since the known tensile bending strengths, see Section 3.2.1.1, are parallel and perpendicular to the bed joint. Additionally, as described by Drysdale & Hamid (1982) the diagonal tensile strength is correlated to an average value of the two orthogonal tensile strengths and was therefore not the tensile strength of highest interest. The tensile strength named f_{xk1} in Section 3.2.1.1 was the comparison for stresses S22 and the tensile strength named f_{xk2} was the comparison for stresses S11. The tensile strengths were factored with the partial factor according to EN 1996-1-1:2005 (European Committee for Standardization 2005), as can be seen in Appendix I.

4

Results

4.1 Ideal FE-model

The results, in terms of displacements and local stresses, will be presented as described in Section 3.2.3.

The graphs illustrating the stresses and displacements will refer to the wall numbering in Figure 4.1.

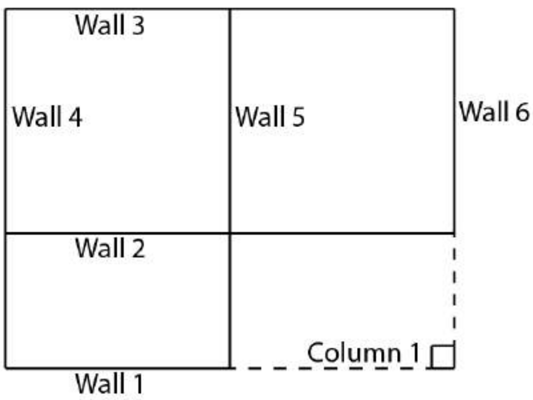


Figure 4.1: Numbering of wall in FE-model

4.1.1 Displacement

The displacements were checked for all load cases and graphs can be seen in Appendix J. The maximum displacement in the x-direction was reached under load case 16, as described in Table 3.2, and was measured to be -0.83 mm. The maximum displacement in the y-direction was reached under load case 10a and was measured to be 0.71 mm, see Table 4.1 and Figure 4.3. The location for the maximum displacements in x- and y-direction respectively can be seen in Figure 4.2.

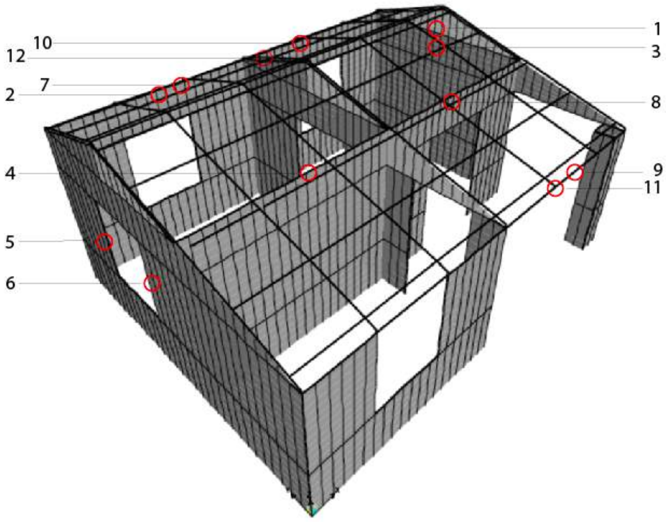


Figure 4.2: Illustration showing locations of maximum displacements.

Table 4.1: Measured maximum displacement from SAP2000, for location see Figure 4.2

	Displacement	Drift	Load case	Location
u_x	-0.83 mm	0.024%	16	5
u_y	0.71 mm	0.020%	10a	2
u_z	0.73 mm	0.021%	10a	9

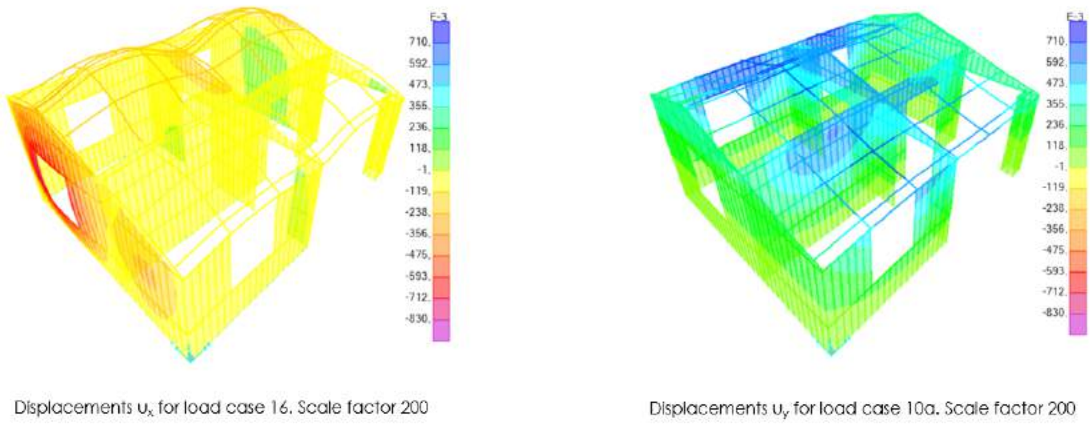


Figure 4.3: To the left: maximum displacements in x-direction, load case 16. To the right: maximum displacements in y-direction, load case 10a. Images from SAP2000, unit [mm] and scale factor 200.

4.1.2 Local stresses

For the defined load cases the largest tensile and compressive stresses were extracted for the top and bottom faces of the shell elements representing the masonry walls, as can be seen in Table 4.2. The locations of the maximum stresses can be seen in Figure 4.4.

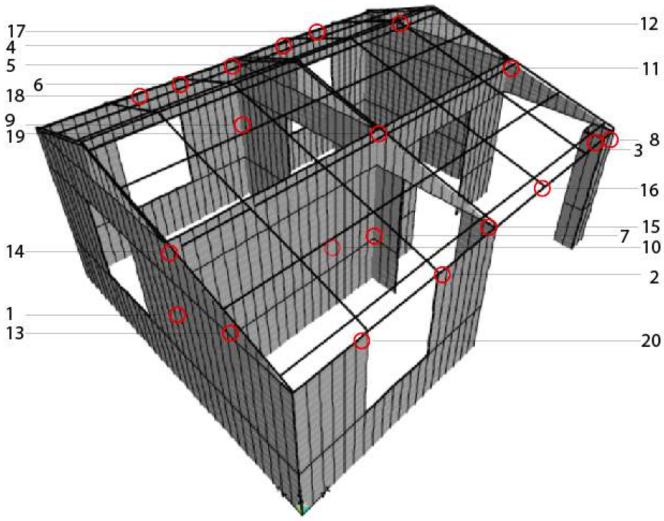


Figure 4.4: Illustration showing locations of maximum stresses.

Table 4.2: Measured maximum and minimum stresses from SAP2000, for locations see Figure 4.4

	Stress [N/mm^2]	Load case	Location
Top			
S11	0.093	16	1
S22	0.176	17	4
SMaxTop	-0.051	17	2
SMinTop	-0.129	17	3
Bottom			
S11	0.097	16	5
S22	0.165	17	3
SMaxBottom	-0.088	16	6
SMinBottom	-0.159	17	4

Figure 4.5 and 4.6 shows the walls where the largest stresses of the different types occurs.

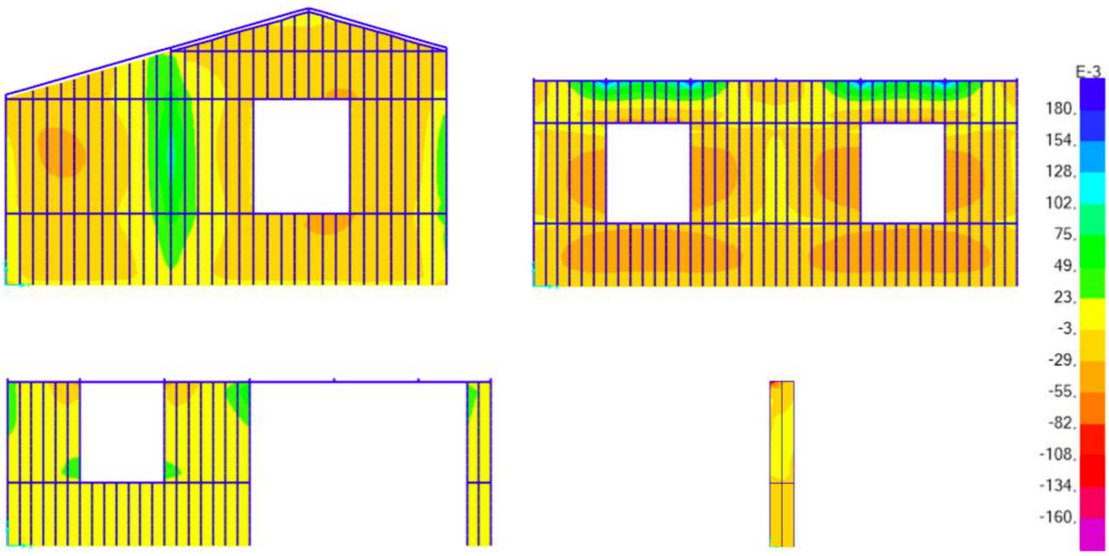


Figure 4.5: Absolute maximum stresses in top surface from Table 4.2 in [N/mm^2]. Start left top.

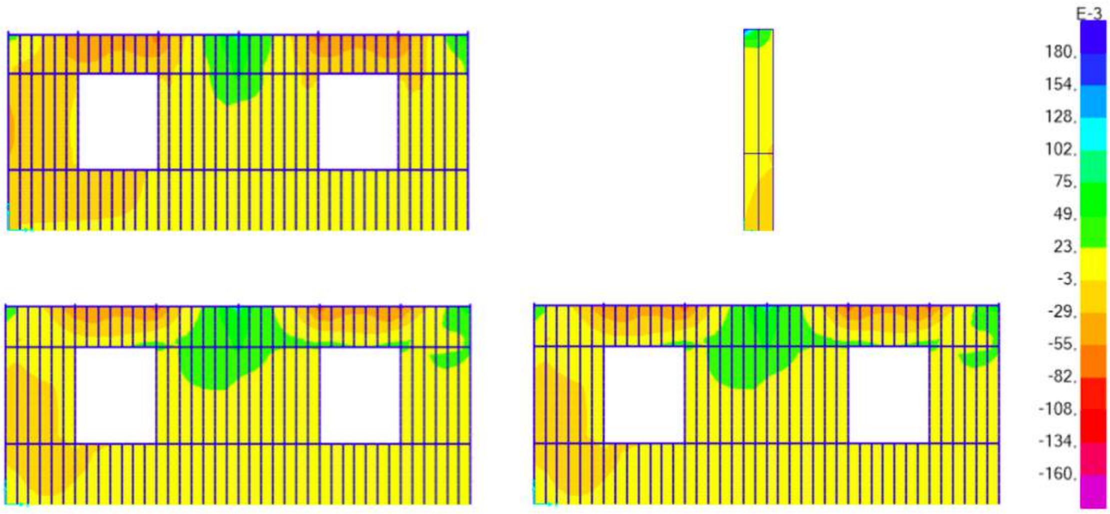


Figure 4.6: Absolute maximum stresses in bottom surface from Table 4.2 in [N/mm^2]. Start left top.

Table 4.2 shows that only stresses S22 exceeds the capacity of the CSEB wall in the ideal model.

4.2 Error Bank

In Table 4.3 is the Error Bank presented, based on Appendix A. Pictures from site in Nepal, describing the errors, can be seen in Appendix B.

Table 4.3: Error Bank

Error nr.	**Error**	**Villages**	**Design***	**Estimated Frequency (low-high)**
		WALLS		
W1	CSEB not levelled when build without mortar	Raleigh, Majhigaun	2	High
W2	CSEB not levelled when build with mortar	Majhigaun	2, 3	High
W3	Missing mortar/concrete in some vertical holes of the CSEB	-	-	Low
W4	Hairline crack trough wall	Raleigh	1, 4	Low
		REINFORCEMENT		
R1	Insufficient lap length of reinforcement	Jyamrung	-	Medium
R2	Field bending of reinforcement	Jyamrung, Bakhang, Raleigh	-	High
R3	All vertical reinforcement start at plinth band	-	-	Low
R4	Vertical reinforcement missing	Kalleri	-	Low
R5	Appropriate stirrup in RCC bands missing	Raleigh	-	Medium
R6	Insufficient radius of stirrup	Raleigh, GoodWeave	-	Medium
R7	Reinforcement exposed to environment (not bent into the top RCC band)	Jyamrung, Majhigaun, Raleigh	2, 3	High

R8	Too low cover thickness of reinforcement in RCC bands	Majhigaun	2	Medium
		OPENINGS		
O1	Openings larger than specified (windows and doors)	-	-	Medium
O2	Openings too close to corners	-	-	Medium
		RCC band		
C1	Too slender lintel RCC bands between corner column and walls in design 2*	Majhigaun	2	Medium
C2	RCC bands are made too thin	Raleigh, Majhigaun	2	Medium
C3	Too coarse aggregate in RCC bands	Raleigh	-	Medium
		FOUNDATION		
F1	Mud mortar is used in foundation instead of cement mortar	-	-	Low
F2	Through-stones missing in stone masonry foundation	-	-	Low
F3	Corner stone is missing in foundation	Kalleri	-	Low
F4	Foundation too shallow	Majhigaun	2	Low
		ROOF		
RO1	Roof is attached in the wrong way to the walls	Majhigaun, Mulabari	2, 3	High
RO2	Reinforcement bend around rafters and purlins	Majhigaun	2, 3	High
RO3	Reinforcement welded to rafters and purlins	Bakrang, GoodWeave, Majhigaun, Raleigh	2	Medium
RO4	Rafters missing	-	2	Medium
		OTHER		

X1	Free standing corner column in design 2	Jyamrung, Dadaguan, GoodWeave, Mulabari, Raleigh	2	High
X2	Gable wall made too high	-	2	Medium
X3	An extra floor is added to building with small bricks	-	-	Medium
X4	Damaged bricks are used	Dadaguan	-	Medium
X5	Roof band made all around building	Majhigaun	2	Medium
QUESTIONS				
Q1	How weak/strong are gable walls?	-	-	-
Q2	Impact of higher and lower compressive strength of the CSEB than specified?	-	-	-
Q3	Dry-stacking vs. use of mortar	-	-	-

* *Design numbers according to Design Catalogue by Build up Nepal (2017a).*

The villages referred to in Table 4.3 can be seen in Figure 4.7.

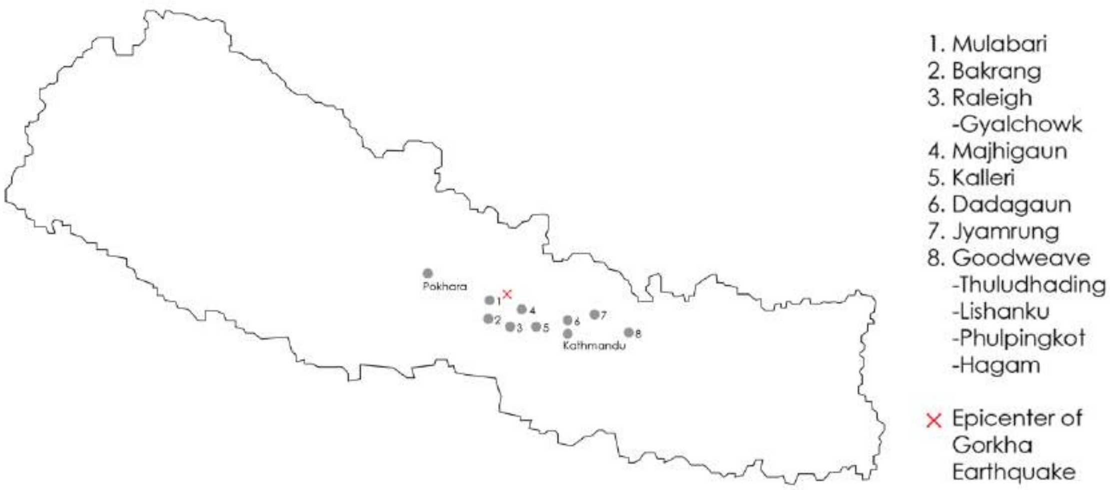


Figure 4.7: Villages where Build up Nepal is active.

4.3 Analysis of Errors

4.3.1 W1 - CSEB not levelled when build without mortar

When Build up Nepal started there work with CSEB the use of mortar was not a recommendation. Uneven layers of bricks have been detected and is thought to be the result of uneven surfaces of bricks or small stones or similar between the bricks.

According to (Maïni 2005) the bricks are not allowed to differ more than 1 mm in height which implies that the levelling of bricks in construction is of extreme importance. It is also stated the the linearity of the plinth band is of high importance since the linearity and quality of the walls depends on it.

According to the study on the impact of height imperfections of masonry blocks on the load bearing capacity of dry stacked masonry by Ngapeya, Waldmann & Scholzen (2018) it was shown that imperfections in height of the bricks strongly influence the load bearing capacity of a masonry wall. Ngapeya et al. created five micro-models analysing the stress distribution of a masonry units subjected to vertical load at the top. The first model was modelled with evenly distributed load over the whole top surface and support along the whole bottom surface. This analysis showed an evenly distributed stress in the brick. The second to fifth model were modelled with different load distributions at the top and different support areas at the bottom. These analysis showed non-uniform stress distributions in the brick units. Furthermore two models were created of masonry walls with different heights of bricks according to imperfection data from produced bricks. These models showed failure at 24 % respectively 5.4 % of the ultimate load of the same wall without imperfections. It can be seen in the analysis that stress concentrations occurred in the brick points over corners with a uplifted height imperfection causing an early crack pattern.

The height imperfections can in reality be due to a difference in height of the bricks or due to a roughness of the surface or dirt on the surface of a brick.

4.3.2 W2 - CSEB not levelled when build with mortar

Currently the use of mortar is recommended by Build up Nepal when building with CSEB. Uneven layers of CSEB have been detected in such buildings and is due to a difference in mortar thickness or uneven foundation.

As described in Section 4.3.1 the lack of mortar gives room for geometrical imperfections to affect the behaviour of a masonry structure. With the additional mortar between the layers of bricks these geometrical imperfections in height can be handled. Within this error uneven levels of CSEB has been seen with, consequently highly uneven layers of mortar. According to Maïni (2005) the mortar should be well laid and 5 mm in thickness and the entire surface of each brick should transfer the vertical load. As seen in the report by Ngapeya et al. (2018) non-consistent load transfer between bricks causes non-uniform stress concentrations.

For further study on dry-stacked and masonry with mortar structures see Section 4.3.27.

4.3.3 W3 - Missing mortar/concrete in some vertical holes of the CSEB

According to Maïni (2005) and Build up Nepal (2017*a*) the CSEB structures should be reinforced by grouting the holes with concrete and reinforcement bars at critical locations. Critical locations can be close to corners, around openings and at ends of walls. Holes without reinforcement should be grouted with only concrete.

The holes with reinforcement bars are grouted with concrete to achieve full interaction between the reinforcement bar and the bricks to ensure a monolithic behaviour of the wall under seismic load (Tomazevic 1999). This is what makes reinforced grouted masonry the most resistant masonry type to resist seismic load.

To further improve the earthquake resistance of a masonry structure the holes without reinforcement can be grouted as well according to Tomazevic (1999).

4.3.4 W4 - Hairline crack through wall

A vertical hairline crack has occurred in a few CSEB-buildings build both with and without mortar. The crack goes through the brick as well as the mortar but not through the RCC bands. The consequence of these hairline cracks occurring in villages is that no more CSEB-buildings are build there due to a lack of trust in the building technique. A few reasons for the emergence of the hairline cracks will be raised in Section 5.2.4.

According to Ngapeya et al. (2018), as described in Section 4.3.1, uneven layers of bricks in a masonry structure build without mortar might result in stress concentrations that leads to crack initiation already at about 6 % of the load capacity of the same masonry wall without height imperfections. Ngapeya et al. also showed that the highest stress concentrations were seen in areas of bricks over the corner of a underlying, uplifted brick which suggests that cracks will be initiated in these points.

It could also be assumed that cracks are initiated even with the use of mortar if the mortar layers or the foundation is uneven, see Section 4.3.2.

The importance of a stable foundations to prevent that ground settlement affect the structure is described in Maïni (2005). Settlement of the ground might lead to cracks due to non-uniformly distributed stress (Tomazevic 1999).

According to Adam & Agib (2001) micro cracking of a CSEB is tolerated on all faces of the bricks. However, for macro cracks the bricks should meet some requirements to still be of good quality, see Table 4.4. The detected cracks on site exceed the maximum tolerated measurements.

Table 4.4: Maximum tolerated measurements of macro-cracks (Adam & Agib 2001)

Macro-cracks	
Location	
Width	1 mm
Length	40 mm
Depth	10 mm
Number of faces	3

4.3.5 R1 - Insufficient lap length of reinforcement

An insufficient lap length of the vertical reinforcement has been detected as an error made during construction. The error is assumed to be a result of lack of knowledge.

A sufficient lap length of reinforcement means that the design loads can be transmitted according to Eurocode 6, EC6:1-1. The lap of reinforcing bars should not be located where high stresses occur nor where a sudden change in dimension of section size takes place, e.g. a step in wall thickness (European Committee for Standardization 2005).

In research done by Melander (1992) regarding how the width of the masonry unit, masonry type, diameter of reinforcing bar and lap length affect the strength of the lap splices in reinforced masonry, Melander found that mainly three failure mechanisms may occur around the splice length. There might occur a brittle tensile splitting between the grout and masonry unit, tensile splitting due to yield and/or pullout of the reinforcing bar or yield of the reinforcing bar which leads to failure due to pullout or fracture of the bar. Melander (1992) could show in his research that with a short lap length a brittle failure is prominent to occur and that the failure load were below the yield load of the reinforcement. The failure mechanism in specimen with short lap lengths would be mechanical interactions between the grout and deformations of the reinforcement. Circumferential tensile stresses would cause splitting cracks in the grout, see Figure 4.8.

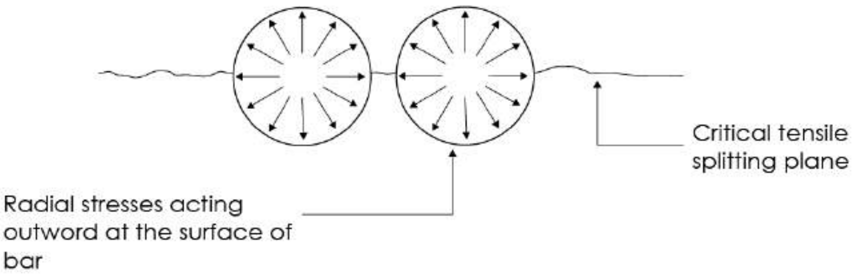


Figure 4.8: Illustration showing mechanical interaction forces and bond stresses based on (Melander 1992).

The research done on bond behaviour in reinforced concrete is greater than on reinforced masonry. However, Scrivener (1986) concludes in his research on bond strength and slip characteristics of deformed bars in grouted hollow masonry that the bond behaviour of reinforced masonry is very similar to reinforced concrete and that reasoning of reinforced concrete can be applied on reinforced masonry as well.

Reported by Abdel-Kareem, Abousafa & El-Hadidi (2013) in their experimental research on how the behaviour of lap splices are affected by transverse reinforcement in concrete beams they concluded that if the lap length was two-thirds or less of the required lap length the prominent failure mechanism was a brittle splitting with a significantly decreased ultimate load capacity.

Melander (1992) concludes that a longer lap length increases the strength of lap splice due to the fact that it decreases the nominal bond stresses along the reinforcement bar.

4.3.6 R2 - Field bending of reinforcement

A commonly occurring error that has been detected during the construction of CSEB-buildings is field bending of the reinforcement. The bending is often done directly over the plinth band to fit the reinforcement into the next layer of bricks. The reinforcement has been seen bend up to three times in one location.

Over-bending or cyclic bending of reinforcement can lead to metal fatigue which decreases the local strength of the steel and consequently weakens the earthquake resistance of a structure (Pook 2007).

In the American Standard Building Code ACI 318-99, reported by the American Concrete Institute (1999), the minimum allowed bend diameter and the approved methods for bending reinforcement bars is provided. The bend diameter of the reinforcement bars are restricted to avoid crushing on the inside of the bend and breakage on the outside of the bend. The allowed bend diameters are tabulated in the code limiting the diameter to be between six and ten times the nominal diameter of the bar, in inches, depending on the diameter of the reinforcement bar.

The ACI 318-99 states regarding bending of reinforcement that "All reinforcement shall be bent cold, unless otherwise permitted by the engineer", and "Reinforcement partially embedded in concrete shall not be field bend, except as shown on drawings or permitted by the engineer". Reinforcement with bends larger than the bend diameter permitted by the code can and shall be bend cold. If reinforcement is embedded bending is not allowed without authorisation by the responsible engineer who then will determine if hot or cold bending should be carried out.

Similar to what is stated in the ACI 318-99 the Australian Standard AS 3600-2001, published by the Standards Australia Committee BD-002 Concrete Structures (2001), states regarding bending of reinforcement that bending can be performed hot or cold as long as the minimum diameter of bend is complied with. Choice of method is decided depending on stated factors. Field bend can be performed if the factors regarding hot or cold bending and requirement for minimum bend diameter is followed and as long as no damages on the concrete embedding the reinforcements occurs. According to the The Australian Reinforcing Company (2008), ARC, the most critical factors when performing in-situ bending of reinforcement is the control of the bend diameter without damaging or impairing the steel and to avoid spalling of the concrete surrounding the reinforcement. According to the ARC, in-situ bending of reinforcement is rarely successfully and is often executed using non-approved methods.

4.3.7 R3 - All vertical reinforcement start at plinth band

In CSEB-buildings the vertical reinforcement should start from and be anchored in the foundation (Build up Nepal 2017*a*), see Figure 2.21. In some cases the foundation is cast before the decision to build a CSEB-buildings has been made. This means that the possibility of anchoring the reinforcement in the foundation is no longer possible and instead it is anchored in the plinth band. In the case where also the plinth band is cast another plinth band is cast on top of the first one and the reinforcement is anchored in the second band.

According to Maïni (2005) the general principle for good design of a CSEB-building should follow the motto "Good boots and a good hat". They explain this in regards of the foundation by a minimum of 90 cm deep foundation. They also emphasise the importance of well anchored vertical ties in the first reinforced concrete ring beam and comment on the not ideal site conditions often used.

In the event of an earthquake the lateral stiffness of all structural walls is a very important factor (Tomazevic 1999). The lateral stiffness depends on the material properties of the wall, the geometry and the boundary conditions and the base shear is distributed between the walls based on their individual stiffness.

According to IS 4236 (Earthquake Engineering Sectional Committee 2005) the main structural elements and the connections in between them should be designed to have ductile failure. The ductility of a reinforced masonry structure comes from the reinforcement bars and the reinforcement also provides strength and stability.

4.3.8 R4 - Vertical reinforcement missing

Due to, what is thought to be, a lack of knowledge and cost reasons vertical reinforcement is sometimes not used in all locations described in the drawings. In some of these cases the only vertical reinforcement used is the corner reinforcement.

In the study that Doğangün, Ural & Livaoğlu (2008) did for the 14^{th} World Conference on Earthquake Engineering they investigated the impact that recent earthquakes in Turkey had on different kinds of traditional masonry buildings. In their findings they could show that missing vertical confining elements around wall openings caused the wall to bend out into the opening and narrowed down the opening significantly. Lack of vertical reinforcing elements at corners caused the load bearing wall collapse.

According to Tomazevic (1999) adding reinforcement to a masonry structure increases the seismic resistance and energy dissipation capacity significantly. The reinforcement ensures that the structure works monolithic and therefore resists the lateral loads from an earthquake better. Reinforced grouted masonry is regarded the most earthquake resistant masonry technique.

4.3.9 R5 - Appropriate stirrup in RCC bands missing

An error that has been detected is that appropriate stirrups around the horizontal reinforcement is missing. What has been done is that short segments of straight reinforcement has been laid on top of the longitudinal reinforcement and then attached by a steel wire or that solely steel wire is used. The cause of this error is believed to be improperly read detail drawings of the reinforcement layout and lack of knowledge of the purpose of stirrups.

To analyse this error an analogy has been made to compare RCC bands to a beam with an evenly distributed load applied. This load case could be compared to when the wall undergoes out-of-plane bending from seismic load.

According to Al-Emrani, Engström, Johansson & Johansson (2013) the principal stresses when the beam is subjected to out-of-plane bending can be explained by flexural stress and shear stress. The direction of the principal stresses have a major importance of the crack propagation in the beam. When the beam undergoes out-of-plane bending a force couple occur in the cross section of the beam in its uncracked condition. However, when the inclined shear cracks occur in the web of the beam the tension component of the force couple disappears and the concrete has to carry the compression force in the inclined struts.

To be able to analyse the equilibrium conditions in a reinforced structure with inclined shear cracks Al-Emrani et al. (2013) recommends to use the truss model. In the truss model it can be seen that shear reinforcement, stirrups, is needed to obtain equilibrium in the cross section to carry the vertical component from the compression force in the web, F_{cw}, see Figure 4.9.

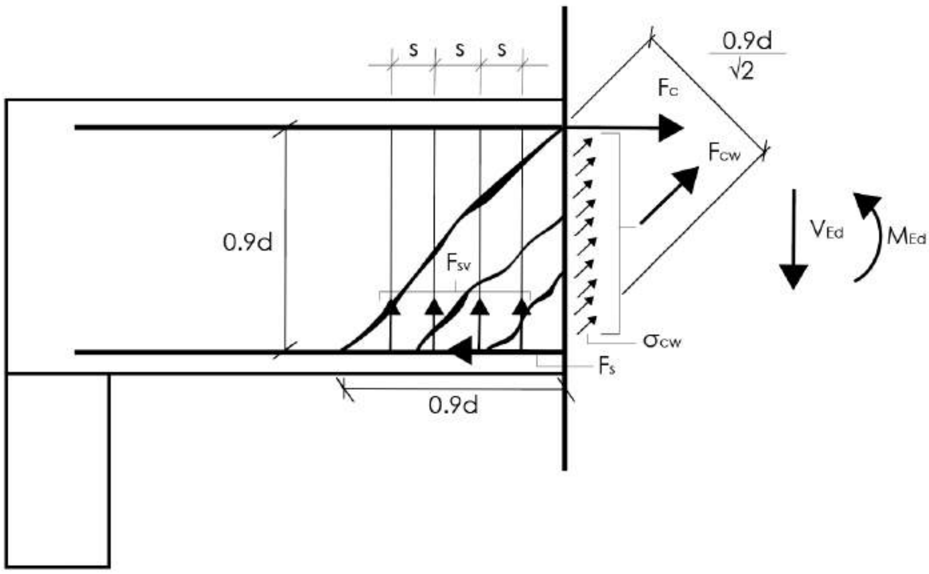


Figure 4.9: Truss model to analyse equilibrium after inclined shear cracks occur based on (Al-Emrani et al. 2013)

4.3.10 R6 - Insufficient radius of stirrup

In some cases it has been seen that the radius of stirrups is insufficient. A radius of less than 90° have been seen which is not accurate according to the drawings in the catalogue of approved designed by Build up Nepal (2017*a*).

The purpose of transverse reinforcement is to prevent transverse movement of the vertical reinforcement bars. According to IS 456:2000 (Earthquake Engineering Sectional Committee 2007) the radius of the stirrups have to be a minimum of 90°. See also Section 4.3.9.

4.3.11 R7 - Reinforcement exposed to environment (not bent in to the top RCC band)

One of the most common construction errors seen in CSEB-buildings is that the vertical reinforcement in the walls are left penetrating the top RCC band. This means that the reinforcement is not sufficiently anchored according to Build up Nepal (2017*a*) and that the reinforcement is left exposed to the environment.

The use of reinforcement in different kind of concrete structures is one of the most common building methods used worldwide (Bertolini et al. 2013). When reinforcement steel is cast in concrete a protective layer forms around the steel protecting it from material damage such as corrosion. Reinforcement steel unprotected by concrete or other protective layers can become exposed to water and oxygen in the air which are the necessary factors for initiation of corrosion.

The consequences of corroded reinforcement can lead to significantly decreased structural performance of a structure. The product of corrosion is often vastly larger than the steel itself and therefor a consequence of corrosion is spalling or cracking of the surrounding concrete. A connected consequence is degradation in the bond between the reinforcement and the concrete. In the case of chloride-induced corrosion the cross section of the reinforcement can decrease and lead to a lower loading capacity and fatigue strength of the reinforcement.

To avoid corrosion caused by different initiation processes an adequate concrete cover thickness has to be provided. Depending on the environment and relevant building code different cover thicknesses are required. In general it can be seen that in the case of chloride induced corrosion that a half of the cover thickness reduces the initiation time to one fourth. This is a crucial design and construction factor to ensure the ductility and performance of a construction with reinforced concrete.

According to EN 1992-1-1-2004 (European Committee for Standardization 2004) the anchorage of reinforcement can be done in a few different manners depending on situation. When the reinforcement is left sticking out the appointed anchorage (Build up Nepal 2017*a*) is disregarded which can lead to a pull-out failure.

4.3.12 R8 - Too low cover thickness of reinforcement in RCC bands

There are four RCC bands in a common CSEB-building, plinth, sill, lintel and floor or roof band. An error detected is that the cover thickness in these bands is made too low and in some cases the reinforcement is even completely exposed.

The RCC bands main role in the CSEB-buildings is to increase the seismic resistance as described in Section 2.3.3.2. By exposing the reinforcement in the RCC bands the capacity might be decreased due to corrosion as described in Section 4.3.11.

4.3.13 O1 - Openings larger than specified (windows and doors)

It has been seen that openings, windows and doors, are made lager than specified in drawings (Build up Nepal 2017*a*). This can be due to faulty production of window and door frames or due to a wish for larger openings.

The size and positions of openings in masonry structures strongly influence the in-plane resistance and consequently the earthquake resistant of shear walls (Tomazevic 1999). Therefore small openings centrally located are preferable. In the event of lateral earthquake load, stress concentrations occur in the areas of openings which can lead to cracks that causes deterioration of shear walls and their resistance. In the different national building codes there are recommendations regarding the maximum allowed size of openings as well as directives of there location in regards to corners and other openings.

According to NBC 203:1994 (Government of Nepal 1994*b*) openings for a one story building made of low strength masonry are not allowed to be more than 35 % in length of the total length of the wall in between two orthogonal walls or cross walls. According to IS 4326:1993 (Earthquake Engineering Sectional Committee 2005), which is also referred to in Auroville (Maïni 2005), the total length of openings is not allowed to exceed 50 % of the total length of a wall between two orthogonal walls or cross walls. The resemblance between low strength masonry in NBC 203:1994 and CSEB is made due to lack of similar directives for CSEB and will be used as reference as it is the conservative value between the directives in NBC 203:1994 and IS 4326:1993.

4.3.13.1 FE-analysis

An analysis was made on windows exceeding the directives of a maximum width of 35 % of the total length of the wall. The assumption was made that the window frames was consequently made to wide in production and therefor all windows were increased with 15cm on each side. Consequently the reinforced grouting on both sides of all windows were moved one step.

An analysis was made on doors exceeding the directive of a maximum width of 35 % of the total length of the wall. The same assumptions were made as in the model with enlarged windows.

Finally a model was made with both windows and doors enlarged as described previously.

Table 4.5 and 4.6 shows the maximum displacement and stresses for a model with too large window openings.

Table 4.5: Measured maximum displacement for too wide window openings from SAP2000, for location see Figure 4.2

	Displacement	Drift	Load case	Location
u_x	-0.71 mm	0.02%	7b	3
u_y	-0.71 mm	0.02%	10b	4

Table 4.6: Measured maximum and minimum stresses for too wide window openings from SAP2000, for locations see Figure 4.4

	Stress [N/mm^2]	Load case	Location
Top			
S11	0.092	16	6
S22	0.172	17	4
SMaxTop	-0.049	7b	13
SMinTop	-0.128	17	3
Bottom			
S11	0.096	16	5
S22	0.164	17	3
SMaxBottom	-0.088	16	6
SMinBottom	-0.155	17	4

Table 4.6 shows that only stresses S22 exceeds the capacity of the CSEB wall in this model. In Figure 4.10, to the left, it can be seen in dark blue where the tensile capacity, f_{xk1}, of 0.065 N/mm^2 is exceeded. To the right the same load case for the ideal model is displayed for comparison. The remaining tensile stress distributions that exceed the CSEB masonry capacity can be seen in Appendix L.

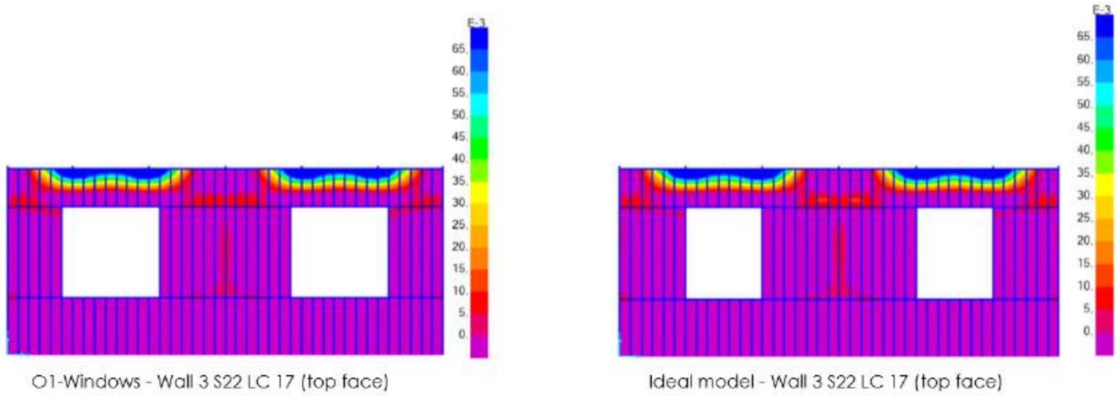


Figure 4.10: Tensile stresses S22 for too wide window openings in Wall 3 from load case 17 displayed to the left in [N/mm^2] and the same load case showed for the ideal model for comparison to the right.

Table 4.7 and 4.8 shows the maximum displacement and stresses for a model with too large doors.

Table 4.7: Measured maximum displacement for too wide door openings from SAP2000, for location see Figure 4.2

	Displacement	Drift	Load case	Location
u_x	-0.84 mm	0.024%	16	5
u_y	0.84 mm	0.024%	10a	2

Table 4.8: Measured maximum and minimum stresses for too wide door openings from SAP2000, for locations see Figure 4.4

	Stress [N/mm^2]	Load case	Location
Top			
S11	0.092	14	1
S22	0.176	17	4
SMaxTop	-0.051	17	2
SMinTop	-0.129	17	3
Bottom			
S11	0.097	16	5
S22	0.164	17	3
SMaxBottom	-0.088	16	6
SMinBottom	-0.159	17	6

Table 4.8 shows that only stresses S22 exceeds the capacity of the CSEB wall in this model. In Figure 4.11, to the left, it can be seen in dark blue where the tensile capacity, f_{xk1}, of 0.065 N/mm^2 is exceeded. To the right the same load case for the ideal model is displayed for comparison. The remaining tensile stress distributions that exceed the CSEB masonry capacity can be seen in Appendix L.

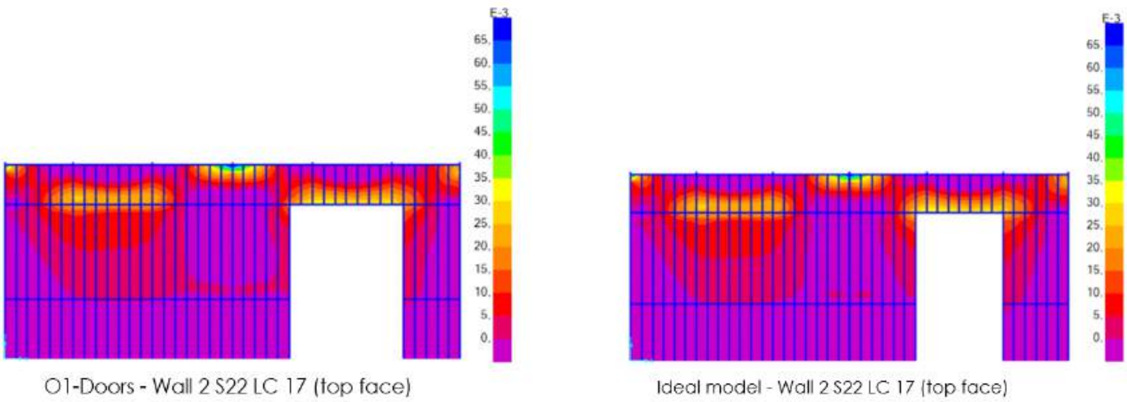


Figure 4.11: Tensile stresses S22 for too wide door openings in Wall 2 due to load case 17 displayed to the left in [N/mm^2] and the same load case showed for the ideal model for comparison to the right.

Table 4.9 and 4.10 shows the maximum displacement and stresses for a model with too large windows and doors.

Table 4.9: Measured maximum displacement for too wide window and door openings from SAP2000, for location see Figure 4.2

	Displacement	Drift	Load case	Location
u_x	-0.72 mm	0.021%	7b	3
u_y	0.85 mm	0.024%	10a	2

Table 4.10: Measured maximum and minimum stresses for too wide window and door openings from SAP2000, for locations see Figure 4.4

	Stress [N/mm^2]	Load case	Location
Top			
S11	0.092	16	6
S22	0.172	17	44
SMaxTop	-0.049	7b	13
SMinTop	-0.129	17	3
Bottom			
S11	0.096	16	5
S22	0.165	17	3
SMaxBottom	-0.088	16	6
SMinBottom	-0.155	17	4

Table 4.10 shows that only stresses S22 exceeds the capacity of the CSEB wall in this model. In Figure 4.12, to the left, it can be seen in dark blue where the tensile capacity, f_{xk1}, of 0.065 N/mm^2 is exceeded. To the right the same load case for the ideal model is displayed for comparison. The remaining tensile stress distributions that exceed the CSEB masonry capacity can be seen in Appendix L.

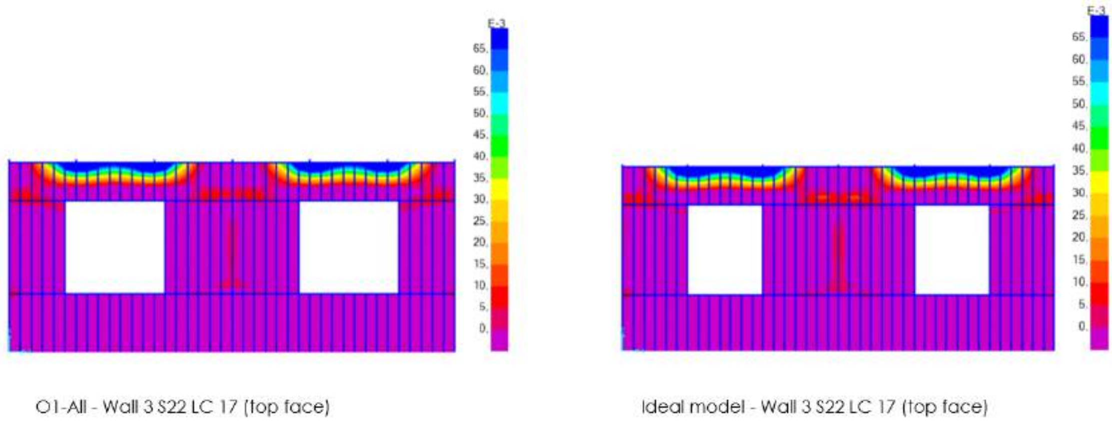


Figure 4.12: Tensile stresses S22 for too wide window and door openings in Wall 3 due to load case 17 displayed to the left in [N/mm^2] and the same load case showed for the ideal model for comparison to the right.

4.3.14 O2 - Openings too close to corners

It has been seen that openings, windows and doors, are made closer to corners or edges than specified in drawings (Build up Nepal 2017*a*).

Openings should preferably be centrally located since the walls are intended to brace each other and resist bending moments (Tomazevic 1999, Government of Nepal 1994*b*). Openings should also be located with a specified internal minimum distance since the walls are acting as shear walls resisting in-plane stresses.

According to NBC 203:1994 (Government of Nepal 1994*b*) a window should be placed no closer than 50 % of its height but no less than 600 mm to a orthogonal wall. A door should be placed no closer than 25 % of its height but no less than 450 mm to a orthogonal wall. The distance between openings should be no less than 50 % of the height of the window but no less than 600 mm.

4.3.14.1 FE-analysis

Table 4.11 and 4.12 shows the maximum displacement and stresses for a model with openings too close to corners. The assumption that all openings are made too close to corners have been made.

Table 4.11: Measured maximum displacement from SAP2000 for openings too close to corners, for location see Figure 4.2

	Displacement	Drift	Load case	Location
u_x	-0.98 mm	0.028%	16	6
u_y	0.96 mm	0.028%	10b	7

Table 4.12: Measured maximum and minimum stresses from SAP2000 for openings to close to corners, for locations see Figure 4.4

	Stress [N/mm^2]	Load case	Location
Top			
S11	0.107	14	1
S22	0.186	17	4
SMaxTop	-0.054	15	4
SMinTop	-0.226	10b	10
Bottom			
S11	0.093	16	5
S22	0.166	17	3
SMaxBottom	-0.087	16	6
SMinBottom	-0.217	10b	10

Table 4.12 shows that only stresses S22 exceeds the capacity of the CSEB wall in this model. In Figure 4.13, to the left, it can be seen in dark blue where the tensile capacity, f_{xk1}, of 0.065 N/mm^2 is exceeded. To the right the same load case for the ideal model is displayed for comparison. The remaining tensile stress distributions that exceed the CSEB masonry capacity can be seen in Appendix L.

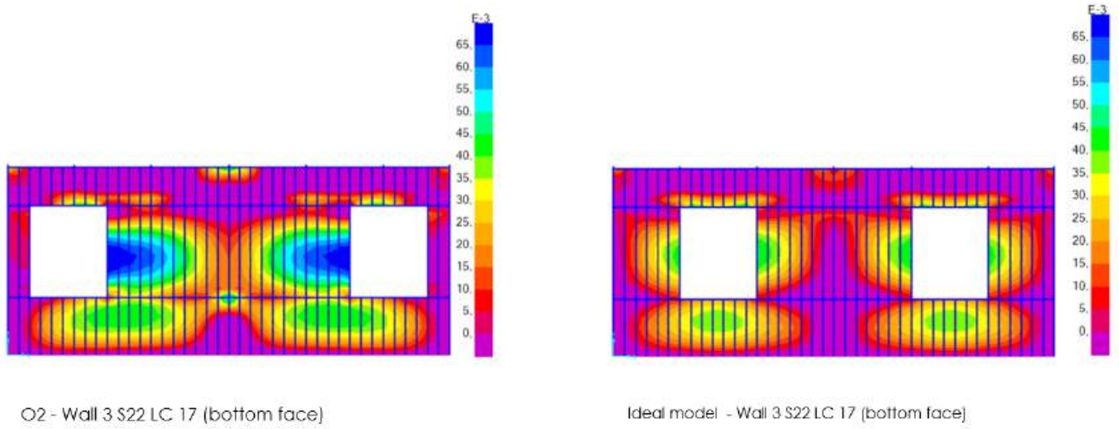


Figure 4.13: Tensile stresses S22 for openings too close to corners in Wall 3 due to load case 17 displayed to the left in [N/mm^2] and the same load case showed for the ideal model for comparison to the right.

4.3.15 C1 - Too slender lintel RCC bands between corner column and walls in design 2

In the design named "Single story house (2 rooms + Kitchen)" in the Design Catalogue by Build up Nepal (2017*a*) there is a free standing corner column. The column is according to the drawing suppose to be attached to the structure through the lintel RCC band. In some cases this RCC band is made thinner than the 100 mm stated in the drawings.

As described in Section 2.3.3.1 and 2.3.3.2 the RCC bands gives the structure its box behaviour. According to the catalogue of approved designs by Build up Nepal (2017*a*) the corner column is only attached to the rest of the structure by the lintel RCC bands and a corner of the roof structure and therefore the only proper structural elements including the corner column in the box behaviour are the lintel RCC bands.

4.3.15.1 FE-analysis

Table 4.13 and 4.14 shows the maximum displacement and stresses for a model with too slender RCC bands throughout the building.

Table 4.13: Measured maximum displacement from SAP2000 for too slender lintel RCC bands between corner column and walls, for location see Figure 4.2

	Displacement	Drift	Load case	Location
u_x	-1.17 mm	0.034%	16	5
u_y	0.88 mm	0.025%	17	8
u_z	2.33 mm	0.067%	16	11

Table 4.14: Measured maximum and minimum stresses from SAP2000 for too slender lintel RCC bands between corner column and walls, for locations see Figure 4.4

	Stress [N/mm^2]	Load case	Location
Top			
S11	0.126	16	6
S22	0.294	17	15
SMaxTop	-0.068	17	20
SMinTop	-0.302	17	3
Bottom			
S11	0.121	7a	14
S22	0.348	17	3
SMaxBottom	-0.118	16	6
SMinBottom	-0.265	17	4

Table 4.14 shows that only stresses S22 exceeds the capacity of the CSEB wall in this model. In Figure 4.14, to the left, it can be seen in dark blue where the tensile capacity, f_{xk1}, of 0.065 N/mm^2 is exceeded. To the right the same load case for the ideal model is displayed for comparison. The remaining tensile stress distributions that exceed the CSEB masonry capacity can be seen in Appendix L.

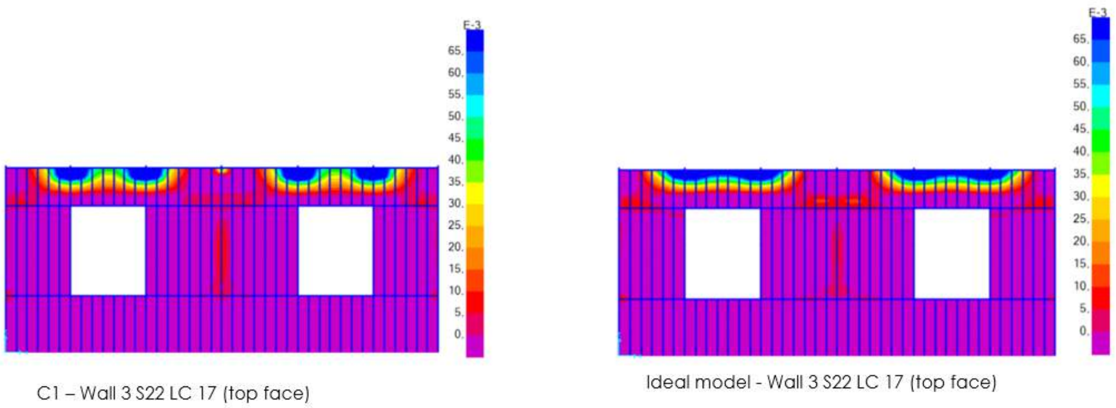


Figure 4.14: Tensile stresses S22 for too slender lintel RCC bands between corner column and walls in Wall 3 due to load case 17 displayed to the left in [N/mm^2] and the same load case showed for the ideal model for comparison to the right.

4.3.16 C2 - RCC bands are made too thin

According to Build up Nepal's drawings of CSEB-buildings all the RCC bands should be 100 mm thick. In some cases it has been detected that the RCC bands are made to thin or not consistent in height.

As described in Section 2.3.3.1 and 2.3.3.2 the RCC bands give the structure its box behaviour that is favourable regarding seismic behaviour.

A side effect of the RCC bands being too thin is that the cover thickness needed for the reinforcement might become to low and consequently lower the capacity of the reinforcement, as described in Section 4.3.11.

A FE-analysis was performed on design 2 with too thin RCC bands throughout the building, see Section 4.3.15. The analysis was primarily considering the effect of the corner column being attached to the rest of the structure by too thin RCC bands but also the general effect of overall to thin RCC bands.

4.3.17 C3 - Too coarse aggregate in RCC bands

According to Maïni (2005) the amount of gravel, sand, slit and clay is specified. It has been seen that in some RCC bands the amount of gravel is to high or that even pebbles are part of the concrete mix.

As described by Wilby (1983) the size of the aggregate and the proportions of cement, aggregate and water strongly affect the properties of the concrete such as, compressive and tensile strength. The factor determining the compressive strength of concrete is the water-cement-ratio. The aggregate size impacts the workability which can affect the properties of both fresh and cured concrete which can affect the final properties of the cured concrete.

According to Build up Nepal (2017*a*) the type of concrete throughout the CSEB-buildings should be M20 1:1.5:2 (cement:sand:aggregate) with a compressive strength of 20 MPa. To achieve the desired material properties for the concrete the coarse aggregate should be chosen to have an as large nominal maximum size as possible within the limits specified (Earthquake Engineering Sectional Committee 2007). The aggregate has to be smaller than one fourth of the minimum thickness of the concrete member and small enough to fill corners and adequately cover reinforcement.

4.3.18 RO1 - Roof is attached to walls in the wrong way

According to Build up Nepal (2017*a*) there are a few different methods provided for the attachment of the roof to the walls, depending on the design. Among the provided methods is an angled metal plate cast into the floor RCC band and welded to the roof rafters. Another method, in a similar way, is a ISA bolted to the floor RCC band and welded to the roof rafter. An error that is frequently seen in the CSEB-buildings is that the roof is not attached to the walls according to the drawings. Different methods are instead among which two of the most common ones are; the vertical reinforcement being bent around the roof rafters and that the roof rafters are welded to the vertical reinforcement from the walls.

In the event of an earthquake the inertia forces developed at the roof level due to the weight of the roof has to be transported to the supporting walls for further transport or energy dissipation (Tomazevic 1999). Therefore the roof has to be sufficiently connected to the walls RCC bands and braced in both directions. The weight of the roof is 4.983 kN compared to the whole structure that weighs 228.33 kN. This make the roof constitute 2 % of the whole weight.

4.3.18.1 RO2 - Reinforcement bent around rafters and purlins

It has been seen that the roof has been attached to the walls by bending the vertical reinforcement, that is left sticking out from the wall, around the rafters of the roof structure.

Disregarding other impacts that bending the vertical reinforcement from the wall around the rafters of the roof might have on the behaviour of the structure, exposed reinforcement, as described in Section 4.3.11, might loose its strength due to corrosion and the attachment of the roof will be weakened.

As described in Section 4.3.11, the vertical reinforcement of the wall should be adequately anchored in the top RCC band to ensure the accurate behaviour. Attaching the roof to the exposed reinforcement might therefore case unwanted structural behaviour.

4.3.18.2 RO3 - Reinforcement welded to rafters and purlins

It has also been seen that the roof rafters and purlins are welded to the vertical reinforcement that is left sticking out from the wall.

As described in Section 4.3.18.1, the strength of exposed reinforcement steel is not reliable and attaching the roof to the exposed reinforcement might lead to unwanted structural behaviour.

4.3.19 RO4 - Rafters missing

According to the approved designs by Build up Nepal (2017*a*) there is a specified amount of rafters and purlins. It has been seen that the specified amount is not always followed but some rafters and/or purlins are left out.

According to Section 3.2.2 dead, wind and imposed loads are applied on the roof. The loads are in reality initially divided between the purlins and thereafter between the rafters. Having less rafters than assigned means that the load will be more concentrated on the remaining rafters and impose higher point loads on the walls.

4.3.19.1 FE-analysis

Table 4.15 and 4.16 shows the maximum displacement and stresses for a model with rafters missing.

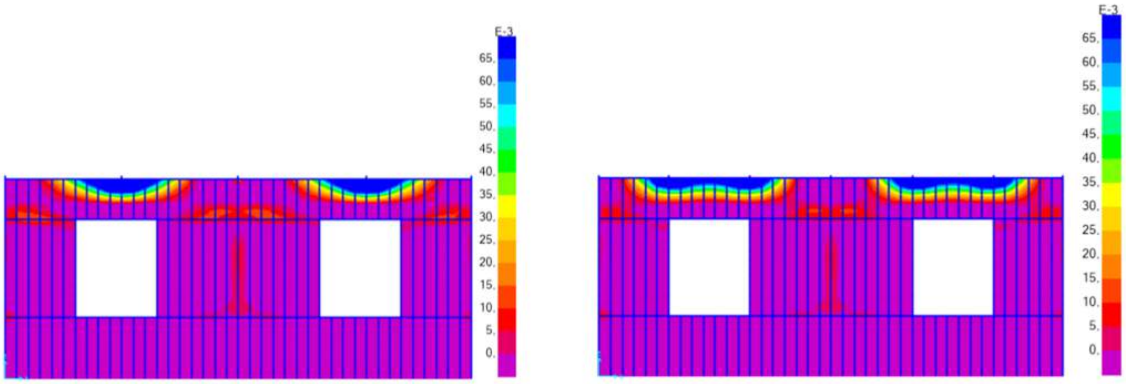
Table 4.15: Measured maximum displacement from SAP2000 for roof rafters missing, for location see Figure 4.2

	Displacement	Drift	Load case	Location
u_x	-0.82 mm	0.024%	16	5
u_y	-0.88 mm	0.025%	16	7

Table 4.16: Measured maximum and minimum stresses from SAP2000 for roof rafters missing, for locations see Figure 4.4

	Stress [N/mm^2]	Load case	Location
Top			
S11	0.133	16	18
S22	0.255	17	17
SMaxTop	-0.05	8b	13
SMinTop	-0.126	17	3
Bottom			
S11	0.092	17	11
S22	0.161	17	3
SMaxBottom	-0.127	16	18
SMinBottom	-0.226	17	14

Table 4.16 shows that stresses S11 and S22 exceeds the capacity of the CSEB wall in this model. In Figure 4.15, to the left, it can be seen in dark blue where the tensile capacity, f_{xk1}, of 0.065 N/mm^2 is exceeded. To the right the same load case for the ideal model is displayed for comparison. The remaining tensile stress distributions that exceed the CSEB masonry capacity can be seen in Appendix L.



RO4 - Wall 3 S22 LC 17 (top face)   Ideal model - Wall 3 S22 LC 1 (top face)

Figure 4.15: Tensile stresses S22 for roof rafters missing in Wall 3 due to load case 17 displayed to the left in [N/mm^2] and the same load case showed for the ideal model for comparison to the right.

4.3.20 X1 - Free standing corner column in design 2

As described in Section 4.3.15 the free standing column in design 2 according to the Design Catalogue by Build up Nepal (2017*a*) should be attached to the structure by a 100 mm thick RCC band. In several cases this band is missed and either an alternative solution is made or the RCC band is left out completely.

4.3.20.1 FE-analysis

Table 4.17 and 4.18 shows the maximum displacement and stresses for a model with a free standing corner column.

Table 4.17: Measured maximum displacement from SAP2000 for free standing corner column, for location see Figure 4.2

	Displacement	Drift	Load case	Location
u_x	-0.82 mm	0.024%	16	5
u_y	-2.67 mm	0.077%	16	9
u_z	11.06 mm	0.32%	17	12

Table 4.18: Measured maximum and minimum stresses from SAP2000 for free standing corner column, for locations see Figure 4.4

	Stress [N/mm^2]	Load case	Location
Top			
S11	0.114	16	19
S22	0.176	17	4
SMaxTop	-0.05	8a	13
SMinTop	-0.141	17	8
Bottom			
S11	0.093	16	5
S22	0.13	17	8
SMaxBottom	-0.087	16	6
SMinBottom	-0.158	17	4

Table 4.18 shows that only stresses S22 exceeds the capacity of the CSEB wall in this model. In Figure 4.16, to the left, it can be seen in dark blue where the tensile capacity, f_{xk1}, of 0.065 N/mm^2 is exceeded. To the right the same load case for the ideal model is displayed for comparison. The remaining tensile stress distributions that exceed the CSEB masonry capacity can be seen in Appendix L.

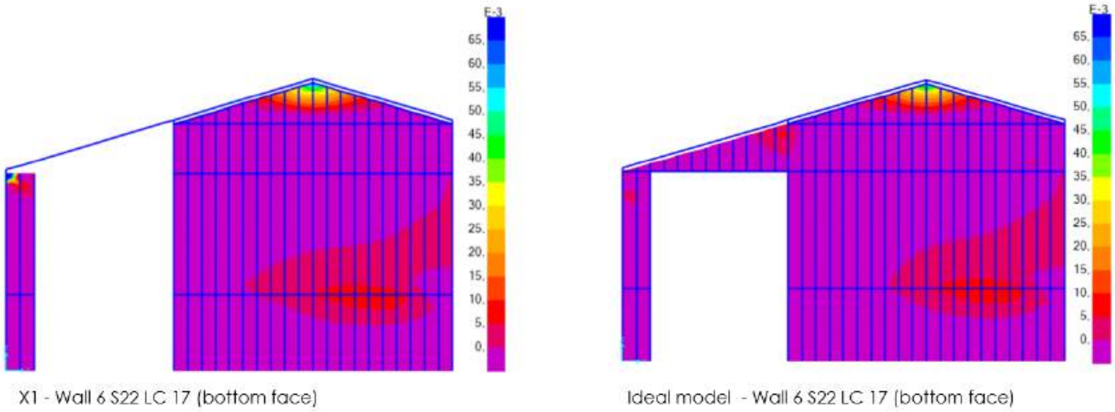


Figure 4.16: Tensile stresses S22 for missing RCC band to corner column in Wall 6 due to load case 17 displayed to the left in [N/mm^2] and the same load case showed for the ideal model for comparison to the right.

4.3.21 X2 - Gable wall made too high

The allowed height of the gable walls are stated in the Design Catalogue by Build up Nepal (2017*a*). In some cases the gable walls are made to high to fit an attic under the roof and consequently the inner walls do not attach to the roof.

There are a few different ways the gable walls have been made and can be made to high. In this analysis a likely and at the same time worst case scenario have been chosen.

4.3.21.1 FE-analysis

Table 4.19 and 4.20 shows the maximum displacement and stresses for a model with the gable walls made to high.

Table 4.19: Measured maximum displacement from SAP2000 for gable wall made too high, for location see Figure 4.2

	Displacement	Drift	Load case	Location
u_x	-1.01 mm	0.029%	7b	1
u_y	0.94 mm	0.027%	10a	10

Table 4.20: Measured maximum and minimum stresses from SAP2000 for gable wall made too high, for locations see Figure 4.4

	Stress [N/mm^2]	Load case	Location
Top			
S11	0.106	16	14
S22	0.103	11a	7
SMaxTop	-0.061	7b	12
SMinTop	-0.13	10b	7
Bottom			
S11	0.088	9a	11
S22	0.157	17	3
SMaxBottom	-0.059	17	12
SMinBottom	-0.139	16	14

Table 4.20 shows that only stresses S22 exceeds the capacity of the CSEB wall in this model. In Figure 4.17, to the left, it can be seen in dark blue where the tensile capacity, f_{xk1}, of 0.065 N/mm^2 is exceeded. To the right the same load case for the ideal model is displayed for comparison. The remaining tensile stress distributions that exceed the CSEB masonry capacity can be seen in Appendix L.

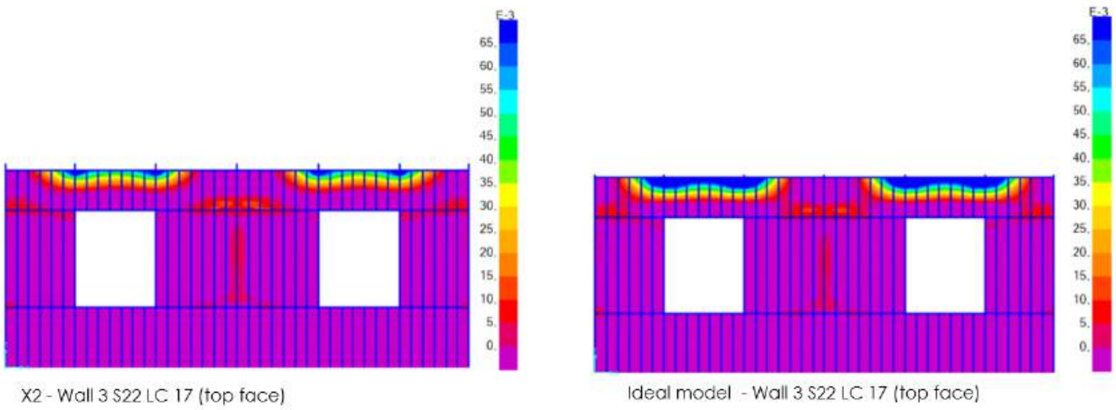


Figure 4.17: Tensile stresses S22 for gable wall made too high in Wall 3 due to load case 17 displayed to the left in [N/mm^2] and the same load case showed for the ideal model for comparison to the right.

4.3.22 X3 - Extra floor is added to building with small bricks

The CSEB-buildings approved by the government of Nepal are according to Build up Nepal (2017*a*). The grant provided for earthquake resistant buildings by the PDNA (Government of Nepal 2015) is paid after completion of the house by inspectors. Since some of the people in poor rural villages of Nepal are dependant on the grant but does not fully understand the importance of following the drawings an extra floor is added to buildings built with Block 295, according to Maïni (2005), that is only approved to be build as a one-storey building after the grant is received.

4.3.22.1 FE-analysis

Table 4.21 and 4.22 shows the maximum displacement and stresses for a model with a second floor.

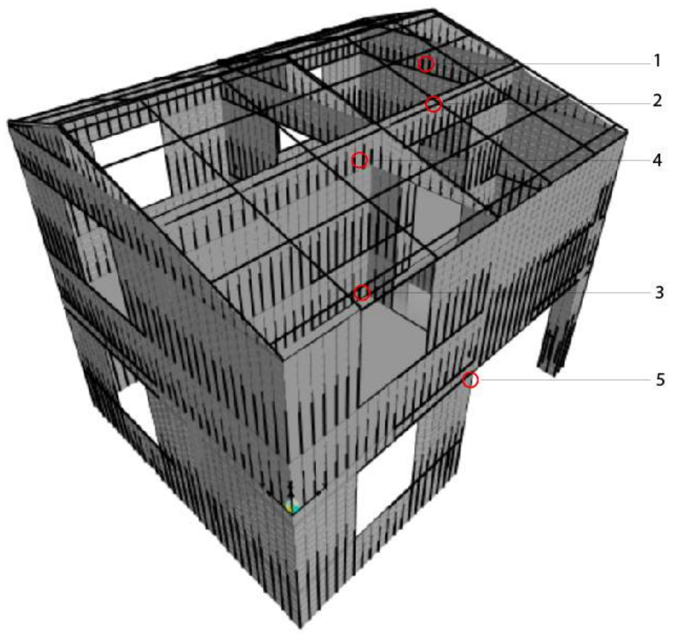


Figure 4.18: Illustration showing locations of maximum displacements and stresses for error X3.

Table 4.21: Measured maximum displacement from SAP2000 for additional floor added to building built with small bricks, for location see Figure 4.18

	Displacement	Drift	Load case	Location
u_x	1.99 mm	0.057%	9a	1
u_y	-2.46 mm	0.07%	10b	2

Table 4.22: Measured maximum and minimum stresses from SAP2000 for additional floor added to building built with small bricks, for locations see Figure 4.18

	Stress [N/mm^2]	Load case	Location
Top			
S11	0.128	10b	3
S22	0.283	11a	3
SMaxTop	-0.129	10b	3
SMinTop	-0.429	7b	5
Bottom			
S11	0.127	11b	3
S22	0.287	11b	3
SMaxBottom	-0.131	11b	3
SMinBottom	-0.377	10b	4

Table 4.22 shows that only stresses S22 exceeds the capacity of the CSEB wall in this model. In Figure 4.19, to the left, it can be seen in dark blue where the tensile capacity, f_{xk1}, of 0.065 N/mm^2 is exceeded. To the right the same load case for the ideal model is displayed for comparison. The remaining tensile stress distributions that exceed the CSEB masonry capacity can be seen in Appendix L.

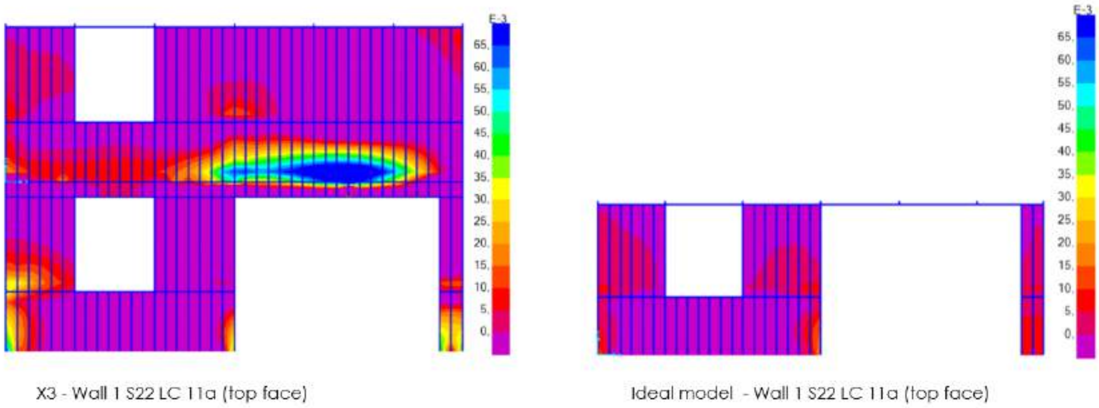


Figure 4.19: Tensile stresses S22 for additional floor added to building built with small bricks in Wall 1 due to load case 11a displayed to the left in [N/mm^2] and the same load case showed for the ideal model for comparison to the right.

4.3.23 X4 - Damages bricks are used

It has been seen that damaged bricks are used sometimes and most often the damages concerns the interlocking key of the brick.

According to Maïni (2005) the interlocking key provides extra resistance to shear compared to other hollow bricks. This will provide a more resilient structure that under earthquake loading will withstand better without as severe damages. Sturm et al. (2014) sees in their study that the main part of the shear capacity of interlocking brick structures comes from the interlocking key.

4.3.24 X5 - Roof band made all around building

In some CSEB-buildings it has been seen that the roof band that according to drawings (Build up Nepal 2017*a*) only should go around part of the building is made all around the building. This leads to the roof being raised on one side of the building and therefore the angle of the roof on one side is lowered and consequently the gable wall becomes higher to attach to the roof.

4.3.24.1 FE-analysis

Table 4.23 and 4.24 shows the maximum displacement and stresses for a model with the roof band made all around the building.

Table 4.23: Measured maximum displacement from SAP2000 for roof band made all around the building, for location see Figure 4.2

	Displacement	Drift	Load case	Location
u_x	-0.68 mm	0.02%	7b	3
u_y	-0.76 mm	0.022%	16	10

Table 4.24: Measured maximum and minimum stresses from SAP2000 for roof band made all around the building, for locations see Figure 4.4

	Stress [N/mm^2]	Load case	Location
Top			
S11	0.124	17	4
S22	0.248	17	6
SMaxTop	-0.074	17	16
SMinTop	-0.13	17	2
Bottom			
S11	0.113	17	5
S22	0.145	17	2
SMaxBottom	-0.118	17	4
SMinBottom	-0.223	17	4

Table 4.24 shows that only stresses S22 exceeds the capacity of the CSEB wall in this model. In Figure 4.20, to the left, it can be seen in dark blue where the tensile capacity, f_{xk1}, of 0.065 N/mm^2 is exceeded. To the right the same load case for the ideal model is displayed for comparison. The remaining tensile stress distributions that exceed the CSEB masonry capacity can be seen in Appendix L.

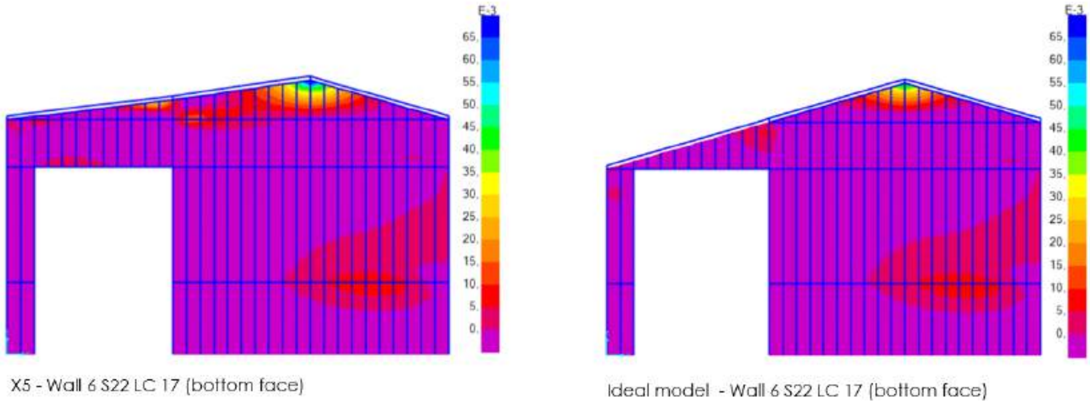


Figure 4.20: Tensile stresses S22 for roof band made all around the building in Wall 6 due to load case 17 displayed to the left in [N/mm^2] and the same load case showed for the ideal model for comparison to the right.

4.3.25 Q1 - How weak/strong are gable walls?

The government of Nepal has raised the question of the strength of gable walls. According to them the gable walls are the weak part of a building which is a statement questioned by Build up Nepal.

As can be seen in Section 3.2.2.3 and Table 3.3 the gable walls for the ideal model get displacements of 0.48 mm in the x-direction and 0.20 in the y-direction. In section 4.3.21.1 it can be seen that an increased gable wall of 500 mm higher in house design 2 the displacements reaches -1.01 mm in the x-direction, see Table 4.19.

4.3.26 Q2 - Impact of higher and lower compressive strength of the CSEB than specified?

Due to varying soil quality, varying amount of soil mixture in the machine during production of the bricks, false curing of the bricks or due to other factors the material parameters of the bricks might vary. Especially a weakened compressive strength is of interest since this might strongly affect the earthquake resistance of the building. A value of 5 MPa for the CSEB is normally used, as can be seen in the structural report carried out by Build up Nepal (2017*b*), in calculations but it would be of interest for Build up Nepal to see how well a building calculated with a compressive strength down to 2 MPa and up to 10 MPa performs under earthquake loading.

4.3.26.1 FE-analysis

Two FE-models were created, one with a lower compressive strength of 2 MPa called Q2a and one with a higher compressive strength of of 10 MPa called Q2b. Consequently the modulus of elasticity was adjusted for the changed compressive strength according to EC6 (European Committee for Standardization 2005). The calculations for this can be seen in Appendix H.

Table 4.25 and 4.26 shows the maximum displacement and stresses for a model with a lower compressive strength than assumed and therefore consequently a lower modulus of elasticity, see Appendix H.

Table 4.25: Measured maximum displacement with a compressive strength of 2 MPa of the bricks from SAP2000, for location see Figure 4.2

	Displacement	Drift	Load case	Location
u_x	-0.98 mm	0.028%	16	5
u_y	0.96 mm	0.028%	10a	2

Table 4.26: Measured maximum and minimum stresses with a compressive strength of 2 MPa of the bricks from SAP2000, for locations see Figure 4.4

	Stress [N/mm^2]	Load case	Location
Top			
S11	0.067	15	9
S22	0.119	17	4
SMaxTop	-0.036	7b	13
SMinTop	-0.103	7a	13
Bottom			
S11	0.064	17	11
S22	0.102	7a	13
SMaxBottom	-0.051	16	6
SMinBottom	-0.11	17	6

Table 4.26 shows that stresses S11 and S22 exceed the capacity of the CSEB wall in this model. In Figure 4.21, to the left, it can be seen in dark blue where the tensile capacity, f_{xk2}, of 0.065 N/mm^2 is exceeded. To the right the same load case for the ideal model is displayed for comparison. The remaining tensile stress distributions that exceed the CSEB masonry capacity can be seen in Appendix L.

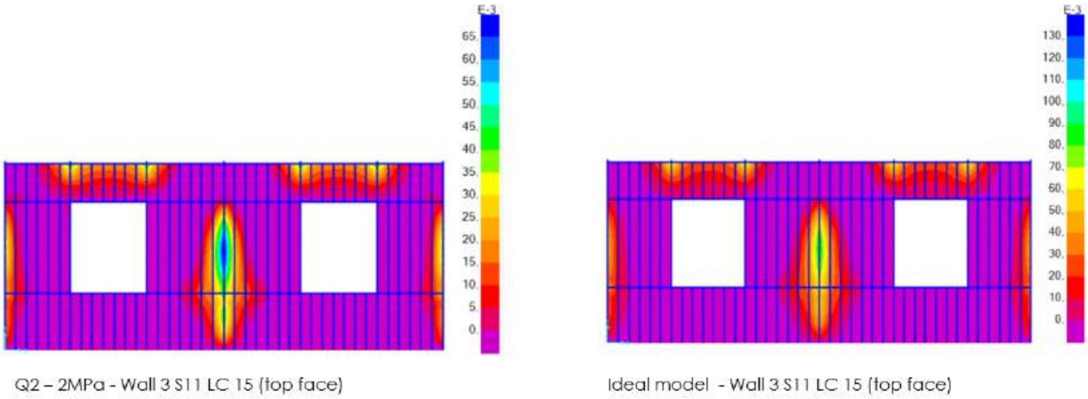


Figure 4.21: Tensile stresses S11 for the model made with a compressive strength of 2 MPa of the bricks in Wall 3 due to load case 15 displayed to the left in [N/mm^2] and the same load case showed for the ideal model for comparison to the right.

Table 4.27 and 4.28 shows the maximum displacement and stresses for a model with higher compressive strength than assumed and therefore consequently a higher modulus of elasticity, see Appendix H.

Table 4.27: Measured maximum displacement with a compressive strength of 10 MPa of the bricks from SAP2000, for location see Figure 4.2

	Displacement	Drift	Load case	Location
u_x	-0.77 mm	0.022%	16	5
u_y	-0.68 mm	0.02%	16	12

Table 4.28: Measured maximum and minimum stresses with a compressive strength of 10 MPa of the bricks from SAP2000, for locations see Figure 4.4

	Stress [N/mm^2]	Load case	Location
Top			
S11	0.113	16	6
S22	0.201	17	4
SMaxTop	-0.062	17	2
SMinTop	-0.159	14	3
Bottom			
S11	0.121	16	5
S22	0.204	17	3
SMaxBottom	-0.108	16	6
SMinBottom	-0.180	17	4

Table 4.28 shows that only stresses S22 exceeds the capacity of the CSEB wall in this model. In Figure 4.22, to the left, it can be seen in dark blue where the tensile capacity, f_{xk1}, of 0.065 N/mm^2 is exceeded. To the right the same load case for the ideal model is displayed for comparison. The remaining tensile stress distributions that exceed the CSEB masonry capacity can be seen in Appendix L.

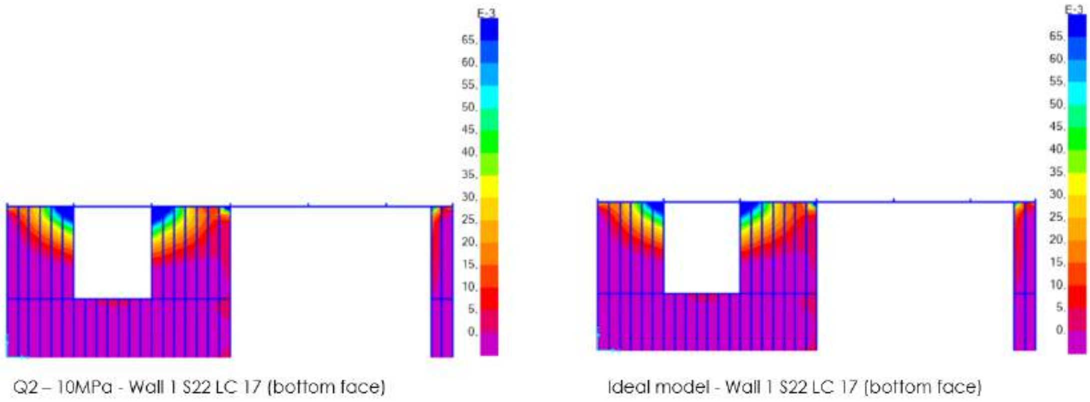


Figure 4.22: Tensile stresses S22 for the model made with a compressive strength of 10 MPa of the bricks in Wall 1 due to load case 17 displayed to the left in [N/mm^2] and the same load case showed for the ideal model for comparison to the right.

4.3.27 Q3 - Dry-stacking vs. use of mortar

When Build up Nepal started there work with CSEB the use of mortar was not a recommendation. Currently mortar is a recommendation following the instructions from Auroville (Maïni 2005). Despite the recommendation to use mortar it is not always used and the need of it is very often questioned. Mortar is relatively expensive and makes the construction process of CSEB-buildings more advanced. Since the vast majority of the people building their homes with CSEB are poor the necessity to use mortar needs to be proven and provided.

In masonry structures mortar is commonly used in between the masonry layers to bind the units together. The mortar increases the shear capacity and the tensile strength of the masonry structure (Tomazevic 1999). Interlocking bricks behave similar to other type of masonry units under compression. Depending on if mortar is used in between the masonry units or not the tensile strength might differ. Without mortar no masonry structure comprises of any significant tensile strength. The shear strength of masonry structures is governed by the friction between the bricks, with influence from the mortar. For interlocking brick structures, such as CSEB-buildings, the interlocking features strongly contributes to the shear strength even without mortar (Sturm et al. 2014).

According to Maïni (2005) the hollow interlocking bricks must be laid with a five millimetre cement-sand mortar in between every course to ensure the earthquake resistance. The mortar is needed to achieve the goal of a homogeneous material which will provide the structure with an increased capacity. It is specifically stated in Auroville's training manual that "they (hollow interlocking bricks) must not be dry stacked".

See Section 4.3.1 and 4.3.2 for specific studies on dry-stacked respectively masonry structures with mortar.

5

Discussion

5.1 Ideal model

The displacements of the ideal model far from causes drifts exceeding the drift limitations of 0.4 % described in Section 3.2.3. The absolute maximum displacement is 0.83 mm which relates to a drift of 0.024 % which, despite the method of application of earthquake loads, can be considered very low.

It can be seen in Table 4.2 that the only stresses exceeding the capacity of the CSEB masonry are the bending tensile stresses S22. The high concentration of bending tensile stresses are found in the top of the walls and the column where the roof is anchored. Most of theses stresses are taken up by the 100 mm thick RCC band but some stresses proceed down into the CSEB masonry wall which could be compromising for the structure and could cause cracks in the walls.

It can also be seen in the analyse of the ideal model that the bending tensile stresses decrease under the RCC bands in the lower part of the building, see Appendix L. This proves the contribution and importance of well constructed RCC bands throughout the building.

The load case with wind in y-direction is the load case causing maximum bending tensile stresses in the ideal model. The high wind loads are caused by the hilly landscapes as well as the high altitudes of Nepal. The Nepal National Building Codes and other national building codes generally approaches the level of applied earthquake load by a average value based on the history in the area and does not necessarily include extreme earthquakes such as the Gorkha Earthquake. This suggests that even though that the analysis shows that load cases including wind loads causes the maximum stresses, severe earthquakes could cause greater stresses in the structure. It is therefor important that the CSEB buildings are constructed accurately to also be able to withstand severe earthquakes.

5.2 Errors

The errors occurring in Build up Nepal's CSEB-building have been detected, compiled and analysed in Section 4.2 and 4.3. In the following Sections the results will be discussed and the severeness of each error will be evaluated based on the findings in previous Sections.

As a general comment it can be said that many of the errors are difficult to detect in retrospect and it is also not always feasible to estimate which errors have been built into the building due to lack of supervision during construction.

5.2.1 W1 - CSEB not levelled when build without mortar

As Ngapeya et al. (2018) showed in their study, even small imperfections in height causes crack initiations at down to 5.4 % of the ultimate load capacity. Since height imperfections can be the cause of one of several factors; difference in height of the brick, roughness of the brick surface or dirt on the surface of the brick, the risk for imperfections to occur in a CSEB wall can be considered to be relatively high. Dry-stacking of bricks is therefore a non-reliable way to assemble CSEB buildings, as stated in Maïni (2005).

The strong suggestion for Build up Nepal is therefore to advice the use of mortar in between layers of CSEB to be able to disguise height imperfections.

5.2.2 W2 - CSEB not levelled when build with mortar

As shown in the report by Ngapeya et al. (2018), non-consistent load transfer between bricks in a masonry wall causes stress concentrations that can lead to premature crack initiations. Due to the difference in material properties between mortar and CSEB and the interface between the two materials in combination with uneven heights of mortar layers and, as a consequence, non-plane levels of CSEB it can be assumed that non-consistent load transfer also occurs in these walls. Non-consistent load transfer leads to stress concentrations and can therefor cause premature risk initiations which can compromise the resistance to earthquake loads and other loads.

To follow the recommendations from Maïni (2005) to have 5 mm mortar layers in between the CSEB layers is to prefer to avoid stress concentrations. It is important for Build up Nepal to emphasise the importance of even and plane layers of mortar and CSEB to reach the desired behaviour that the use of mortar provides.

The importance of a levelled foundation to make it possible to continue to lay even layers of CSEB and mortar should also be emphasised.

5.2.3 W3 - Missing mortar/concrete in some vertical holes of the CSEB

According to the drawings by Build up Nepal (2017*a*) and Maïni (2005) all cavities should be filled with concrete and in some critical locations there should be vertical reinforcement in the holes. Consequently there will be two types of errors regarding missing concrete in the holes; missing concrete in the vertical holes with reinforcement and in the holes without reinforcement.

Since the reinforcement is added in the CSEB structure to provide stability and ductility it is important that the demands on construction to achieve these properties are fulfilled. Grouting in the cavities with reinforcement fill the function of creating an interaction between the CSEB and the reinforcement. Without grouting the interaction wanted between the two materials is missing and consequently the structure will not possess the full earthquake resistant provided by grouted reinforced masonry.

According to Tomazevic (1999) grouting in the holes without reinforcement provides additional earthquake resistance which is sought after in earthquake prone zones such as Nepal. Although the severity of missed grout in holes without reinforcement is not as great at in the holes with reinforcement it should be advised to be used.

Due to the importance of keeping the monolithic box behaviour of the whole structure for the earthquake resistance the recommendation to Build up Nepal is to advise and stress the importance of filling all the vertical cavities with mortar especially where vertical reinforcement is integrated.

5.2.4 W4 - Hairline crack through wall

As described in Section 4.3.4, the occurrence of different type of hairline cracks has led to an unwillingness to build more CSEB buildings in that village due to a lack of trust in the system. This study has shown that there can be multiple reasons for a crack to occur in a CSEB wall and the type of resulting cracks might be of different natures.

One of the likely reasons for a hairline crack to occur in a masonry structure is uneven layers of bricks. This could be when built without or with mortar in between the layers. CSEB structures without mortar are assumed to be likelier to experience premature cracking than when build with mortar, see Sections 5.2.1, 5.2.2 and 5.2.27.

One likely reason could also be ground settlement. It is important to build a CSEB on a stable ground but in reality in rural Nepal it can be difficult to find such land within the possession of a certain village. Here the emphasis should be put into initially finding an appropriate piece of land and then build a proper foundation.

Minor earthquakes could be the reason of these hairline cracks. This is an unlikely reason since the buildings are designed according to the Nepal National Building Code which takes stronger earthquakes into account, see Section 2.5.1. Further, a minor earthquake in combination with one of the above mentioned reasons or another error described in this report could lead to earlier initiations of cracks in zones with high stress concentrations close to the capacity of the CSEB masonry.

For knowledge about the actual reason why different type of hairline cracks occur, a more detailed study on this specific error has to be performed.

The danger of a hairline crack in a CSEB wall might also differ based on the type of crack and the location of a crack. Requirements on cracks in a CSEB structure according to Adam & Agib (2001) can be seen in Table 4.4.

In relation to location of the cracks the vertical reinforcement might play an important role, see Section 5.2.8.

The hair line cracks discovered are not likely to affect the monolithic behaviour of the structure directly due to the fact that the crack does not reach through the RCC bands. The reason behind the cracks is not ambiguous and might be due to different reasons in varies villages. The recommendation to Build up Nepal would be to further do studies on the reason behind the cracks. Nevertheless, the hair line cracks discovered go straight though the bricks splitting in two as mentioned and needs due to this to be retrofitted according to Adam & Agib (2001).

5.2.5 R1 - Insufficient lap length of reinforcement

As presented in Melander (1992) research the failure of the lap splices changed from a ductile to a brittle one due to the insufficient lap length of the reinforcement. This can be viewed as critical especially if the insufficient lap length is at vulnerable locations such as corners or around door and window openings.

If the lap length is decreased to two-thirds or less the ultimate load capacity is significantly decreased reported by Abdel-Kareem et al. (2013). However, the lack of knowledge on how much the lap lengths have been decreased in construction as well as how often this occur in the same structure makes it difficult to estimate the severeness of this error. If the error happens due to a lack of knowledge on how long the lap length has to be it can be assumed that several lap splices are inadequate in the same building. This will then have a major impact on the earthquake resistance of the building hence a brittle failure will be more likely to occur.

Due to this the advise to Build up Nepal is to emphasise the importance of sufficient lap lengths of the reinforcement so the stresses the building will endure during its service life will be transferred over the splices.

5.2.6 R2 - Field bending of reinforcement

As Pook (2007) describes, over bending of reinforcement can lead to metal fatigue and in this case, consequently, a decrease in strength of the vertical reinforcement. The vertical reinforcement is included in the structure to provide ductility and ensure a monolithic behaviour as described in Section 4.3.8. Depending on at what level the over bending of the vertical reinforcement is performed, the consequences and severeness of the error might differ. If assumed that the over bending is performed, as often seen, over the plinth band and that it leads to significantly weakened properties so that the reinforcement will break before the CSEB masonry under earthquake load, the impact on the earthquake resistance will be significant since the anchorage in the foundation needed to assure the correct contribution from the reinforcement will be lacking. Also, the general monolithic behaviour might be lost in the parts where over bending has been performed.

The aim should be to avoid filed bending. In cases with faulty placement of the vertical reinforcement and field bending is needed to proceed with construction, regulations should be drawn up and attempted to be followed. Adequate tools to successfully follow regulations for field bending might need to be provided to avoid over bending and spalling of the surrounding concrete. Both American Concrete Institute (1999) and Standards Australia Committee BD-002 Concrete Structures (2001) have provide regulations regarding field bending of reinforcement that could be used as a basis for a safe method for field bending when necessary.

The recommendation to Build up Nepal is therefore, as mentioned above, to avoid field bending. To ensure that field bending does not occur a properly constructed foundation with integrated vertical reinforcement at the correct locations, according to drawings, is needed. To achieve this Build up Nepal might need to supervise the constructions more during this phase or provide more preparation to the locals and entrepreneurs before the construction process begins.

5.2.7 R3 - All vertical reinforcement start at plinth band

If the vertical reinforcement starts from and is anchored in the plinth band or an additional cast plinth band above, the ductile elements, the reinforcement, is not connected with the foundation. That means that in the event of an earthquake the lateral loads imposed on the structure causing shear stresses in the structure can only be resisted by the concrete from the plinth band and up. That does not comply with IS 4236 (Earthquake Engineering Sectional Committee 2005) directive that all structural elements and their connection should be designed to have ductile failure to resist earthquake loads.

The recommendation to Build up Nepal is therefor to inform the locals that they need to integrate the vertical reinforcement in the first phase of construction, in the foundation, to not decrease the monolithic behaviour of the building during seismic loads, which might lead to much lower capacities of the buildings.

5.2.8 R4 - Vertical reinforcement missing

Accurately added vertical reinforcement in a masonry structure, CSEB building, assures a monolithic behaviour which is necessary for earthquake resistance.

Without vertical reinforcement in critical locations such as corners and around openings the possibility of a collapse under earthquake load or high wind loads is significantly increased as seen in the study by Doğangün et al. (2008). Even though the most needed vertical reinforcement is in the critical locations, other vertical reinforcement throughout the building ensures an overall monolithic behaviour.

From the analysis of the ideal model in Section 4.1 it can be seen that the critical stresses occurs in relation to corners of openings, at connections between roof and walls and in the top of the column. If the reinforcement would be forgotten in these places it would be an extra severe error. As can be seen in the results from the FE-analysis of the errors in Section 4.3 different errors causes locations of high stresses to occur in different places. Forgotten reinforcement in other places than the ones described previous could therefor also impact the overall behaviour of the structure and cause the CSEB masonry to crack if other errors are made.

Therefore it is advised to Build up Nepal that adequate vertical reinforcement, accurately anchored in the foundation and the top RCC band is necessary for the monolithic behaviour and an earthquake resistant CSEB building.

5.2.9 R5 - Appropriate stirrups in RCC bands missing

As long as the cross section of the RCC bands remains uncracked the lack of appropriate stirrups is not critical hence all the tensile force is taken care of by the concrete, as explained by Al-Emrani et al. (2013). However, when the inclined shear cracks occur in the web the force couple is no longer balanced in the cross section. This lead to flexural shear crack failure in the web if no stirrups are provided.

If the RCC bands crack the whole building will loose it's monolithic box behaviour during an earthquake which is a very important characteristic to possess in earthquake resistance design. Additionally, the bending strength of the masonry wall will decreased.

The advice to Build up Nepal is due to this to inform the locals about the importance of the stirrups and the decrease in the box behaviour of the building if not adequate stirrups are provided.

5.2.10 R6 - Insufficient radius of stirrup

Insufficient radius of the stirrup might cause the stirrups to loose their function. Therefore the same consequences might be the result of insufficient radius of stirrups as from other types of inappropriate stirrups, see Section 5.2.9.

5.2.11 R7 - Reinforcement exposed to environment (not bent into the top RCC band)

To avoid corrosion in reinforcement it is important to keep a sufficient concrete cover thickness throughout the building. For obvious reasons, reinforcement that is left completely penetrating the concrete does not reach the requirements of adequate cover. Reinforcement has to be bent into the concrete and adequate concrete cover thickness has to be ensured.

The structural consequences of corroding concrete is that the reinforcement gets weakened. The structural response in the CSEB buildings by Build up Nepal from not anchoring the vertical reinforcement in the top is that the aimed for, monolithic behaviour is lost. The risk for corrosion in combination with absent anchorage of the vertical reinforcement is alarming.

After reviewing the drawings by Build up Nepal (2017*a*) is can be seen that a detail for how to properly cover and anchor the vertical reinforcement in the top RCC band is missing. Clear drawings and information about the proper way to handle the ends of the vertical reinforcement could decrease the risk of this error to occur. By properly anchoring the reinforcement in the top RCC band the error described in Section 4.3.18 regarding how the roof structure is connected to the walls could possibly easier be avoided.

The recommendation to Build up Nepal is therefore to include detail drawings of the anchorage of the vertical reinforcement in the top RCC band as well as inform the local of the consequences of a decrease in the monolithic behaviour of the structure under an earthquake due to poor anchorage of vertical reinforcement as well as corroding reinforcement.

5.2.12 R8 - Too low cover thickness of reinforcement in RCC bands

As discussed in Section 5.2.11, exposed reinforcement might lead to corrosion and consequently weakened material properties of the reinforcement and unwanted structural behaviour. The consequences of weakened reinforcement in the important RCC bands for the structural behaviour of the CSEB buildings could be that the strength of the box behaviour and the monolithic behaviour is lost or becomes less than intended. In these cases the response under earthquake load is unknown and the risk for vast damages in the event of an earthquake is higher.

5.2.13 O1 - Openings larger than specified (windows and doors)

The impact from enlarged openings with 15 cm on each side is shown in the FE-analysis to be insignificant compared to the ideal model. The wall parts in between the openings and the wall parts remaining to the edge are still wide enough to resist the applied loads.

The assumption about enlarged openings with a total of 30 cm might not always be the way this error is made in reality. If the openings would be enlarged significantly or enlarged in such a way that they lower the width of the wall part in between two openings or to a corner, the result might resemble the result in Section 4.3.14 more and therefore be more severe.

The displacements are in the same range as for the ideal model and therefor also here considered acceptable.

The advice to Build up Nepal is due to the result from this study that 30 cm enlarged openings in house design 2 from their design catalogue will not largely decrease the resistance of the building. Nevertheless, larger enlargements of the openings or in combination with openings too close to corner or other openings as well as enlargements in other designs might have a bigger impact of the resistance.

5.2.14 O2 - Openings too close to corners

In the FE-analysis with openings modelled close to corner a significant difference from the results of the ideal model can be seen regarding both displacements and stresses.

The bending tensile stresses S22 are not higher than in the ideal model but there are more areas where the bending tensile capacity is exceeded and the areas are larger. To move openings towards corners creates relatively weak columns in the corners and the wider and stiffer wall part therefore attract the stresses. As can be seen in Figure 4.13 and Appendix L that the areas with bending tensile stresses of the type S22 that exceeds the bending tensile capacity suggests that severe cracking of the CSEB masonry occurs.

To avoid damage to the CSEB building it is important to follow the directives in NBC 203:1994 (Government of Nepal 1994*b*) and the suggestion to Build up Nepal is to emphasise the importance of this in there drawing and instructions.

5.2.15 C1 - Too slender top RCC band between corner column and walls in design 2

When the CSEB building is modelled with RCC bands throughout the building with a height of 6 cm instead of 10 cm the stress concentrations go further down from the top RCC band into the CSEB masonry wall, as can be seen in Figure 4.14 and Appendix L. The maximum tensile bending stresses increased with almost 100 % due to the slender RCC bands and problems increase around openings and in corners of the walls. This is likely due to the lowered capacity to resist loads in the RCC band. It can also be seen that the lower RCC bands do not prevent the stresses to go further down in the wall as can be seen for the ideal model.

Another consequence caused by thinner RCC bands is that the RCC band between wall 1 (see Figure 4.1) and the corner column get large deflections. Where the ideal model got a maximum deflection in location 9, see Figure 4.2, of 0.73 mm the model with too slender RCC bands got a deflection in location 11 of 2.33 mm. As a reference value it can be said that according to IS 456:2000 (Earthquake Engineering Sectional Committee 2007) the total deflections of a concrete beam should not exceed the span divided by 350. For the RCC band with a length of 3000 mm the maximum allowed deflection becomes 8.6 mm. This means that the deflection of 2.33 mm is within the allowed deflection however this deflection does not take creep, effects of temperature nor shrinkage in to account.

A direct consequence of too slender RCC bands is that the reinforcement within the bands get exposed or that the concrete cover becomes to low, see Section 5.2.11. This will further decrease the strength of the RCC bands.

The RCC bands are an essential part of the resistance of the structural system of the CSEB buildings. A decrease of the capacity of the RCC bands will imperil the monolithic box behaviour of the CSEB buildings and its resistance to both earthquake and wind loads. The advice to Build up Nepal will therefor be to inform the locals to ensure that not too slender RCC bands are built.

5.2.16 C2 - RCC bands are made too thin

The model made in Section 4.3.15 was modelled with all through too slender RCC bands but was primarily looking at the effect of too slender RCC bands between the corner column and the walls in design 2. The effect from the trough all too slender corner columns can be transferred to other designs.

As described in Section 5.2.15 the too slender RCC bands weakens the resistance of the RCC bands and the loads goes further into the structure and the CSEB walls. Too slender RCC bands also risks exposing the reinforcement and in that way further weaken the RCC bands, see Section 5.2.11.

5.2.17 C3 - Too coarse aggregate in RCC bands

The use of the prescribed concrete mix is to prefer for the concrete elements in CSEB buildings. As stated in IS 456:2000 (Earthquake Engineering Sectional Committee 2007) should be chosen to have as large nominal size as possible but be able to fill corners of framework and adequately cover the reinforcement. One of the risk with an uncontrolled size of aggregate in the concrete for RCC bands is therefore that the reinforcement will be exposed and the consequences described in Section 5.2.12 might occur.

The material properties of the cured concrete might also differ if the prescribed concrete mix is not used. This could lead to a weakened concrete that cracks earlier than expected and decrease the resistance of the building. The recommendation to Build up Nepal is therefor to ensure the information to the locals about the consequences of too coarse aggregates in the concrete mix might have on decreased monolithic behaviour or that the mixture might not be able to flow down the vertical cavities in the CSEB with or without vertical reinforcement and decrease the resistance of the whole building.

5.2.18 RO1 - Roof is attached to walls in the wrong way

The purpose of the vertical reinforcement is to give the CSEB building a monolithic behaviour. The anchorage is important to assure that this behaviour is kept and that no pull out occurs when the structure is subjected to loads. Leaving the reinforcement sticking out jeopardises the anchorage and exposes the reinforcement as described in Section 5.2.11.

5.2.18.1 RO2 - Reinforcement bent around rafters and purlins

Bending the reinforcement around the roof structure could be considered as a type of anchorage of the vertical reinforcement. Furthermore, it might give structural unknown and unwanted repercussions. A more detailed study will have to be made on this type of anchorage of the roof and the vertical reinforcement to see if this is acceptable or if the repercussions are negative.

5.2.18.2 RO3 - Reinforcement welded to rafters and purlins

The same goes for this way of attaching the roof to the walls and anchoring the vertical reinforcement as discussed in Section 5.2.18.1.

5.2.19 RO4 - Rafters missing

As can be seen in Table 4.16 both the tensile bending stresses S11 and S22 exceeds their capacities. S22, of 0.255 N/mm^2, became significantly higher in relation the ideal model where the maximum tensile bending stress was 0.176 N/mm^2. As seen in Figure 4.15 the stress concentration for S22 is increased due to the missing rafters and therefore the high stresses also penetrates lower into the CSEB wall.

It can be concluded based on the analysis and the results shown in Table 4.16 that the maximum stresses mainly comes from load cases with wind loads, partly applied on the roof, when the CSEB building is constructed with fewer rafters than designed for. Furthermore, that the stresses have exceeded there capacities and the maximum stresses seen in the ideal model, see Table 4.2.

The analysis also shows that the tensile bending capacity S22 is exceeded in wall 2 (Figure 4.1) and in the gable walls, see Appendix L.

The recommendation is therefore to Build up Nepal that the correct number of roof rafters should be advised to the locals to sustain the buildings resistance during its service life.

5.2.20 X1 - Free standing corner column in design 2

It can be seen from the analysis in Section 4.3.20 that very large deflections, 11.06 mm, of the roof in z-direction occurs. These deflections are especially around the column and the missing RCC bands. According to IS 800:1984 (Earthquake Engineering Sectional Committee 1998) the deflections of a steel element should not exceed the span width divided by 325 which in this case, with a span of 1000 mm, gives a maximum allowed deflection 3.08 mm. This maximum allowed deflection is exceeded with about 260 % but it should be taken into account that the CGI sheets on the roof are not modelled. The CGI sheets would contribute to the stiffness of the roof and in that way lower the deflections. In this report the roof structure will not be looked further into.

The stresses due to this error does not increase significantly from the ideal model as can be seen in Table 4.18. But the stress concentrations spread in some walls, for example the gable wall, wall 6 in Figure 4.1. This could be because the corner column no longer contribute to the structural system and the box behaviour to the same extent as in the ideal model.

Larger areas of high stresses can be seen concentrated in the top of the corner column, which was not seen in the ideal model, see Appendix L. This is due to the load of the wall being supported solely on one point on the corner column and the walls around, in the area of the column and the intended RCC band to the column. This high stress concentration at the top of the corner column could be problematic since the missing RCC bands to the column could mean that the top RCC band on the corner column also is missed. This would mean that the roof is attached directly to the CSEB with significantly lower capacity than the RCC band possesses.

As described in Section 4.3.20, when this error is detected alternative solutions have been found. An alternative solution could be sufficient to make up for the missing RCC band but could also change the structural behaviour of the CSEB building in ways that cant be anticipated without adequate structural analysis. From this analysis it can be concluded that the important factors for an alternative solution to attach the corner column to the rest of the building is to include the column in the box behaviour and to protect the CSEB in the column from direct loading from the roof.

The provided drawings showing all the necessary RCC bands can be found difficult to read in Build up Nepal's design catalogue. The recommendation to them would therefor be to include either 3D-drawings of the building or elevation views where all the RCC bands are visible as well as informing the locals of the importance of all the RCC bands to fulfil the monolithic behaviour of the structure.

5.2.21 X2 - Gable wall made too high

In Table 4.19 it can be seen that the maximum displacements are slightly higher than for the ideal model. With the absolute maximum displacement of 1.01 mm during load case 7b in the x-direction. The drift limitations are nonetheless achieve with good margins.

The maximum compressive and bending tensile stresses measured for this FE-model are similar to the ideal model with some minor differences, see Table 4.20. The difference are believed to be due to new geometry of the building and applied loads, see Appendix I. As it can be seen in Figure 4.17 the bending tensile capacity is exceeded in wall 3 during load case 17 but the stress propagates over a smaller area than in the ideal model. This is likely to be due to the fact that the wind loads applied on the roof decreases with the new geometry of the gable walls. As seen in Appendix L the decreased area of exceeded capacity for stress S22 during load case 17 is also believed to be due to the decreased applied wind loads.

There is no defined way that gable walls are made too high but an assumptions was made that the configurations of the walls in the building will not change but rather just increased the height of the gable wall to enable an attic. As it can be seen in Section 2.3.2.4 the most prominent failure mode for gable walls are overturning. In this analysis both the displacements and the measured stresses are within the limitations so there is no immediate problems with a higher gable wall that can be seen in this study as long as it is reinforced correctly.

The advice to Build up Nepal is therefor that in house design 2 with an height increase of 500 mm of the gable wall there is no immediate increased risk for decreased resistance of the building as long as adequate reinforcement is provided. However, the results from this study is not sufficient to transfer the results to different house design and varied heights of gable walls.

5.2.22 X3 - Extra floor is added to building with small bricks

As it can be seen in Table 4.21 the displacements are increased with the additional storey that added to the ideal model in this error. The maximum displacements occur during load case 9a and 10b and reach 1.99 mm and 2.49 mm in the x- and y-direction respectively. Although they are increased compared to the ideal model they still satisfy the drift limitation.

The floor slab of this model has not been in the scope of analysis of the resistance of the building. All stresses and displacements have been extracted for the masonry walls and RCC bands from the FE-model. The maximum bending tensile and compressive stresses can be seen in Table 4.22. Compared to the ideal model the stresses are higher. This is an expected outcome due to the increased earthquake loads and due the fact that the wind loads are applied on bigger wall areas. It can be seen in Table 4.22 that there is no risk of crushing of the bottom CSEB units in the wall, which has been a concern from Build up Nepal if the smaller unit bricks are used for building two storey buildings. However, it can be seen in Figure 4.19 that bending tensile capacity S22 is exceeded over the terrace for load case 11a.

As it can be seen in Appendix L the bending tensile capacity is also exceeded in wall 1 during load case 17. During this load case it can be seen that the uplifting wind load on the roof influence the wall in the extent that the bending tensile capacity is exceeded at the locations of the anchorage of roof rafters and that the stress then propagates down in the wall in a similar manner as for the ideal model. In wall 2 it can be seen that a stress concentration occurs at the floor RCC band, at the location of where a perpendicular shear wall stands, and propagates out to the door opening. The building is stiffer at the location of the two perpendicular shear walls and thereby attain more of the stresses from the applied loads. The floor slab is also located at this elevation which increase the earthquake loads applied on the floor RCC band. In wall 3 during load case 17 then bending tensile capacity is exceed with a high resemblance to the stress propagation in the ideal model, see Appendix L.

Due to the fact the this error has not occurred yet, but is a problem that Build up Nepal can foresee for the near future, it is hence not clear on how the additional floor will be constructed on top of design 2. Therefore the assumption that the terrace would remain on the ground floor and window and door placements would take place on the first floor as well as the angle of the roof slope would remain the same.

The strong recommendation to Build up Nepal is therefore to inform and dissuade the locals from building an additional floor if the ground floor is build with the smaller brick size.

5.2.23 X4 - Damaged bricks are used

According to Maïni (2005) and as described by Sturm et al. (2014) the interlocking key of the CSEB provides shear capacity to the CSEB building. If the interlocking key of the CSEB is damaged then consequently the shear capacity of the CSEB building might be lowered. It can be discussed and analysed how much of the interlocking key that have to be damaged for the shear capacity to be lost of significantly lowered. It could also be discussed if a damage of the interlocking key is a sign of an overall weakened CSEB.

The advice to Build up Nepal inform the locals to use undamaged CSEB hence the reason for damaged bricks could decrease the resistance of the building, for example weakened characteristic strength of the bricks.

5.2.24 X5 - Roof band made all around building

The results of the maximum displacements from the FE-analysis can be seen in Table 4.23, -0.68 mm and -0.76 mm in the x- and y-direction respectively, which is close to the results from the analysis of the ideal model.

The maximum compressive and bending tensile tresses can be seen in Table 4.24. It can be seen in Appendix L that the stress propagation differ slightly from the ideal model, in particular in wall 1 due to the additional roof RCC band all around the building, which can be seen in Figure 4.20. Due to the additional RCC band the bending tensile capacity does not reach down on the sides of the window in wall 1 nor does it influence the column. The exceeded capacity of the bending tensile capacity in wall 3 reaches slightly further down in than compared to the ideal model. The additional roof band changes the slope of the roof and hence the wind loads on the roof, see Appendix I. The increased wind loads on the roof is believed to be the reason for the exceeded bending tensile capacity of stresses S22 in wall 6 and the bigger propagation of exceeded capacity in wall 3.

The advice to Build up Nepal is that this is not believed to be a crucial error for the overall resistance of the building. However, the lower slope of the roof increases the uplifting force from the strong winds which increases the importance of properly anchored roof rafters.

5.2.25 Q1 - How weak/strong are gable walls?

From the analysis of the ideal model no immediately problem of the gable walls are visible. The displacements, seen in Table 4.2, are well within the drift limitation and no big compressive nor tensile stresses are measured in the gable wall. According to Nepalese National Building Code NBC 203:1994 it is stated that all buildings with sloping roofs need to be constructed with a RCC band around the gable wall. The RCC band in the ideal model help to handle both the in-plane and out-of-plane stresses, see Section 2.3.3.2. The higher the gable wall the more important the RCC band around the wall end become due to the increased earthquake loads at higher elevations.

To ensure the quality of the RCC bands it is important to ensure that the bands are anchored correctly and integrated in the rest of the structure to attain a box behaviour of the building. In the context of the building sites in Nepal this might be harder to achieve for the RCC bands around the gable walls than the rest of the RCC bands due to the perforated profiles of the masonry end walls.

The advice to Build up Nepal will therefor be to inform the locals of the importance of properly constructed gable walls with RCC bands and vertical reinforcement, especially with an increased height of the gable wall. This is crucial due to the increased risk of overturning for end walls. Nevertheless, the gable walls should always be constructed according to drawings.

5.2.26 Q2 - Impact of higher and lower compressive strength of the CSEB than specified?

As presented in Table 4.25 the maximum displacements obtain in x-direction is -0.98 mm and 0.96 mm in y-direction. These displacements are well within the drift limitation stated in the Nepalese Building Codes. The higher displacements, compared to the ideal model, are due to the fact that the modulus of elasticity is decreased with the lower compressive strength of the bricks.

It can be seen in Table 4.26 that the maximum tensile and comrpessive stresses are slightly lower than in the ideal model. The FE-model made from bricks with a characteristic compressive strength of 2 MPa seems to reach approximately 20-40 % lower maximum stresses than attained in the ideal model. The reason behind this is likely to be due to the fact that the modulus of elasticity decreases with the compressive strength of the bricks. A FE-model to verify this was made where all the FE-model elements' modulus of elasticity were decreased with same percentage. The same stress propagation as in the ideal model was reached in this verification model which indicated that this is indeed the reason behind the lower stresses in the masonry although higher displacements are reached. This leads to the rest of the structure's elements, vertical and horizontal reinforcement as well as roof structure, will resist more of the applied loads and attain higher stresses. This will increase the quality assurance of the rest of the materials as well as the detail connections and anchorage of reinforcement.

Nevertheless, the bending tensile capacity S22 is exceeded in wall 1 and wall 3 during load case 17, which can be seen in Appendix L. The bending tensile capacity is exceeded due to the large uplifting wind loads. The stress distribution in the wall is decreased compared with the ideal model. The bending tensile stresses are distributed over a smaller area where the roof anchorage is located. This will increase the importance of proper anchorage of the roof rafters in the RCC band. Where the bending tensile capacity, S22, is exceeded horizontal cracks will occur.

The bending tensile capacity S11 is exceed as well in wall 3 during load case 15, which can be seen in Figure 4.21. In the area with exceeded tensile capacity vertical cracks are likely to occur.

The maximum displacements, for the FE-model made out of bricks with 10 MPa compressive strength, which can be seen in Table 4.27 are well within the drift limitation defined in the Nepalese Building Codes. The maximum displacement obtained in the x- and y-direction reach -0.77 mm and -0.68 mm respectively.

Presented in Table 4.28 it can be seen that the maximum compressive and bending tensile stresses in the building reach slightly higher values than in the ideal model, approximately 15-25 % higher. This is believed to be due to the same reasons as presented for the FE-model with bricks of 2 MPa compressive strength. The masonry wall will be stiffer due to the higher compressive strength and hence attract more stress than in the ideal model. It can be seen in Figure 4.22 that the gradient displaying the bending tensile stresses, in wall 1 for load case 17, reaches a bit further down in the wall and more of the walls bending tensile capacity S22 is exceeded. Additional exceeded bending tensile capacities for the remaining shear walls can be seen in Appendix L.

The recommendation to Build up Nepal would be to inform the locals and entrepreneurs of the importance of sufficient compressive strength of the CSEB to ensure the overall resistance of the building and to ensure the expected service life of the building.

5.2.27 Q3 - Dry stacking vs. use of mortar

From the analyse of error W1 an W2, see Section 5.2.1 and 5.2.2, it can be concluded that the use of mortar is to be recommended since the quality of a dry-stacked CSEB building is hard to ensure. Even if the use of mortar in general can make up for height imperfections, the importance of well laid CSEB and mortar layers have to be emphasised as well as an even and plane foundation as a basis to enable for levelled CSEB walls.

Even though the interlocking key of the CSEB provides an increased shear capacity in comparison with other bricks the use of masonry increase the shear capacity significantly compared to dry-stacked masonry structures. Additionally the mortar provides the CSEB-masonry structure with tensile strength which otherwise would be neglectably small.

The recommendation to Build up Nepal is thereby to always advice the locals and entrepreneurs to use mortar between the courses of CSEB due to its advantages discussed in Section 5.2.1 and 5.2.2.

5.3 Combinations of errors

The individual errors have been analysed but an important aspect is the reality in Nepal surrounding a building site. It has been observed that in the villages close to Kathmandu and Build up Nepal's office, where Build up Nepal actively can participate and supervise constructions very few errors have been detected. In remote villages where Build up Nepal can not be as participating in the construction, several errors have been detected in the same house.

5.4 Severe earthquakes

As described in Section 2.5.1 the level of earthquake load taken into account in the Nepal National Building Code, Indians Standards and other national building codes does not necessarily include the most severe earthquakes and their caused ground shaking intensity.

If the CSEB buildings are build according to regulations the intent is that they withstands without complete collapse but might suffer severe structural damage so that the buildings no longer are safe to live in but needs to be completely rebuilt.

This analysis have shown that the capacities of the CSEB masonry are exceeded, primarily the bending tensile stress S22, already under the level of earthquake loads prescribed by the National Building Code. This means that in the event of major earthquakes the impact on the CSEB buildings might exceed the impact shown in this study. Therefore, for a conservative approach the impacts in this study should be looked at with critical eyes.

5.5 Handover to Build up Nepal

The intention is that Build up Nepal should be able to use the findings of this study in their work to avoid errors being made and understand the severity of the errors to be able to direct there focus.

5.5.1 Checklist

Based on the analysis and after discussion with Build up Nepal a Checklist have been made. The Checklist can be seen in Appendix C and shows the errors that might occur in different phases of the construction of a CSEB building and illustrations of the consequences of the different errors.

The Checklist is designed together with Build up Nepal and is intended to be used by Build up Nepal's employees on site. The purpose of the Checklist is to detect errors before they are made by directing focus and creating awareness. The Checklist is designed based on the four phases of construction of a CSEB building, "foundation and plinth band", "walls openings and RCC bands", "roof" and "construction complete". The idea is that Build up Nepal's employees brings the Checklist on site, check what phase the construction currently is in, with the help of the Checklist see what errors are likely to occur during that specific phase and in that way be able to put focus on avoiding those specific errors. The illustrations in the Checklist will help the employees to describe the errors and their consequences to the people in the villages in a pedagogic way.

5.6 Continuation of this study

The aim of this report was to make a high level overview of the impact on resistance of errors in CSEB buildings. To deepen the understanding of the impact from different errors a more detailed analysis could be made, for example non-linear analysis to also understand, among other things, the crack propagation or use a micro model approach to get a more detailed understanding of the contribution from the CSEB and the mortar.

A continuation on the work presented in this report could be to present further means to avoid the errors being made. This could be down to a psychological level.

A study on how to retrofit the CSEB buildings when errors have been made or damages has occur could also be a continuation of the work presented in this report.

6
Conclusion

This report gives a high level overview of the structural resistance of CSEB buildings built with errors, in terms of stress and displacement analysis for the FE-analysis. As a first step in Build up Nepal's work to avoid these kind of errors, with the aim to build resistant and safe buildings, this is a good initial step.

Thirty-three errors were detected and documented in an error bank. The consequences of the errors were evaluated based on a literature study, and linear FE-analysis were performed on ten of the errors. Based on the results, recommendations are given to Build up Nepal, and a checklist for avoidance of errors is presented as a support to Build up Nepal's employees on site.

In general it can be concluded that with the impact of loads according to the Nepal National Building Code, complemented by the Indian Standards, the ideal CSEB building performs well. Wind loads determined by the Nepal National Building Code are relatively high and consequently load cases regarding wind gives the highest stresses and deflections in the CSEB buildings. The tensile bending stress in the structure exceeds the tensile capacity in the ideal model, indicating cracking, and more extensively in the models modelled with errors. It can also be concluded that drift is in general not a problem in relation to the drift limitation set by the Indian Standards. Furthermore, earthquakes regarded in the Nepal National Building Codes does not necessarily include the severest earthquakes and it should be noted that severe earthquakes could cause larger impacts on the structures than the impacts shown in this study.

In summary, the results and discussion concludes that it is essential for the CSEB buildings' structural resistance and structural integrity that the monolithic box behaviour of the structure is maintained. Crucial for the monolithic box behaviour is primarily well constructed RCC bands and consistent vertical reinforcement throughout the building.

The results and discussion handles errors occurring separately. In combination with the context of rural Nepal this is rarely the case, and the risk of combined errors should therefore be taken into consideration when reviewing the results.

Non-linear and/or a micro approach analysis needs to be carried out to get more extensive and unambiguous result. This report can serve as a base for further work, giving an, initial picture of the severeness of the impact from the errors, and in that way the computationally heavier modelling approaches could be used to study selected errors.

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A

Error Bank

ERROR BANK							
nr.	Error	Seen in villages	Design	Estimated Frequency (low-high)	Assumed cause	Comment from Build up Nepal	Question
WALLS							
W1	CSEB not leveled when build without mortar	Raleigh, Majhigaun	2	High	Uneven CSEB. Rough surface of CSEB. Small stones in between layers. Uneven foundation.	In the beginning mortar was not recommended to be used.	Does this impact the earthquake resistance?
W2	CSEB not leveled when build with mortar	Majhigaun	2,3	High	Uneven layer of mortar. Uneven height of bricks.	Now mortar is recomendet to be used. The need is questioned.	Does this impact the earthquake resistance?
W3	Missing mortar/concrete in some vertical wholes of the CSEB	Possibly all	Possibly all	Low	Done to reduce cost. Done due to lack of knowledge.		Does this impact the earthquake resistance?
W4	Hairline crack trough wall	Raleigh	1, 4	Low	Bricks not leveled when build without mortar. Weak bricks. Expanding concrete in wholes. Setting of foundation.	Hairline crack goes through whole wall or parts or the wall, but not through RCC-bands. Has led to that no more CSEB-buildings are build in that village due to a lack of trust in the technique.	What is the reason for the crack? Does this impact the earthquake resistance?
REINFORCEMENT							
R1	Insufficient lap length of rebars	Jyamrung	Possibly all	Medium	Due to a lack of knowledge. Saving material.	Should be 60xDiameter of rebar but is not always done.	Does this impact the earthquake resistance?
R2	Field bending of reinforcement	Jyamrung, Bakhang, Raleigh	Possibly all	High	Reinforcement from underneath does not fit the wholes of the CSEB.	Often done just over plint to fit reinforcement into bricks. Could be done up to 3 times with one rebar.	Does this impact the earthquake resistance?
R3	All vertical reinforcement starts at plinth band	Possibly all	Possibly all	Low	They decide for CSEB-building after foundation is made.	Should start from foundation. Solved by anchoring the rebars into the plinth band. If plinth band is done as well then an extra plinth band is cast where the reinforcement is anchored.	Does this impact the earthquake resistance?
R4	Vertical reinforcement missing	Kaileri	Possibly all	Low	Due to other plan from the start or for cost reasons. Due to lack of knowledge.	Reinforcement is skipped compleately or use in corners only.	Does this impact the earthquake resistance?
R5	Appropriate stirrup in RCC band missing	Raleigh	Possibly all	Medium	Due to lack of knowledge. Due to lack of tools.	They use simple metal wire instead of reinforcement hooks. They use straight parts of reinforcement bars atached with wire to main reinforcement.	Does this impact the earthquake resistance?
R6	Insufficient radius of stirrup	Raleigh, GoodWeave	Possibly all	Medium	Due to lack of knowledge. Due to lack of tools.	The angle of the stirrups seem to be insufficient.	Does this impact the earthquake resistance?
R7	Reinforcement exposed to environment (not bent in to the top RCC band)	Jyamrung, Majhigaun, Raleigh	2, 3	High	Unclear drawings. See the possibility to attach roof to the vertical reinforcement.	Are left to stick up from top RCC-bands.	Does this impact the earthquake resistance?
R8	Too low cover thickness of reinforcement in RCC bands	Majhigaun	2	Medium	RCC bands are not mde properly. Due to a lack of knowledge.	In combination with to coarse aggregate in the concrete and to thin RCC bands.	Does this impact the earthquake resistance?
OPENINGS							
O1	Openings larger than specified (windows and doors)	Possibly all	Possibly all	Medium	Wish for larger openings. Error in production of window and door frames.	Larger than the percentage allowed.	Does this impact the earthquake resistance?
O2	Openings to close to corners	Possibly all	Possibly all	Medium	Due to lack of knowledge. Wish to alter drawings.	600 mm is min. Sometimes build closer to corner.	Does this impact the earthquake resistance?
RCC-BAND							
C1	Too slender lintel RCC bands between corner column and walls in design 2	Majhigaun	2	Medium	Unclear drawings. Saving material. Same error throughout the building.	RCC band is made 6-7 cm but should be 10 cm.	Does this impact the earthquake resistance?
C2	Sill and lintel RCC bands are made too thin	Raleigh, Majhigaun	2	Medium	Faulty framework. Saving material.	RCC band is made 6-7 cm but should be 10 cm.	Does this impact the earthquake resistance?
C3	Too coarse aggregate in RCC band	Raleigh	Possibly all	Medium	Due to lack of knowledge. DUe to lack of proper ingrediens of the concrete mix.	To coarse aggregate have ben seen used in the cement for the RCC bands.	Does this impact the earthquake resistance?

FOUNDATION							
F1	Mud mortar is used in foundation instead of cement mortar	Possibly all	Possibly all	Low	Due to lack of knowledge. Saving money.	Cement mortar should be used but sometimes mud mortar is used instead.	Does this impact the earthquake resistance?
F2	Through-stones missing in stone masonry foundation	Possibly all	Possibly all	Low	Due to lack of knowledge. Saving money. Due to lack of material.	Bigger stone prevents the foundation from splitting.	Does this impact the earthquake resistance?
F3	Corner stone is missing in foundation	Kalleri	Possibly all	Low	Due to lack of knowledge. Saving money. Due to lack of material.	Has been seen missing.	Does this impact the earthquake resistance?
F4	Foundation too shallow	Majhigaun	2	Low	Due to lack of knowledge. Saving money. To reduce working hours.	Has been spotted in a few cases and stoped. Risk that it has happened and not been detected.	Does this impact the earthquake resistance?
ROOF							
RO1	Roof is attached in the wrong way to the walls	Majhigaun, Mulabari	2, 3	Hig	Saving money. Reduce working hours.	Which methods are acceptable? (better than not attached at all..)	Does this impact the earthquake resistance?
RO2	Reinforcement bent around rafters and purlins	Majhigaun	2, 3	High	Saving money. Reduce working hours.	Rebar should be bend into RCC band. Roof should be attached according to detail in drawings. Seem to be the common way to attach the roof.	Does this impact the earthquake resistance?
RO3	Reinforcement welded to rafters and purlins	Bakrang, GoodWeave, Majhigaun, Raleigh	2	Medium	Saving money. Reduce working hours.	Rebar should be bend into RCC band. Roof should be attached according to detail in drawings. Has been seen as a method to attach the roof.	Does this impact the earthquake resistance?
RO4	Rafters missing	Possibly all	2	Medium	Saving money. Unclear drawings.	Has been seen in quite a few cases.	Does this impact the earthquake resistance?
OTHER							
X1	Free standing corner column in design 2	Jyamrung, Dadaguan, GoodWeave, Mulabari, Raleigh	2	High	Unclear drawings.	There are different types of this error and different solutions for this?	Does this impact the earthquake resistance?
X2	Gable wall made to high	Possibly all	2	Medium	Want an attic	They make it higher to get an attic.	Does this impact the earthquake resistance?
X3	Adding an extra floor afterwards, on house with small bricks that isn't approved for 2 stories.	Possibly all	Possibly all	Medium	Lack of knowledge. Want to get the grant but want and extra floor to an as low cost as possible.	This will be done after they recieve grant from the government. Then they might not care that the building made with the smalles size of CSEB is not approved for 2 floors.	Does this impact the earthquake resistance?
X4	Damaged Bricks	Dadaguan	Possibly all	Medium	Saving money. Due to lack of knowledge.	Damaged bricks are used.	Does this impact the earthquake resistance?
X6	Roof band made all around building	Majhigaun	2	Medium	Unclear drawings. Want an additional low attic.	Consequently the roof angle is changed and the gable walls become higher.	Does this impact the earthquake resistance?
QUESTIONS							
Q1	How weak/strong are gable walls?	-	-	-	-	The government thinks gable walls are weak. Is that true with CSEB?	Can this comment from the government be ignored?
Q2	Impact of higher and lower compressive strength of the CSEB blocks than specified?	-	-	-	Soil quality. Amount of material in press in production.	A genereal lower compressive strength of the building. Certain parts a lower compressive strength.	Does this impact the earthquake resistance?
Q4	Dry stacking vs. use of mortar	-	-	-	-	Initially drystacking was recomended. Now the use of mortyar is recomendes based on Auroville's instructions.	Does this impact the earthquake resistance? Can either one cause damages?

B

Picture Bank

W1

W2

W4

R1

R2

R4

R5

R6

R7

R8

C1 & C2

C3

F3

F4

RO1-3

R4

X1

X4

X5

C

Checklist

On Site Checklist

Aim

This document is intended to be used by the employees of Build up Nepal. The document should be brought on site during the different phases of construction of a CSEB building with the purpose to help inform the locals about what different choices during construction can lead to in terms of damages and risks. The document also helps in the process of checking if different common errors have occurred and then to, in a systematic way, transfer that information back to the management.

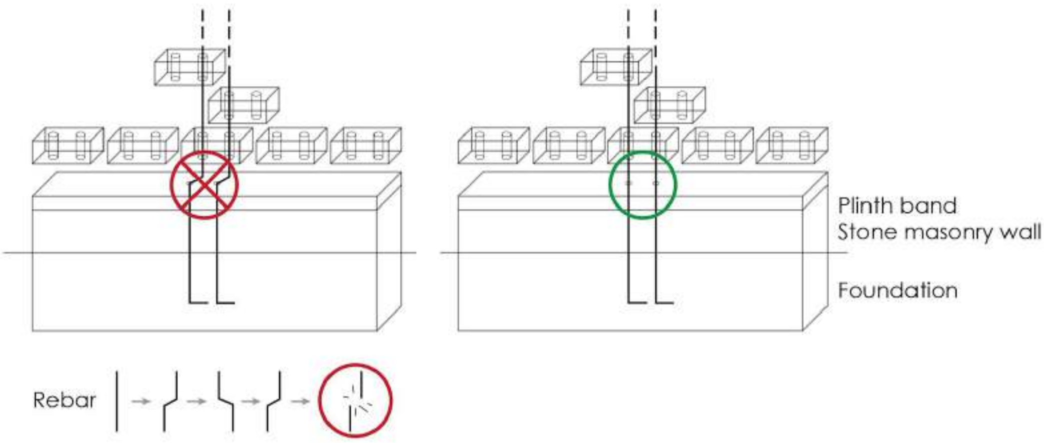
Instructions

When on site, use this document as a help to inform the locals about consequences of different construction choices.

Also use this document to check, in each phase, if one or more of the common errors have occurred. Tick the box next to the title of the error if it has occured. See example below.

Example:

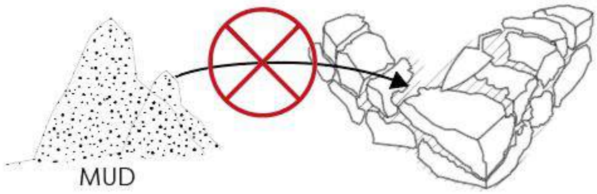
R2 – Are rebars being bend in the field? ☐

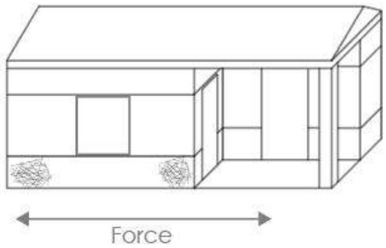


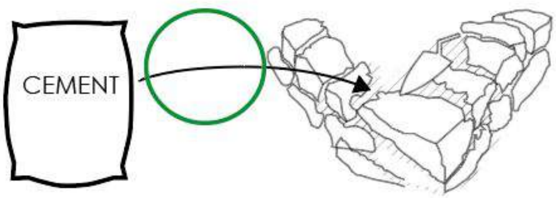
Phase 1 – Foundation and plinth band

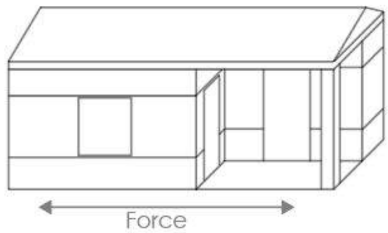
<u>Foundation</u>

F1 – Is mud mortar used instead of cement mortar? ☐



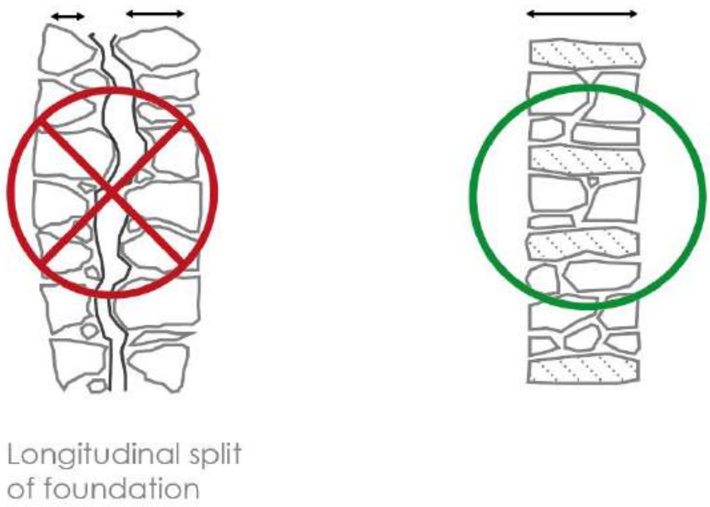




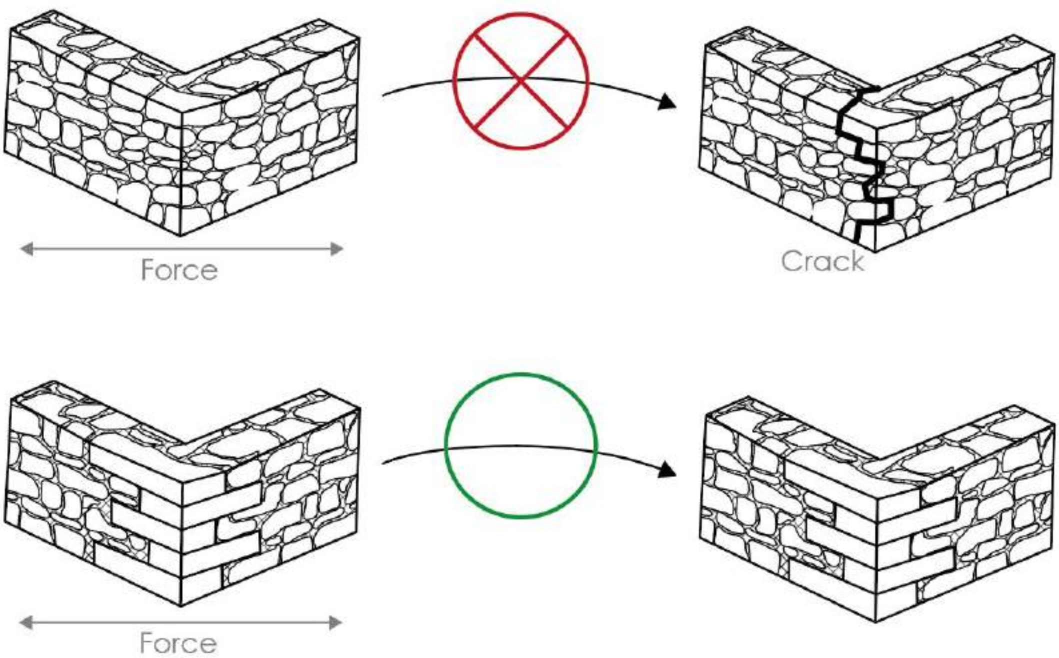


F2 – Are through stones missing in stone-masonry foundation? ☐

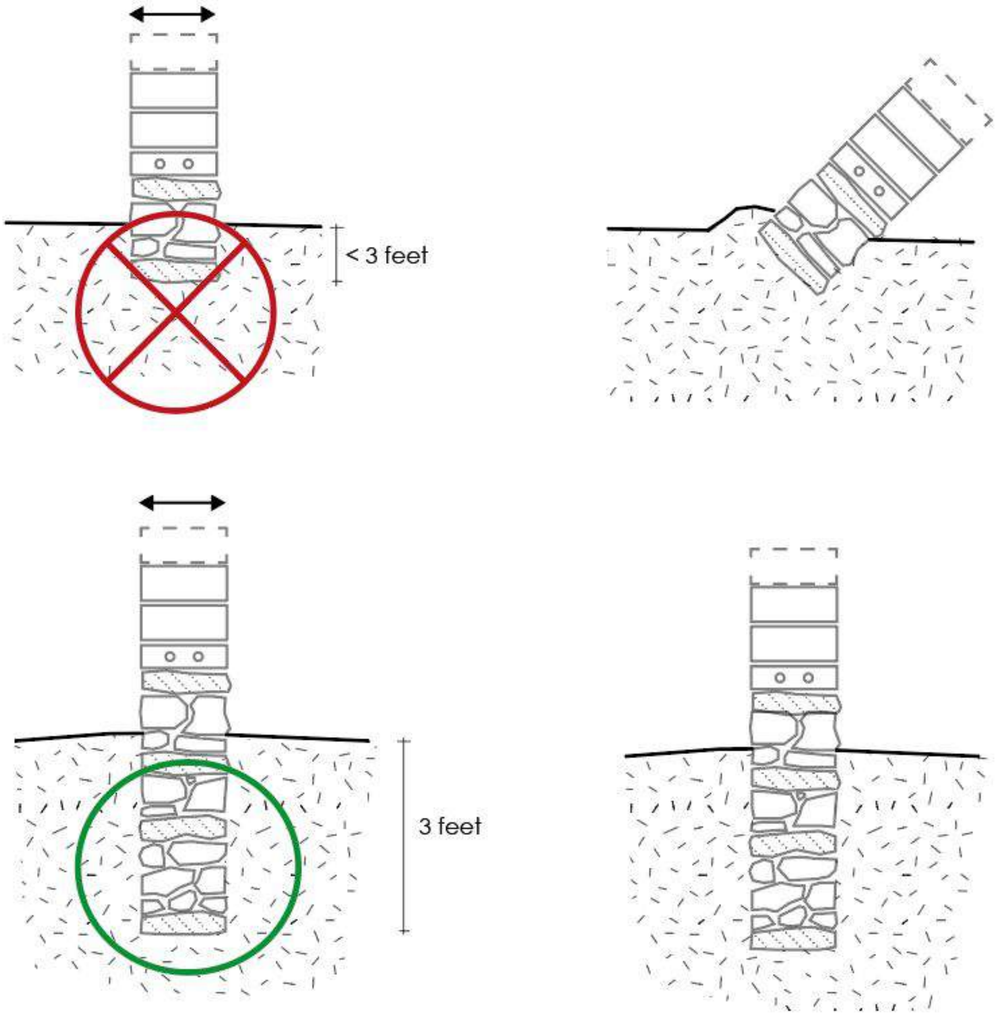
Top view:



F3 – Are cornerstones missing in foundation? ☐

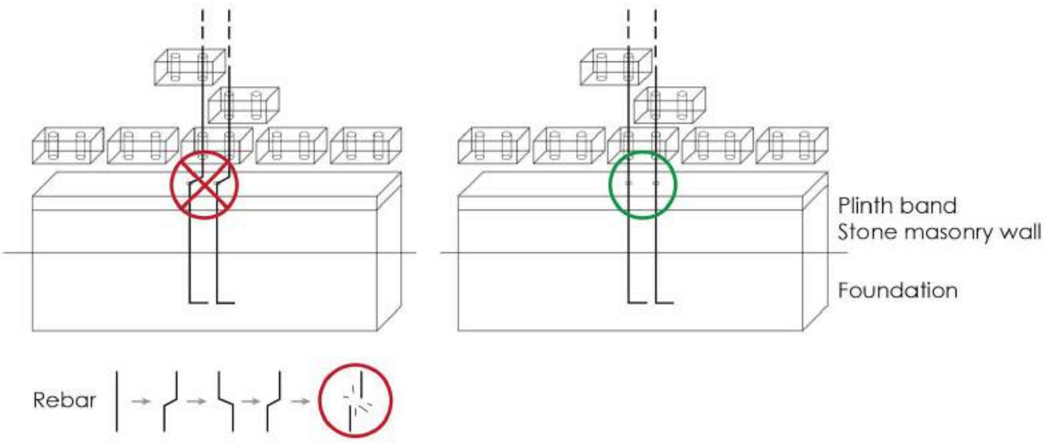


F4 – Is the foundation made deep enough? ☐

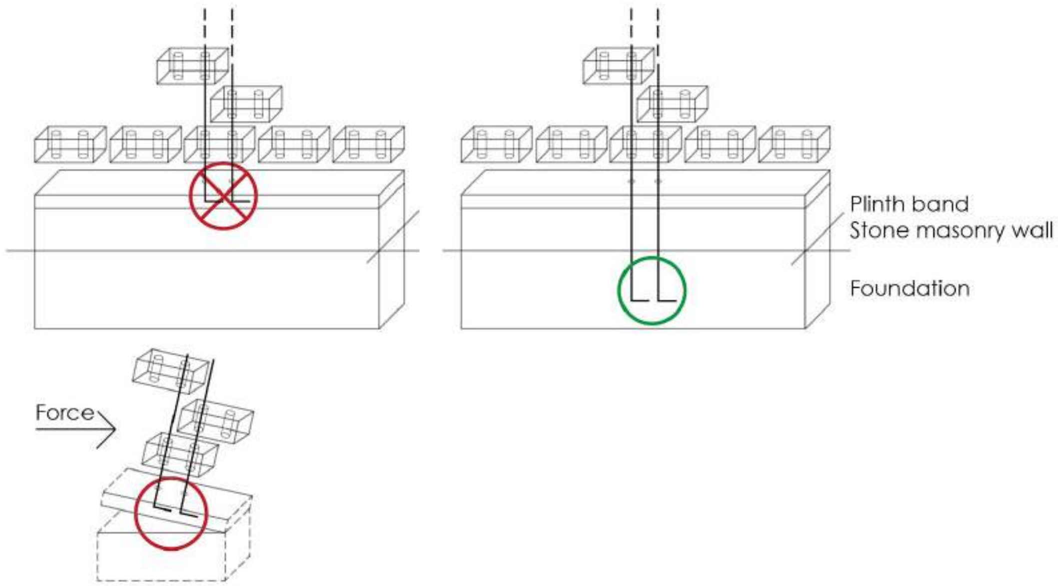


Reinforcement

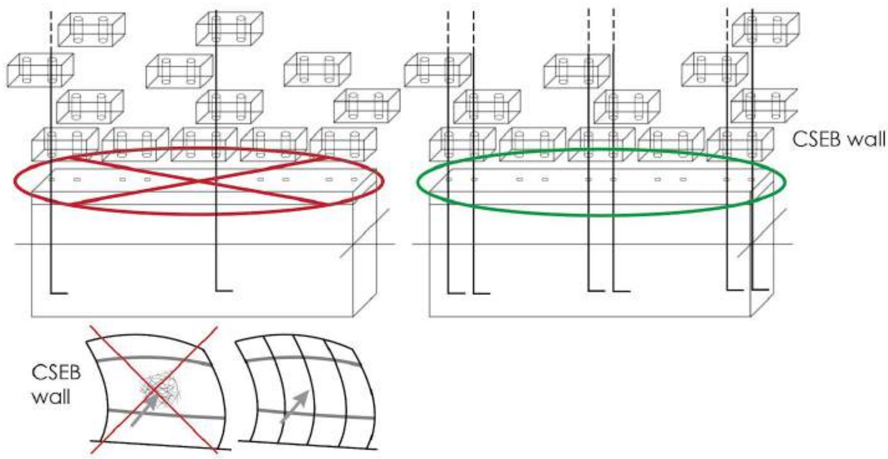
R2 – Are rebars being bend in the field? ☐



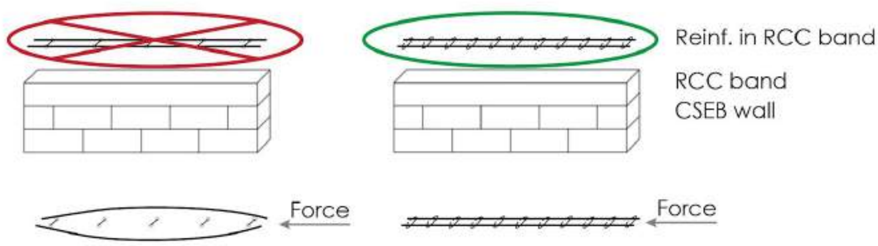
R3 – Is the reinforcement starting from plinth band and not anchored in the foundation? ☐



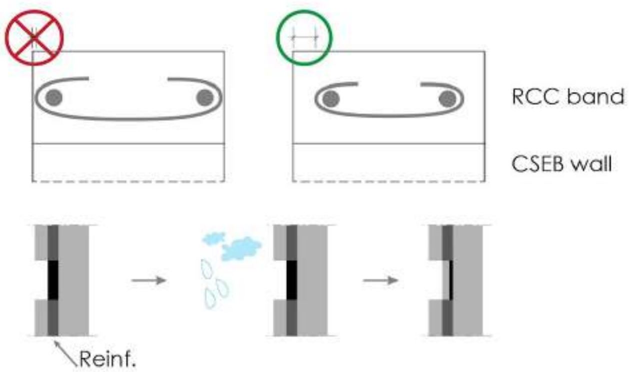
R4 – Are vertical reinforcement missing in some places?



R5/6 – Are the accurate type of stirrup used?

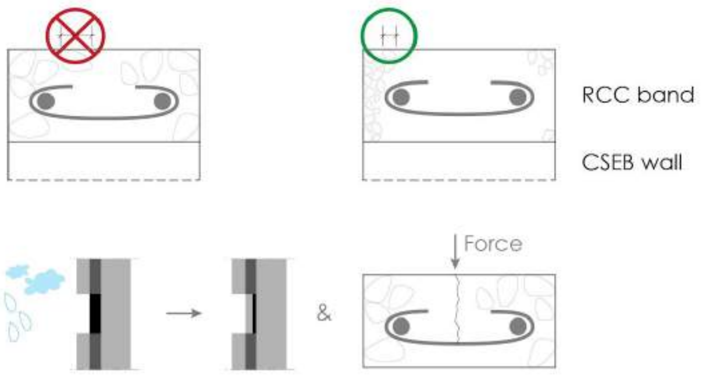


R8 – Is the cover thickness to low in the RCC bands?



RCC-bands

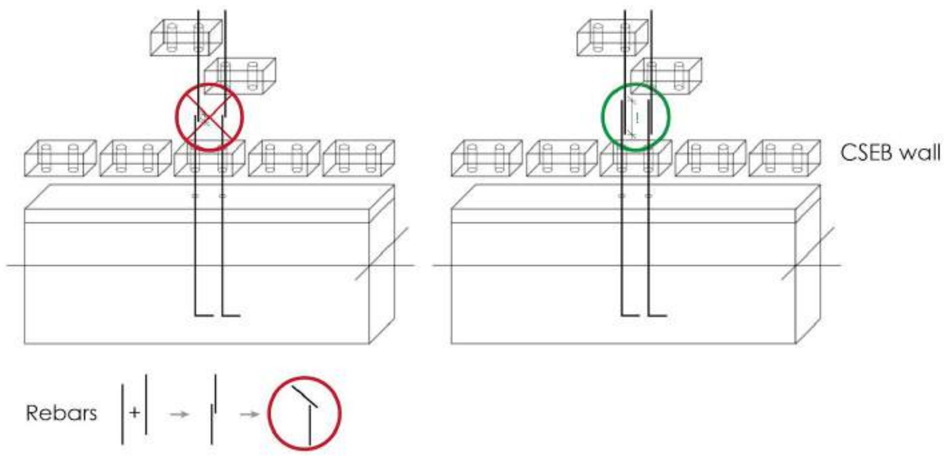
C3 – Is the aggregate in the concrete to coarse? ☐



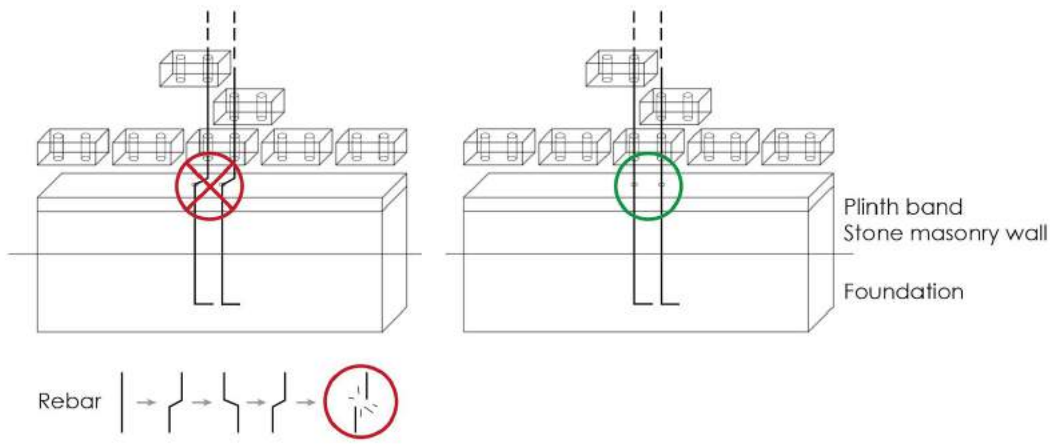
Phase 2 – Walls, openings and RCC-bands

Reinforcement

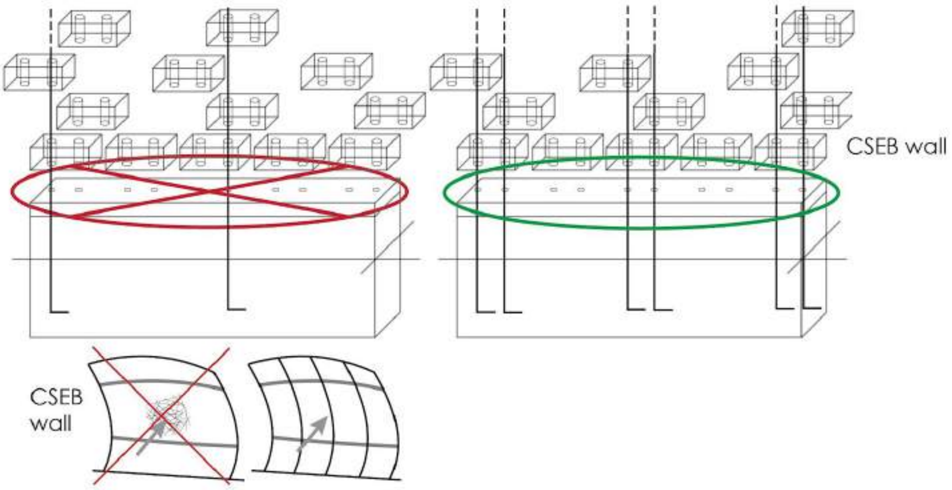
R1 – Are the reinforcements lap length long enough? ☐



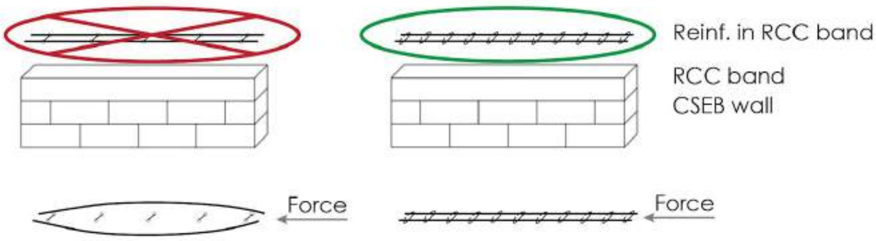
R2 – Are rebars being bend in the field? ☐



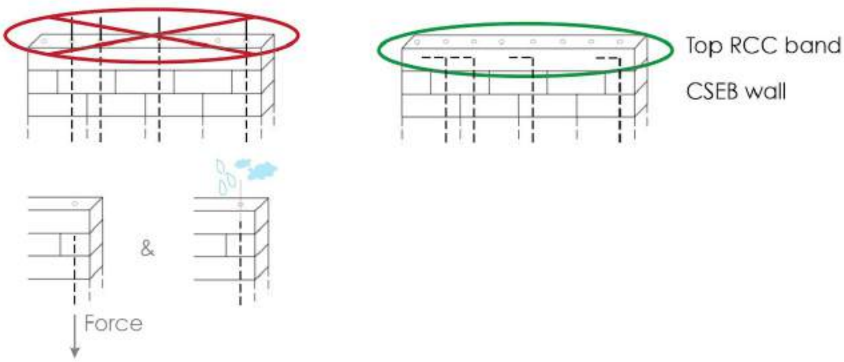
R4 – Are vertical reinforcement missing in some places? ☐



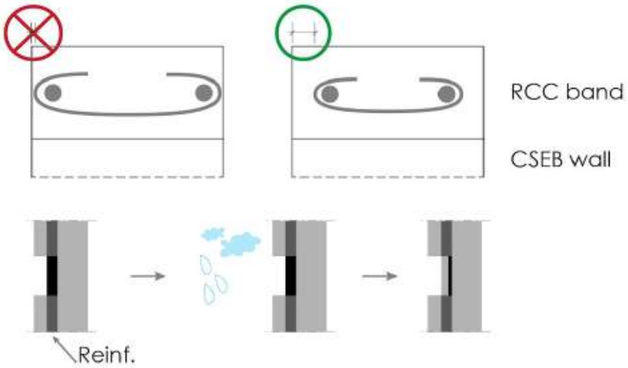
R5/6 – Is the accurate type of stirrup used? ☐



R7 – Are reinforcement left to stick up from the top RCC-bands? ☐

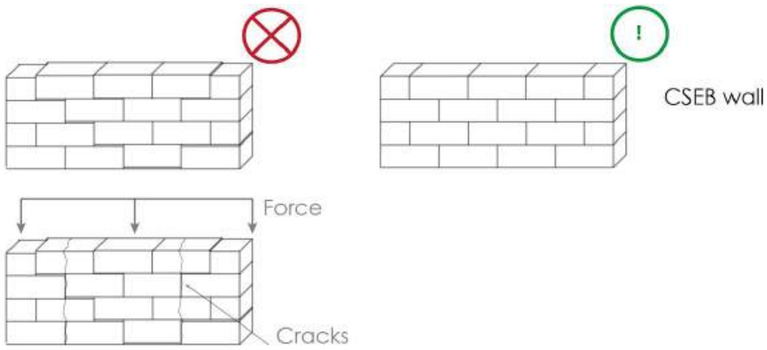


R8 – Is the cover thickness to low in the RCC bands? ☐

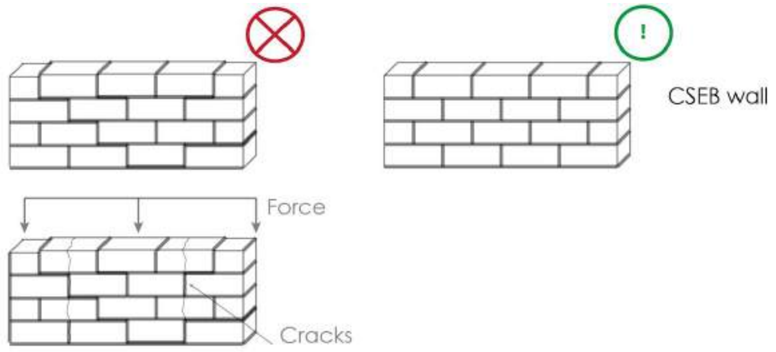


Walls

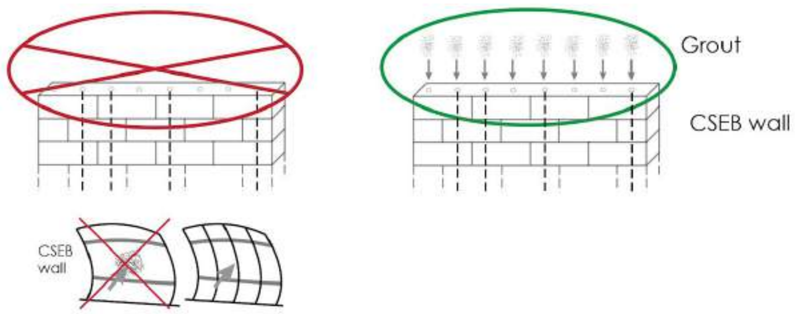
W1 – Are the CSEB laid in level when build without mortar? ☐



W2 – Are the CSEB laid in level when build with mortar?

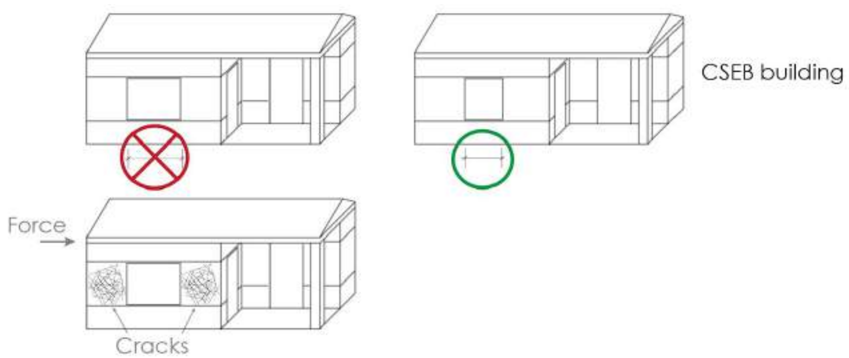


W3 – Is concrete missing in some vertical holes?

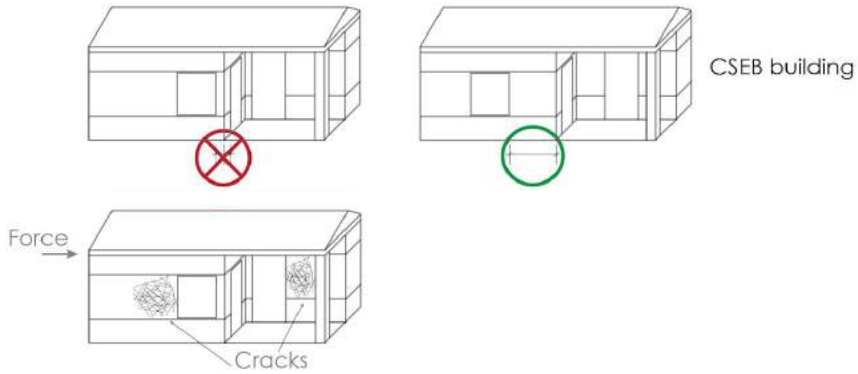


Openings

O1 – Are openings larger than on drawings?

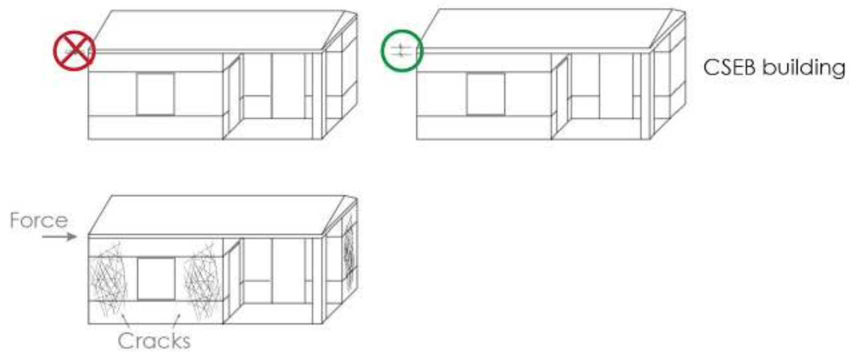


O2 – Are openings closer than 60 cm to corners?

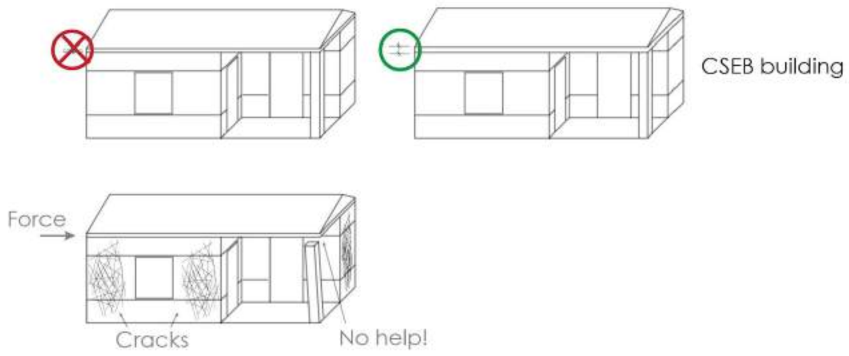


RCC-bands

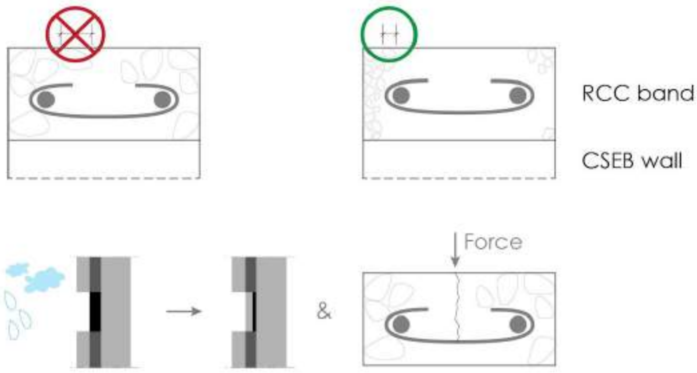
C1 – Are RCC bands made to thin (should be 100 mm)?



C2 – Are RCC bands supposed to attach to the corner column (design 2) made to thin? ☐

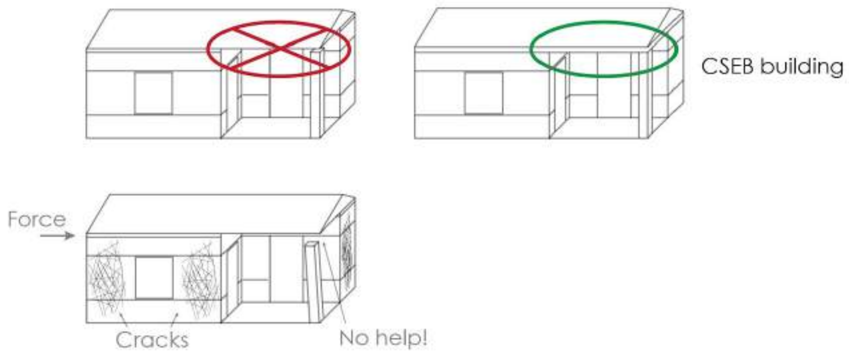


C3 – Is the aggregate in the concrete to coarse? ☐

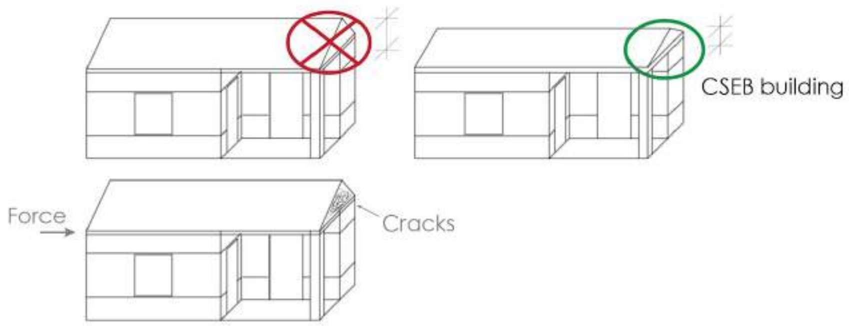


Other

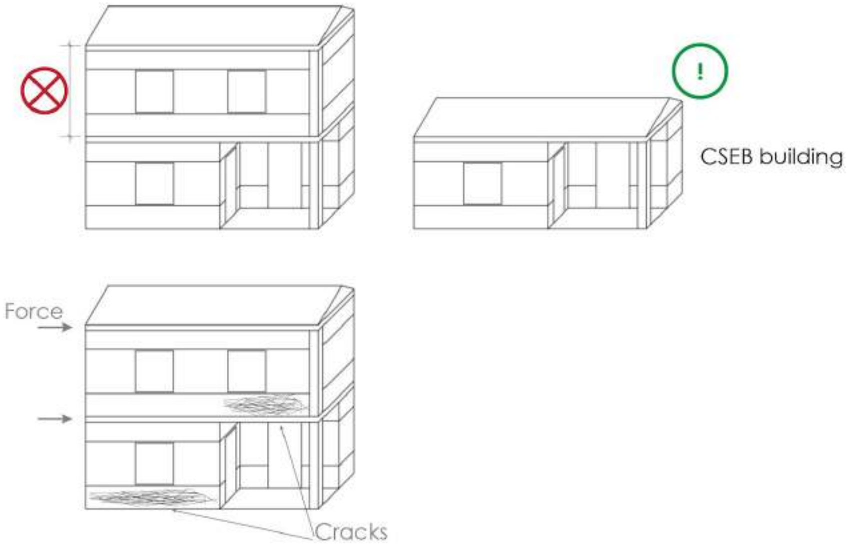
X1 – Are RCC bands supposed to attach to the corner column (design 2) missing? ☐



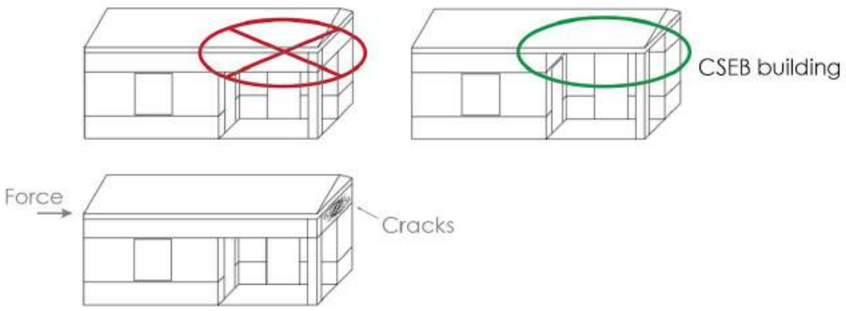
X2 – Are gable walls made to high? ☐



X3 - Is there a plan to add another floor to the building made with small bricks? ☐



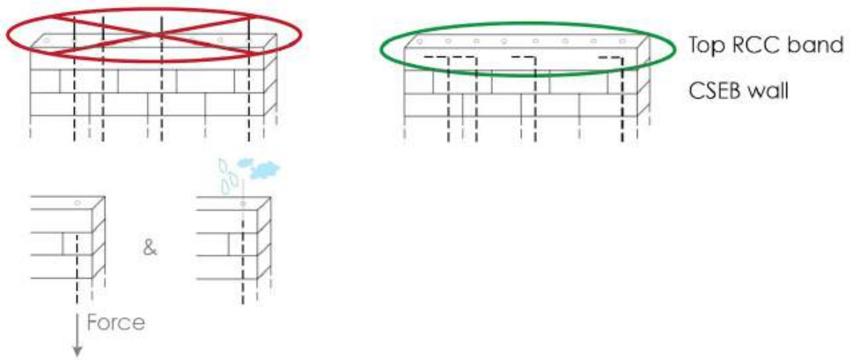
X5 - Is the roof band made all the way around the building (in Design 2) ? ☐



Phase 3 – Roof

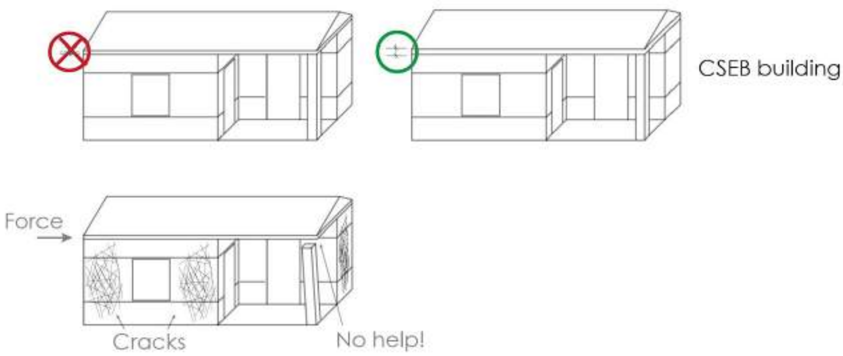
Reinforcement

R7 – Are reinforcement left to stick up from the top RCC-bands? ☐



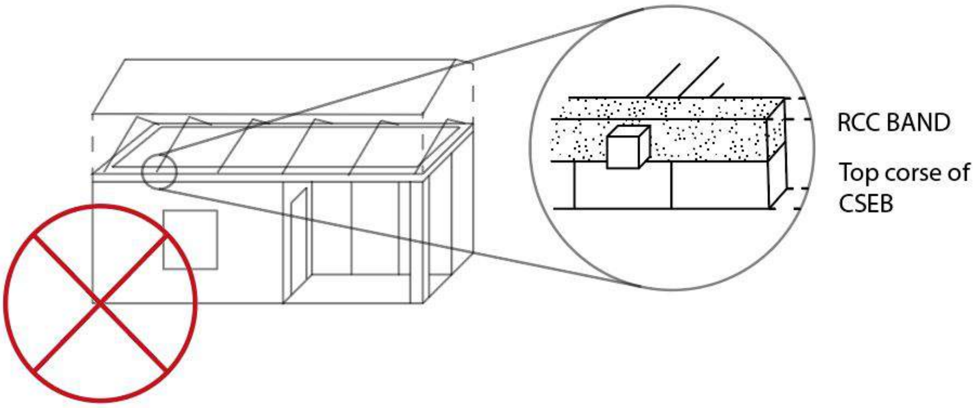
RCC-bands

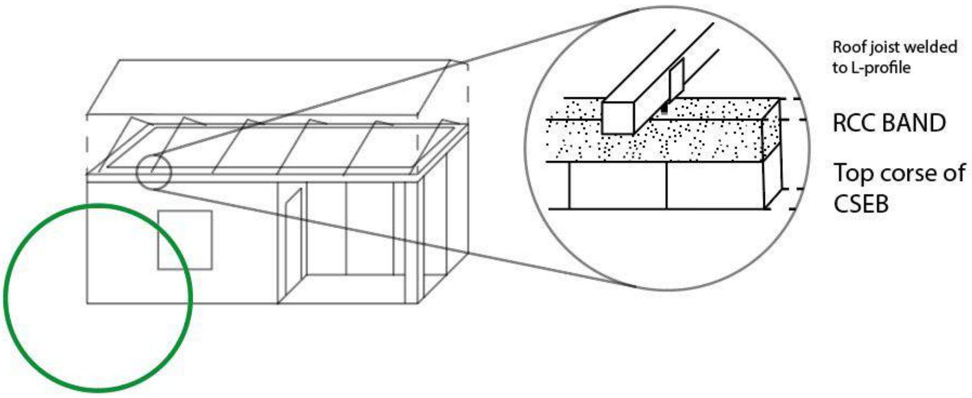
C2 – Are RCC bands supposed to attach to the corner column (design 2) made to thin? ☐



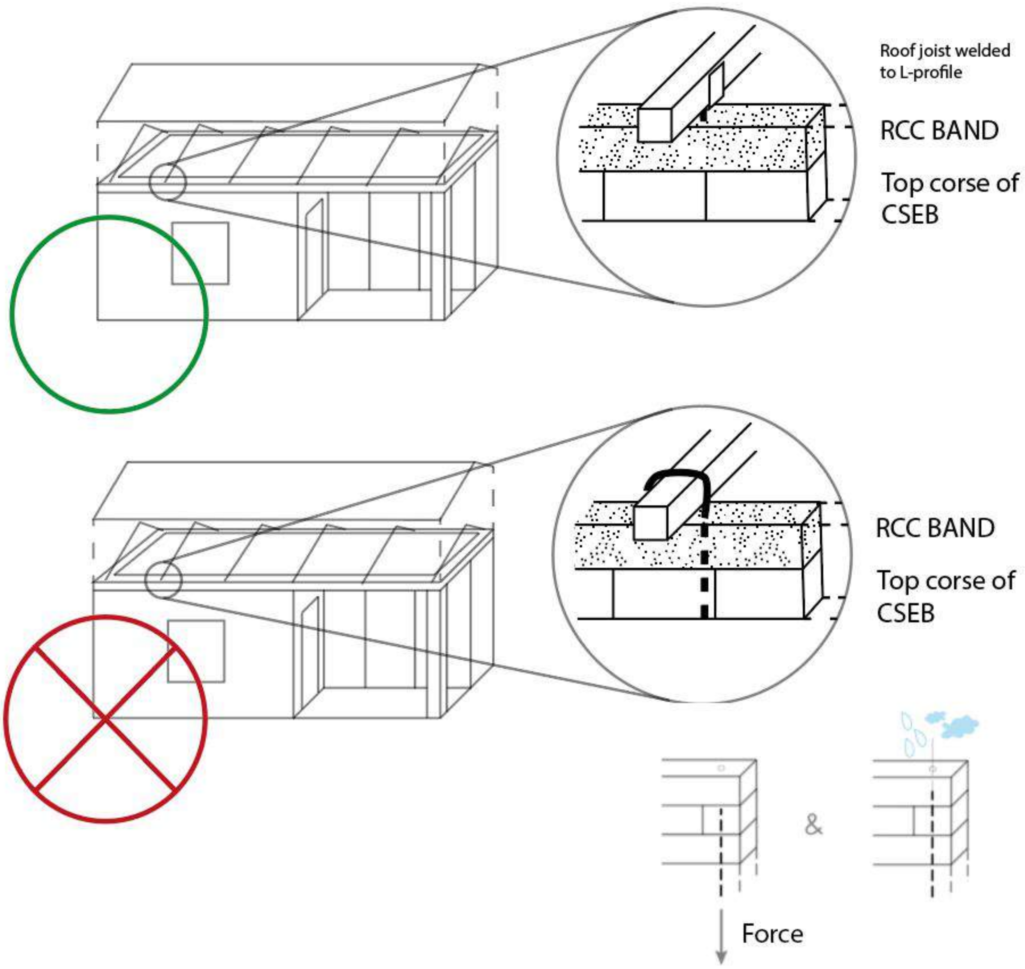
Roof

RO1 – Is the roof not attached to the walls according to drawings? ☐

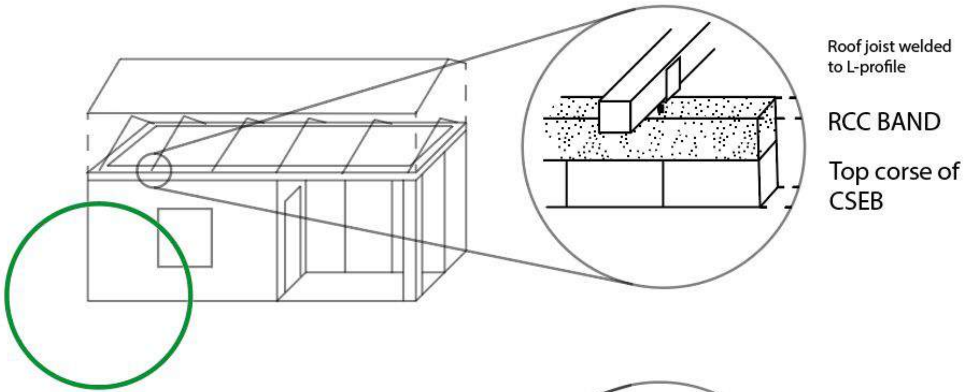


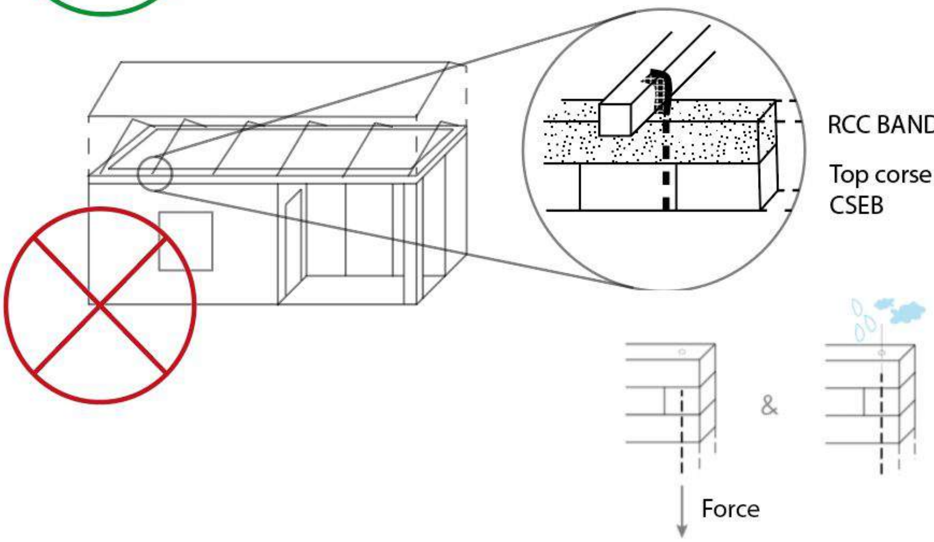


RO2 – Is the roof attached to the walls by the rebars being bent around the rafters? ☐

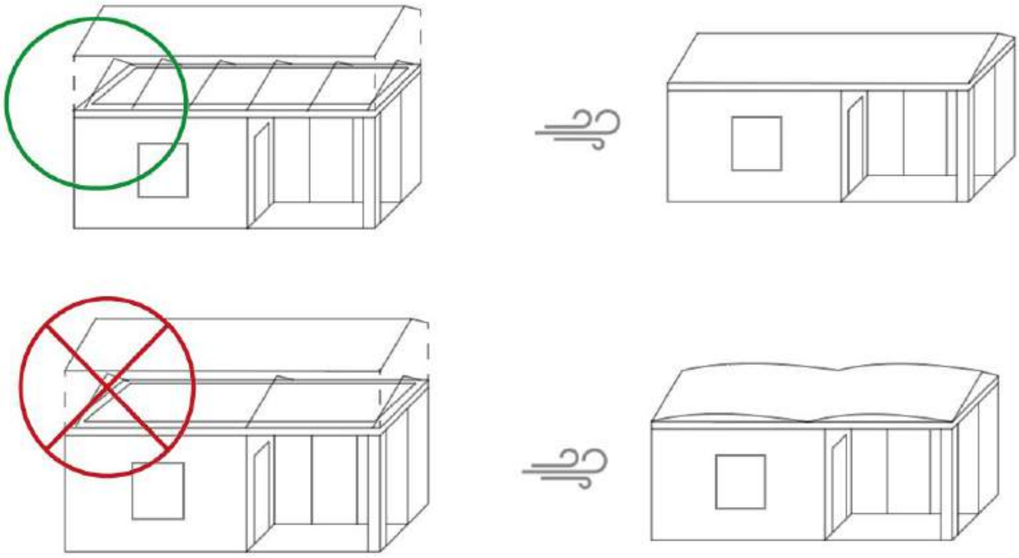


RO3 - Is the roof attached to the walls by the rebars being welded to the rafters? ☐



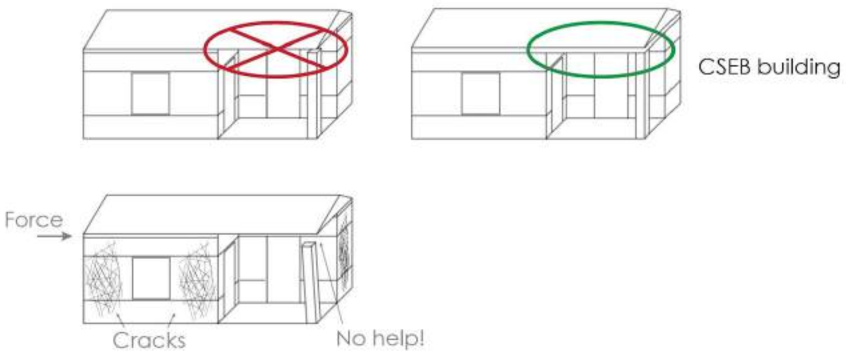


RO4 - Are roof rafters missing?

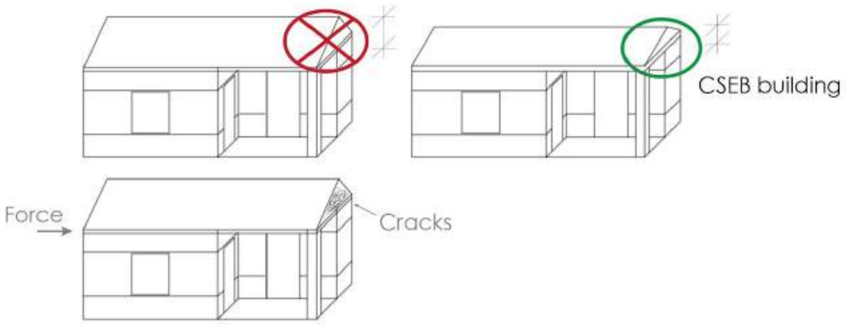


Other

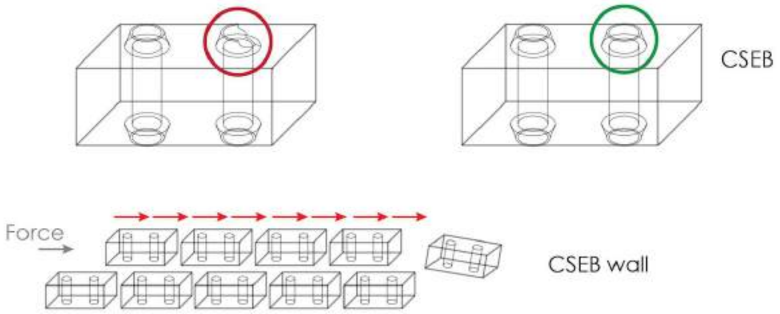
X1 – Are RCC bands supposed to attach to the corner column (design 2) missing?



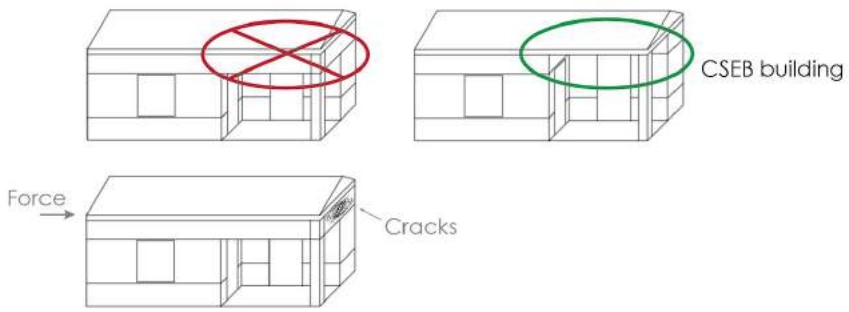
X2 – Are gable walls made to high?



X4 – Are the bricks used damaged?



X5 - Are the roof bands made all the way around the building?



Phase 4 – Construction complete

Walls

W4 – Is there a hairline crack through the wall?

Please answer the following questions and inform the management.

Is the CSEB wall build with mortar? Yes No

Where does the crack start and where does it end?

____

____

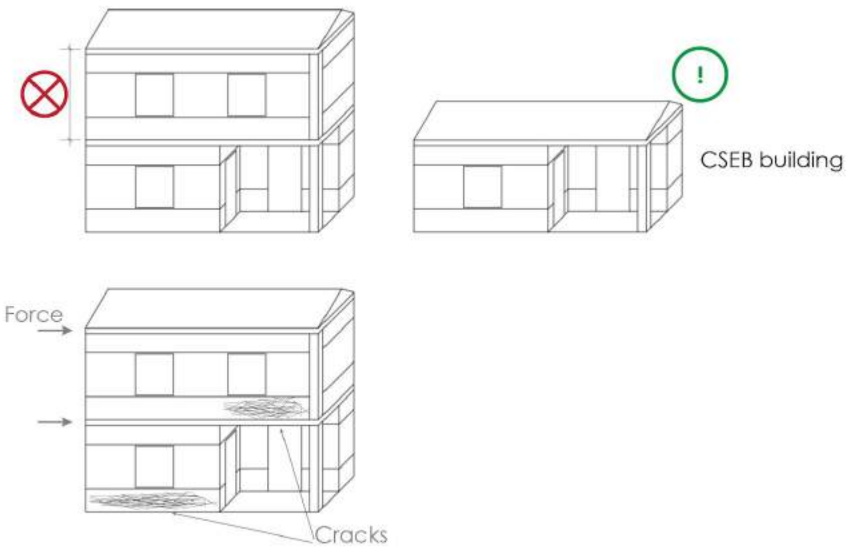
How far through the wall does the crack go?

____

Is the crack visible on the inside and outside of the wall? Yes No
If no, on which side is it visible? ______________________

Other

X3 - Is there a plan to add another floor to the building made with small bricks? ☐



D

Material properties in SAP

Table D.1: Material properties in SAP model

Material model	**Value**	**Unit**
Concrete, M20 (IS 456:2000)		
Weight per Unit Volume	24.9926	kN/m^3
Mass per Unit Volume	2.5486	$10^3 kg/m^3$
Modulus of Elasticity, E	22360680	kPa
Poisson, U	0.2	$-$
Coeff. of Thermal Expansion, A	$5.5 * 10^{-6}$	$-$
Shear modulus, G	9316950	kPa
Specified Concrete Compressive Strength, fc	20000	kN/m^2
Expected Concrete Compressive Strength, fc	20000	kN/m^2
Masonry		
Weight per Unit Volume	19	kN/m^3
Mass per Unit Volume	1.9375	$10^3 kg/m^3$
Modulus of Elasticity, E	476000	kPa
Poisson, U	0.15	$-$
Coeff. of Thermal Expansion, A	$9.9 * 10^{-6}$	$-$
Shear modulus, G	206956	kPa
Specified Concrete Compressive Strength, fc	3400	kN/m^2
Expected Concrete Compressive Strength, fc	3400	kN/m^2
Reinf. Steel, Fe500 (IS 1786:2008)		
Weight per Unit Volume	76.9729	kN/m^3
Mass per Unit Volume	7.849	$10^3 kg/m^3$
Modulus of Elasticity, E	200000000	kPa
Poisson, U	0.3	$-$
Coeff. of Thermal Expansion, A	$1.17 * 10^{-5}$	$-$
Shear modulus, G	76923077	kPa
Minimum Yield Stress, fy	500000	kN/m^2
Minimum Tensile Stress, fu	545000	kN/m^2
Expected Minimum Yield Stress, fy	550000	kN/m^2
Expected Minimum Tensile Stress, fu	599500	kN/m^2
Structural Steel, Fe250 (IS 800:2007)		
Weight per Unit Volume	76.9729	kN/m^3
Mass per Unit Volume	7.849	$10^3 kg/m^3$
Modulus of Elasticity, E	210000000	kPa
Poisson, U	0.3	$-$

Coeff. of Thermal Expansion, A	$1.17 * 10^{-5}$	$-$
Shear modulus, G	80769231	kPa
Minimum Yield Stress, fy	250000	kN/m^2
Minimum Tensile Stress, fu	410000	kN/m^2
Expected Minimum Yield Stress, fy	275000	kN/m^2
Expected Minimum Tensile Stress, fu	451000	kN/m^2

E

Verification of model

Verification of SAP modelling:

Displacement for simply supported beam:

$L = 2\ m$

$E_c = 700\ MPa$

$b = 0.1\ m$
$h = 0.15\ m$

$q = 10\ \frac{kN}{m}$

$I = b \cdot \frac{h^3}{12} = 2.813 \cdot 10^{-5}\ m^4$

$p = 5 \cdot L^4 \cdot \frac{q}{384 \cdot E_c \cdot I} = 105.82\ mm$

Displacement at mid span from SAP model,
unreinforced
$p_{sap} = 105.578\ mm$

Displacement at mid span from SAP model,
reinforced
$p_{sap} = 105.054\ mm$

F
Average weight of walls

Seismic weight calculations for a CSEB -building

Material properties

$\rho_{CSEB} = 19 \ \frac{kN}{m^3}$ $\qquad \rho_s = 76.97 \ \frac{kN}{m^3}$ $\qquad \rho_c = 24.99 \ \frac{kN}{m^3}$

Area properties

$t_{wall} = 0.15 \ m$

$b_{CSEB} = 0.3 \ m$ $\qquad h_{CSEB} = 0.15 \ m$

$d_s = 0.01 \ m$ $\qquad r_s = \frac{d_s}{2} = 0.005 \ m$

$d_c = 0.05 \ m$ $\qquad r_c = \frac{d_c}{2} = 0.025 \ m$

$A_s = \pi \cdot r_s^2 = 7.854 \cdot 10^{-5} \ m^2$ Area of reinforcement

$A_{c1} = \pi \cdot r_c^2 = 0.002 \ m^2$ Area of concrete w.o. reinforcement

$A_{c2} = A_{c1} - A_s = 0.002 \ m^2$ Area of concrete around reinforcement

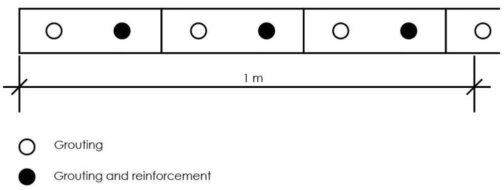
$A_{CSEB1} = b_{CSEB} \cdot h_{CSEB} = 0.045 \ m^2$

$A_{CSEB} = A_{CSEB1} - A_{c1} = 0.043 \ m^2$ Area of CSEB

$W_{CSEB} = A_{CSEB} \cdot \rho_{CSEB} = 0.818 \ \frac{kN}{m}$

$W_c = A_{c1} \cdot \rho_c = 0.049 \ \frac{kN}{m}$

$Ws = A_s \cdot \rho_s + A_{c2} \cdot \rho_c = 0.053 \ \frac{kN}{m}$



$t_{wall} = 150 \ mm$

The seismic weight on average of the walls per area:

$$W_{tot} = \frac{\frac{10}{3}\left(W_{CSEB} + 3\,W_c + 3\,Ws\right)}{1 \ m} = 3.032 \ \frac{kN}{m^2}$$

This value adjusted to reach a dead weight in Load considerations close to the value from SAP

G

Verification of application of earthquake loads

Hand caluculations for load considerations
Single storey - 2 rooms + kitchen

VERIFICATION
Divide the load over the lintel RCC bands

$V_{bx} = 34.249\ \mathrm{kN}$ Base shear in x - and y -direction equally big

$W_{tot} = 228.328\ \mathrm{kN}$

$h_{roof} = 2.545\ \mathrm{m}$ $D_x = 6.15\ \mathrm{m}$

Equivalent line load over roof band:

$$Q_l = V_{bx} \cdot W_{tot} \cdot \frac{h_{roof}^{\ 2}}{W_{tot} \cdot h_{roof}^{\ 2}} = 34.249\ \mathrm{kN}$$

Load distributed over total length of lintel band in x -direction :

$$\frac{Q_l}{(1.65\ \mathrm{m} + 1.9\ (\mathrm{m}))} = 9.648\ \frac{\mathrm{kN}}{\mathrm{m}}$$

Load distributed over total length of lintel band in y -direction

$$\frac{Q_l}{D_x} = 5.569\ \frac{\mathrm{kN}}{\mathrm{m}}$$

Hand caluculations for load considerations
Single storey - 2 rooms + kitchen

VERIFICATION - Point loads applied on shear walls

Center of mass:

$x_{cm} = 2.84\ m$ From SAP
$y_{cm} = 2.66\ m$

Centre of ridigity:

Shear piers in x -direction

$$L_x = \begin{vmatrix} 1.35 \\ 1.35 \\ 2.0 \\ 1.35 \\ 1.35 \\ 1.35 \\ 2.25 \\ 1.05 \\ 0.3 \end{vmatrix}\ m$$

Shear piers in y -direction

$$L_y = \begin{vmatrix} 5.1 \\ 3.6 \\ 0.6 \\ 2.85 \\ 0.3 \end{vmatrix}\ m$$

Moment of inertia of shear piers:

$$I_x = t_{brick} \cdot \frac{L_y^{\ 3}}{12} = \begin{vmatrix} 1.658 \\ 0.583 \\ 0.003 \\ 0.289 \\ 3.375 \end{vmatrix} 10^{-4}\ m^4$$

$$I_y = t_{brick} \cdot \frac{L_x^{\ 3}}{12} = \begin{vmatrix} 0.031 \\ 0.031 \\ 0.1 \\ 0.031 \\ 0.031 \\ 0.031 \\ 0.142 \\ 0.014 \\ 3.375 \end{vmatrix} 10^{-4}\ m^4$$

Position of shear piers in x -direction

$$x = \begin{vmatrix} 0 \\ 3.45 \\ 3.45 \\ 6.3 \\ 6.225 \end{vmatrix}\ m$$

Position of shear piers in y -direction

$$y = \begin{vmatrix} 0 \\ 0 \\ 2.265 \\ 2.265 \\ 2.265 \\ 4.95 \\ 4.95 \\ 4.95 \\ 0.075 \end{vmatrix}\ m$$

Hand caluculations for load considerations
Single storey - 2 rooms + kitchen

Position of center of ridigity:

$$x_{cr} = \frac{\sum_{i=0}^{4} \left(I_x(i) \cdot x(i) \right)}{\sum_{i=0}^{4} I_x(i)} = 1.518 \ \text{m} \qquad y_{cr} \ \frac{\sum_{i=0}^{8} \left(I_y(i) \cdot y(i) \right)}{\sum_{i=0}^{8} I_y(i)} = 3.15 \ \text{m}$$

<u>Design of eccentricity</u> as per 7.9.2

$$e_{cx} = \text{abs}\left(x_{cm} - x_{cr} \right) = 1.322 \ \text{m}$$

$$e_{cy} = \text{abs}\left(y_{cm} - y_{cr} \right) = 0.49 \ \text{m}$$

b_i floor plan dimension of floor i, perpendicular to the direction of force

$b_x = 6.45$ m $\qquad$ $b_y = 5.1$ m

$$e_{dx} = \begin{array}{|l} \text{if } e_{cx} < 0.1\, b_x \\ \quad 0 \ \text{m} \\ \text{else if } \ 0.1\, b_x \le e_{cx} < 0.3\, b_x \\ \quad e_{cx} + 0.1\, b_x \\ \text{else} \\ \quad \text{"Another analysis is needed"} \end{array} = 1.967 \ \text{m}$$

$$e_{dy} = \begin{array}{|l} \text{if } e_{cy} < 0.1\, b_y \\ \quad 0 \ \text{m} \\ \text{else if } \ 0.1\, b_y \le e_{cy} < 0.3\, b_y \\ \quad e_{cy} + 0.1\, b_y \\ \text{else} \\ \quad \text{"Another analysis is needed"} \end{array} = 0 \ \text{m}$$

No eccentricity in y - direction

Force dirstibution in y:

$A_{y1} = 13.42 \ \text{m}^2$ $\qquad$ $A_{y2} = 11.38 \ \text{m}^2$ $\qquad$ $A_{y3} = 10.34 \ \text{m}^2$

The ratio relations between the areas are:

$$A_{y3n} = \frac{A_{y3}}{A_{y3}} = 1 \qquad A_{y2n} = \frac{A_{y2}}{A_{y3}} = 1.101 \qquad A_{y1n} = \frac{A_{y1}}{A_{y3}} = 1.298$$

Hand caluculations for load considerations
Single storey - 2 rooms + kitchen

This gives the force distribution of:

$$F_{y1} = \frac{V_{by}}{A_{y1n} + A_{y2n} + A_{y3n}} \cdot A_{y1n} = 13.08 \;\; kN$$ Force applied at height of lintel level

$$F_{y2} = \frac{V_{by}}{A_{y1n} + A_{y2n} + A_{y3n}} \cdot A_{y2n} = 11.092 \;\; kN$$ Force applied at height of lintel level

$$F_{y3} = \frac{V_{by}}{A_{y1n} + A_{y2n} + A_{y3n}} \cdot A_{y3n} = 10.078 \;\; kN$$ Force applied at height of lintel level

$F_{y1} + F_{y2} + F_{y3} = 34.25 \;\; kN$ $\quad M = V_{bx} \cdot e_{dx}$

Force dirstibution in x

$A_{x1} = 14.7 \;\; m^2$ $\quad A_{x2} = 19.99 \;\; m^2$ $\quad A_{x3} = 9.3 \;\; m^2$

The ratio relations between the areas are:

$$A_{x3n} = \frac{A_{x3}}{A_{x3}} = 1 \qquad A_{x2n} = \frac{A_{x2}}{A_{x3}} = 2.149 \qquad A_{x1n} = \frac{A_{x1}}{A_{x3}} = 1.581$$

This gives the force distribution of:

$$F_{x1} = \frac{V_{bx}}{A_{x1n} + A_{x2n} + A_{x3n}} \cdot A_{x1n} = 11.445 \;\; kN$$ Force applied at height of lintel level

$$F_{x2} = \frac{V_{bx}}{A_{x1n} + A_{x2n} + A_{x3n}} \cdot A_{x2n} = 15.564 \;\; kN$$ Force applied at height of lintel level

$$F_{x3} = \frac{V_{bx}}{A_{x1n} + A_{x2n} + A_{x3n}} \cdot A_{x3n} = 7.241 \;\; kN$$ Force applied at height of lintel level

$F_{x1} + F_{x2} + F_{x3} = 34.25 \;\; kN$

H

Adjustment of E-modulus following compressive strength

Calculations of the characteristic compressive strength of masonry

There's a lack of methodology or equations to use to calculate the modulous of elasticity given an characteristic compressive strength.
Due to this the characteristic compressive strength of masonry have been calculated using SS-EN 1996-1-1.

Classification of mortar types as per table H-2 in EKS 10:

Tabell H-2 Blandningsproportioner för murbruk

Murbruksklass Bindemedel	Viktdelar	Volymdelar	Murbruksklass Beteckning[1]
Murbruksklass M10 (A)			*Murbruksklass M10 (A)*
Cement	C 100/450	C 1:4	M10-1:0:4C
Kalk, Cement	KC 20/80/400	KC 1:3:15	M10-3:1:15CK
Kalk, Cement	KC 10/90/350	KC 1:4:15	M10-4:1:15CK
Murcement	M 100/350	M 1:3	M10-1:3M
Murbruksklass M2,5 (B)			*Murbruksklass M2,5 (B)*
Kalk, Cement	KC 35/65/550	KC 1:1:8	M2,5-1:1:8CK
Murcement	M 100/600	M 1:5	M2,5-1:5M
Murbruksklass M1 (C)			*Murbruksklass M1 (C)*
Kalk, Cement	KC 50/50/650	KC 2:1:12	M1-1:2:12CK
Murcement	M 100/900	M 1:7	M1-1:7M
Murbruksklass M0,5 (D)			*Murbruksklass M0,5 (D)*
Kalk, Cement	KC 50/50/950	KC 2:1:18	M0,5-1:2:18CK
Hydraulisk kalk	Kh 100/850	Kh 1:5	M0,5-1:5Kh

[1] I beteckningen anges murbruksklass och volymdelar; cement, kalk, sand samt bindemedeltyp.

We choose to comapre the recommended mortar from Build up Nepals approved drawings (1:5 sand cement- mortar) with M2,5 M1:5 -murcement

-> characteristic compressive strength of 2.5 MPa for the mortar

Characteristic compressive strength as per table H-4 in EKS 10:

Tabell H-4 Karakteristiska värden

Murstenar/ murblock	Hållfast-hetsklass	f_k (MPa) Murbruksklass enligt SS-EN 998-2				Tunn-fogs-bruk
		M10	M2,5	M1	M0,5	
Tegelblock	6	–	–	–	–	2,0[c]
	8	–	–	–	–	2,5[c]
	10	–	–	–	–	2,8[c]
	12	–	–	–	–	3,3[c]
Tegelsten	12	5,2	3,6	2,7	1,0	–
	15	5,8	4,2	3,2	1,3	–
	25	7,5	6,0	4,5	1,8	–
	35	8,9	7,5	5,7	2,3	–
	45	10,0	9,0	6,8	2,3	–
	55	11,1	10,3	7,8	2,3	–
	65	12,1	11,6	8,8	2,3	–
Kalksandsten	25	–	6,0	4,5	–	12,3[a]
Betongsten	25	7,5	6,0	–		
Betonghålblock	5	–	2,0	1,5	–	2,6[a]
	10	2,4	2,4	2,4	–	4,6[a]
Massiva betong-block	10	3,8	3,6	2,8	–	5,7[a]
	15	4,7	4,7	3,7	–	8,0[a]
Lättbetongblock	2,0	–	1,2	0,9	–	1,4[a]
	2,5	–	1,4	1,0	–	1,7[a]
	3	–	1,6	1,2	–	2,0[a]
	3,5	–	1,7	1,3	–	2,3[a]
	4,0	–	1,9	1,5	–	2,6[a]
	4,5	–	2,1	1,6	–	2,9[a]
	5	–	2,2	1,7	–	3,1[a]
Lättklinkerblock[b]	2	–	1,8	1,2	0,8	1,4[a]
	3	–	2,4	1,6	1,0	2,0[a]
	5	–	3,4	2,2	1,2	3,1[a]
	10	–	4,3	3,4	1,2	5,7[a]

The CSEB blocks are compared with "lättklinkerblock" with the same characteristic compressive strength as the CSEB of 5MPa

$f_{cu} = 5\ MPa$ Characteristic compressive strength of the masonry units

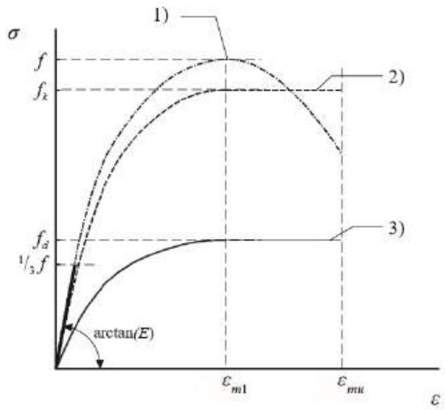
$f_{cm} = 2.5\ MPa$ Characteristic compressive strength of the mortar, group B

This give the combined characteristic compressive stength of the masonry as tabulated:

$f_k = 3.4\ MPa$ Characteristic compressive strength of the masonry

To the be able to calculate the E -modulous of masonry the recommended practise by SS-EN 1996 -1-1, 3.7.1 was decided to be used with a slight adoptation to take into account the material properties of the CSEB

E-modulous for masonry as per SS -EN 1996 -1-1, 3.7.1



Figur 2.2. Spännings-töjningssamband för tryckt murverk. Källa: SS-EN 1996-1-1, 3.7.1. Förklaringar: 1) typkurva, 2) idealiserad kurva (parabolisk-rektangulär), 3) dimensioneringskurva.

The corelation between E and f_k acording to SS -EN 1996 -1-1

$E_{masonry} = K_E \cdot f_k$

fk = masonry characteristic compressive strength
Ke = contant for different masonry unit groups as per tabel 2.5 in Utformning av murverkskonstruktioner enlight Eurokod 6

$E_{CSEB} = 700$ MPa From Shrestha (2012) and Maïni (2005)

We calulated a specific K_e for CSEB accoring to:

$$K_e = \frac{E_{CSEB}}{f_{cu}} = 140$$

E-modulous of masonry:

$E = f_k \cdot K_e = 476$ MPa M5 masonry units & mortar group B

Changed E -modulous with changed compressive strength

As on of the erros where to investigate how a changed compressive strength of the masonry units change the earthquake resistance of the building we need to calculate a new E -modulous of the masonry walls

Characteristic compressive strengths as per tabel 2.2 EKS 10

$f_{k2} = 1.8$ MPa M2 masonry units & mortar group B

$f_{k10} = 4.3$ MPa M10 masonry units & mortar group B

$E = f_{k2} \cdot K_e = 252$ MPa

$E = f_{k10} \cdot K_e = 602$ MPa

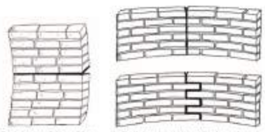
The characteristic bending strength of the masonry walls have been decided to define in a similar way as the compressive strength as per SS -EN 1996.1.1

Table H -6 EKS -10 characteristic bending strength for masonry

Tabell H-6 *Karakteristisk böjhållfasthet*

Murstenar/ Murblock	Hållfast-hetsklass	f_{xk1} (MPa) M1,0-M2,4	f_{xk1} (MPa) M2,5-M10	f_{xk1} Tunn-fogs-bruk	f_{xk2} (MPa) M1,0-M2,4	f_{xk2} (MPa) M2,5-M10	f_{xk2} Tunn-fogs-bruk
Tegelblock	–	–	–	0,29	–	–	0,12
Håltegel	15–65	0,12	0,3	–	0,90	1,1	–
Massivtegel	15–65	0,12	0,25	–	0,90	1,1	–
Kalksandsten	25	0,05	0,10	0,20	0,70	0,90	0,30
Betongsten	25	0,05	0,20	0,20	0,70	0,90	0,30
Betonghålblock	5–10	0,05	0,20	0,20	0,30	0,40	0,30
Massiva betong-block	10–15	0,05	0,20	0,20	0,30	0,40	0,30
Lättbetongblock	2,0	0,08	0,10	0,15	0,08	0,10	0,30
	2,5	0,08	0,10	0,15	0,15	0,20	0,30
	3	0,15	0,15	0,20	0,20	0,25	0,30
	3,5	0,15	0,15	0,20	0,20	0,25	0,30
	4,0	0,15	0,15	0,20	0,20	0,25	0,30
	4,5	0,15	0,15	0,20	0,20	0,25	0,30
	5	0,15	0,15	0,20	0,20	0,25	0,30
Lättklinkerblock[a]	2	0,12	0,15	0,20	0,12	0,15	0,30
	3	0,12	0,15	0,20	0,25	0,30	0,30
	5	0,12	0,15	0,20	0,25	0,30	0,30
	10	0,12	0,15	0,20	0,25	0,30	0,30

[a] För murverk av torrstaplade lättklinkerblock med nätarmerad puts används ett av tillverkaren deklarerat värde på $f_{xk1} = f_{xk2}$, dock högst 0,15 MPa. *(BFS 2015:6).*



a) Brottplan parallellt med liggfogar; f_{xk1} b) Brottplan vinkelrätt mot liggfogar; f_{xk2}

f_xk1 compare to S22 in SAP

f_xk2 compare to S11 in SAP

The tabulated characteristic bending strength for the studied masonry wall can be found in table H -6

$f_{xk1} = 0.15$ MPa $f_{xk2} = 0.3$ MPa For 5 MPa bricks

The tabulated characteristic bending strength for the studied masonry wall can be found in table

$f_{xk1\_2} = 0.15\ \mathrm{MPa}$ $\quad f_{xk2\_2} = 0.15\ \mathrm{MPa}$ $\quad$ For 2 MPa bricks

$f_{xk1\_10} = 0.15\ \mathrm{MPa}$ $\quad f_{xk2\_10} = 0.3\ \mathrm{MPa}$ $\quad$ For 10 MPa bricks

Design values:

The design values were picked from Table H -1 in EKS 10

Tabell H-1 Partialkoefficienter γ_M i brottgränstillstånd

	Utförandeklass[a] (medelvärde)	
Murverk utfört med:	I	II
Stenar/block kategori I, specialmurbruk[a]	1,9	2,1
Stenar/block kategori I, receptmurbruk[b]	2,1	2,5
Stenar/block kategori II, valfritt murbruk[a, b, d]	2,6	3,0
	Utförandeklass[e] (karakteristiskt värde)	
Murverk utfört med:	I	II
Stenar/block kategori I, specialmurbruk[a]	1,8	2,0
Stenar/block kategori I, receptmurbruk[b]	2,0	2,3
Stenar/block kategori II, valfritt murbruk[a, b, d]	2,3	2,7
	Utförandeklass[e]	
	I	II
Armeringsförankring	2,0	2,5
Armeringshållfasthet	1,3	1,3
Murkramlors förankring[c]	2,5	2,7
Murkramlors hållfasthet	1,5	1,7

$\gamma_M = 2.3$

For bricks with characteristic compressive stregth of 5 MPa

$$f_{cd} = \frac{f_k}{\gamma_M} = 1.478\ \frac{\mathrm{N}}{\mathrm{mm}^2} \qquad f_{xd1} = \frac{f_{xk1}}{\gamma_M} = 0.065\ \frac{\mathrm{N}}{\mathrm{mm}^2} \qquad f_{xd2} = \frac{f_{xk2}}{\gamma_M} = 0.13\ \frac{\mathrm{N}}{\mathrm{mm}^2}$$

For bricks with characteristic compressive stregth of 2 MPa

$$f_{cd\_2} = \frac{f_{k2}}{\gamma_M} = 0.783\ \frac{\mathrm{N}}{\mathrm{mm}^2} \qquad f_{xd1\_2} = \frac{f_{xk1\_2}}{\gamma_M} = 0.065\ \frac{\mathrm{N}}{\mathrm{mm}^2} \qquad f_{xd2\_} = \frac{f_{xk2\_2}}{\gamma_M} = 0.065\ \frac{\mathrm{N}}{\mathrm{mm}^2}$$

For bricks with characteristic compressive stregth of 10 MPa

$$f_{cd\_10} = \frac{f_{k10}}{\gamma_M} = 1.87\ \frac{\mathrm{N}}{\mathrm{mm}^2} \qquad f_{xd1\_10} = \frac{f_{xk1\_10}}{\gamma_M} = 0.065\ \frac{\mathrm{N}}{\mathrm{mm}^2} \qquad f_{xd2\_10} = \frac{f_{xk2\_10}}{\gamma_M} = 0.13\ \frac{\mathrm{N}}{\mathrm{mm}^2}$$

I

Load calculations

Hand calculations for load considerations

Geometrical inputs:

$l = 6.15\ m$ Length of building $t_{brick} = 0.15\ m$
$h = 3.472\ m$ Heigth of building
$w = 4.95\ m$ Width of building
$\alpha = 17\ °$ Angle of roof slope
$alpha = 17$ Angle of roos for imposed loads
$D_x = 6.15\ m$ Dimensions of the building along x -axis
$D_y = 4.95\ m$ Dimensions of the building along y -axis

Material properties of CSEB:

$f_{ck\_CSEB} = 5\ \frac{N}{mm^2}$ $E_{CSEB} = 700\ MPa$

$\rho_{CSEB} = 19\ \frac{kN}{m^3}$ $v_{CSEB} = 0.15$

$f_{y\_CSEB} = 500\ \frac{N}{mm^2}$ $W_{CSEBm} = 19.0\ \frac{kN}{m^3}$

Material properties of structural steel (roof):

$\rho_{steel} = 78.5\ \frac{kN}{m^3}$ $E_{steel} = 200000\ MPa$

IS 800 : 2007
Fe 250

$f_{y\_strc\_steel} = 250\ \frac{N}{mm^2}$ $v_{steel} = 0.3$

Material poperties of reinforcing steel:

$f_{y\_steel} = 500\ \frac{N}{mm^2}$ IS 1786 : 2008
Fe 500

Material poperties of concret

$f_{ck\_concret} = 20\ \frac{N}{mm^2}$ Ordinary concrete M 20
IS 456:2000

Material poperties of mortar

$f_{ck\_mortar} = 2.5\ \frac{N}{mm^2}$ EKS 10, Table H -2

Hand caluculations for load considerations
Single storey - 2 rooms + kitchen

<u>LOADS:</u>

- Dead loads: as per IS 875 : 1984 Part I
- Live loads: as per IS 875 : 1984 Part II
- Wind loads: as per NBC 104 : 1994 and IS 875 : 1984 Part III
- Earthquake load: as per IS 1893 : 2002 (and NBC 105:1994)

<u>Dead load:</u>

$G = 232.071\ \mathrm{kN}$ From SAP without false ceiling

<u>Imposed load:</u>

Table 2: Imposed loads on various types of roofs
ii) sloping roof with slope greater than 10 degreees

$$q_r = 0.75\ \frac{\mathrm{kN}}{\mathrm{m}^2} - \left(0.02\ \frac{\mathrm{kN}}{\mathrm{m}^2}\left(alpha - 10\right)\right) = 0.61\ \frac{\mathrm{kN}}{\mathrm{m}^2}$$

Divide the imposed load over the roof rafters

$$q_{r1} = q_r \cdot 31.673\ \mathrm{m}^2 = 19.321\ \mathrm{kN}$$

$$l_r = 8 \cdot 6.15\ \mathrm{m} = 49.2\ \mathrm{m}$$

$$q_{rdist} = \frac{q_{r1}}{l_r} = 0.393\ \frac{\mathrm{kN}}{\mathrm{m}}$$

Hand caluculations for load considerations
Single storey - 2 rooms + kitchen

Wind loads:

$V_b = 50\ \frac{m}{s}$ Basic wind speed

$k_1 = 1$ Probability factor (risk coeff.) from Table 1

$k_2 = 1.05$ Terrain, height and structure size factor from Table 2
Category 1 Class A

$k_3 = 1$ Topography factor

$V_z = V_b \cdot k_1 \cdot k_2 \cdot k_3 = 52.5\ \frac{m}{s}$ Design wind speed

$p_z = 0.6\ \frac{Pa \cdot s^2}{m^2} \cdot V_z^{\,2} = (1.654 \cdot 10^3)\ \frac{N}{m^2}$ Design wind pressure, accor. to 5.4 " the coeff. 0.6 SI units depends on a number of factors..."

$\frac{h}{w} = 0.701$ $\frac{l}{w} = 1.242$ Wind angle tetha = 0

WALLS:
Decide external pressure coefficients (C_pe) from ratios (Table 4 - Rectangular clad buildings):

$\frac{h}{w} \le \frac{1}{2} = 0$

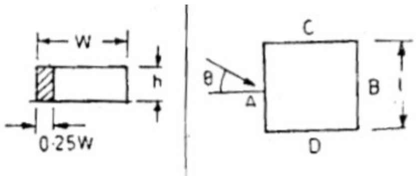
$\frac{1}{2} < \frac{h}{w} \le \frac{3}{2} = 1$

$\frac{3}{2} < \frac{h}{w} \le 6 = 0$

$1 < \frac{l}{w} \le \frac{3}{2} = 1$

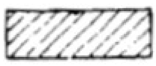
$\frac{3}{2} < \frac{l}{w} < 4 = 0$

These coefficent used from second h/w condition and first l/w condition



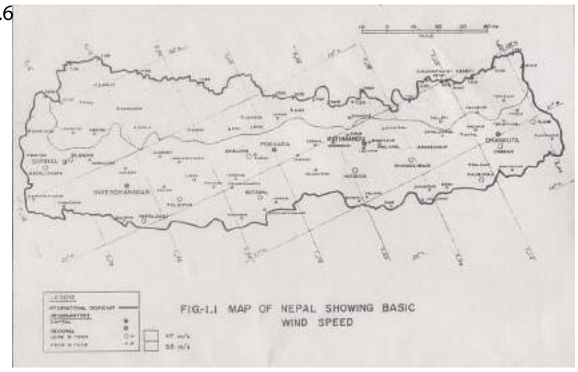
$C_{pe\_A} = 0.7$ $C_{pe\_C} = -0.6$

$C_{pe\_B} = -0.25$ $C_{pe\_D} = -0.6$



W

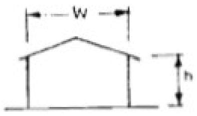
$C_{pe\_W} = -1.1$



Hand caluculations for load considerations
Single storey - 2 rooms + kitchen

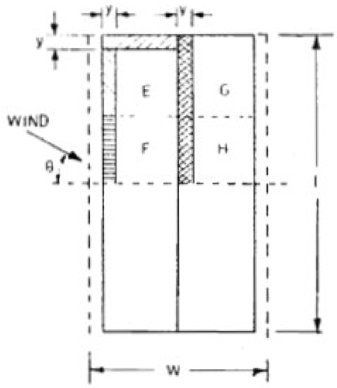
ROOF:
Ratios for C_pe on the roof (Table 5)

$\frac{h}{w} \le \frac{1}{2} = 0$



$\frac{1}{2} < \frac{h}{w} \le \frac{3}{2} = 1$

$\frac{3}{2} < \frac{h}{w} \le 6 = 0$



KEY PLAN

$y = h$ or $0{\cdot}15\,w$, whichever is the lesser.

$x = 17$ -> Linear interpolation between 10 -20 degrees

$x_0 = 10$
$x_1 = 20$

EF Tetha = 0 degrees

$y_0 = -1.1$
$y_1 = -0.7$

$$C_{pe\_EF0} = y_0 + (x - x_0) \cdot \frac{(y_1 - y_0)}{(x_1 - x_0)} = -0.82$$

GH Tetha = 0 degrees
$y_0 = -0.6$
$y_1 = -0.5$

$$C_{pe\_GH0} = y_0 + (x - x_0) \cdot \frac{(y_1 - y_0)}{(x_1 - x_0)} = -0.53$$

EG Tetha = 90 degrees
$y_0 = -0.8$
$y_1 = -0.8$

$$C_{pe\_EG90} = y_0 + (x - x_0) \cdot \frac{(y_1 - y_0)}{(x_1 - x_0)} = -0.8$$

FH Tetha = 90 degrees
$y_0 = -0.6$
$y_1 = -0.6$

$$C_{pe\_FH90} = y_0 + (x - x_0) \cdot \frac{(y_1 - y_0)}{(x_1 - x_0)} = -0.6$$

Hand caluculations for load considerations
Single storey - 2 rooms + kitchen

walls:

$$C_{pe\_max\_wx} = \max\left(C_{pe\_A}, C_{pe\_B}, C_{pe\_W}\right) = 0.7$$

$$C_{pe\_min\_wx} = \min\left(C_{pe\_A}, C_{pe\_B}, C_{pe\_W}\right) = -1.1$$

$$C_{pe\_max\_wy} = \max\left(C_{pe\_C}, C_{pe\_D}, C_{pe\_W}\right) = -0.6$$

$$C_{pe\_min\_wy} = \min\left(C_{pe\_C}, C_{pe\_D}, C_{pe\_W}\right) = -1.1$$

Roof:

$$C_{pe\_max\_rx} = \max\left(C_{pe\_EG90}, C_{pe\_FH90}\right) = -0.6$$

$$C_{pe\_min\_rx} = \min\left(C_{pe\_EG90}, C_{pe\_FH90}\right) = -0.8$$

$$C_{pe\_max\_ry} = \max\left(C_{pe\_EF0}, C_{pe\_GH0}\right) = -0.53$$

$$C_{pe\_min\_ry} = \min\left(C_{pe\_EF0}, C_{pe\_GH0}\right) = -0.82$$

Internal pressure coefficents

$C_{pi\_neg} = -0.2$ $\qquad$ $C_{pi\_pos} = 0.2$

Design load in x direction for the walls:

$$F1x = \left(C_{pe\_min\_wx} - C_{pi\_neg}\right) \cdot p_z = -1.488 \ \frac{kN}{m^2}$$

$$F2x = \left(C_{pe\_min\_wx} - C_{pi\_pos}\right) \cdot p_z = -2.15 \ \frac{kN}{m^2}$$

$$F3x = \left(C_{pe\_max\_wx} - C_{pi\_neg}\right) \cdot p_z = 1.488 \ \frac{kN}{m^2}$$

$$F4x = \left(C_{pe\_max\_wx} - C_{pi\_pos}\right) \cdot p_z = 0.827 \ \frac{kN}{m^2}$$

$$F_{wx} = F2x = -2.15 \ \frac{kN}{m^2}$$

Design load in y direction for the walls:

$$F1y = \left(C_{pe\_min\_wy} - C_{pi\_neg}\right) \cdot p_z = -1.488 \ \frac{kN}{m^2}$$

$$F2y = \left(C_{pe\_min\_wy} - C_{pi\_pos}\right) \cdot p_z = -2.15 \ \frac{kN}{m^2}$$

Hand caluculations for load considerations
Single storey - 2 rooms + kitchen

$$F3y = (C_{pe\_max\_wy} - C_{pi\_neg}) \cdot p_z = -0.662\ \frac{kN}{m^2}$$

$$F4y = (C_{pe\_max\_wy} - C_{pi\_pos}) \cdot p_z = -1.323\ \frac{kN}{m^2}$$

$$F_{wy} = F2y = -2.15\ \frac{kN}{m^2}$$

Design load in x direction for the roof:

$$F1x = (C_{pe\_min\_rx} - C_{pi\_neg}) \cdot p_z = -0.992\ \frac{kN}{m^2} \qquad F3x = (C_{pe\_max\_rx} - C_{pi\_neg}) \cdot p_z = -0.662\ \frac{kN}{m^2}$$

$$F2x = (C_{pe\_min\_rx} - C_{pi\_pos}) \cdot p_z = -1.654\ \frac{kN}{m^2} \qquad F4x = (C_{pe\_max\_rx} - C_{pi\_pos}) \cdot p_z = -1.323\ \frac{kN}{m^2}$$

$$F_{rx} = F2x = -1.654\ \frac{kN}{m^2}$$

Design load in y direction for the roof:

$$F1y = (C_{pe\_min\_ry} - C_{pi\_neg}) \cdot p_z = -1.025\ \frac{kN}{m^2}$$

$$F2y = (C_{pe\_min\_ry} - C_{pi\_pos}) \cdot p_z = -1.687\ \frac{kN}{m^2}$$

$$F3y = (C_{pe\_max\_ry} - C_{pi\_neg}) \cdot p_z = -0.546\ \frac{kN}{m^2}$$

$$F4y = (C_{pe\_max\_ry} - C_{pi\_pos}) \cdot p_z = -1.207\ \frac{kN}{m^2}$$

$$F_{ry} = F2y = -1.687\ \frac{kN}{m^2}$$

$$WL_{wall} = \begin{vmatrix} F_{wx} \\ F_{wy} \end{vmatrix} = \begin{vmatrix} -2.15 \\ -2.15 \end{vmatrix}\ \frac{kN}{m^2} \qquad WL_{roof} = \begin{vmatrix} F_{rx} \\ F_{ry} \end{vmatrix} = \begin{vmatrix} -1.654 \\ -1.687 \end{vmatrix}\ \frac{kN}{m^2}$$

Divide WL on roof over the rafters:

$$q_{rwx} = F_{rx} \cdot 31.673\ m^2 = -52.379\ kN \qquad q_{rwy} = F_{ry} \cdot 31.673\ m^2 = -53.427\ kN$$

$$l_r = 8 \cdot 6.15\ m = 49.2\ m \qquad l_r = 8 \cdot 6.15\ m = 49.2\ m$$

$$q_{rdistx} = \frac{q_{rwx}}{l_r} = -1.065\ \frac{kN}{m} \qquad q_{rdisty} = \frac{q_{rwy}}{l_r} = -1.086\ \frac{kN}{m}$$

Hand caluculations for load considerations
Single storey - 2 rooms + kitchen

Earthquake load

Classification of building:
- General provisions and buildings

Zone factor Z Table 2 (Clause 6.4.2)

$Z = 0.36$ Very severe

Importance factor I Table 6

$I = 1.0$

Response reduction factor R as per Table 7 and clause 6.4.2)

$R = 3$ Category v) c) Reinf h RC + vertical $\frac{I}{R} < 1 = 1$ Ok

Time period of the building as per 7.6.2

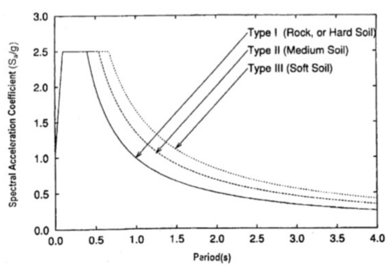
$T_x = 0.09 \frac{s}{m^{0.5}} \frac{h}{D_x^{0.5}} = 0.126\ s$ Along x

$T_y = 0.09 \frac{s}{m^{0.5}} \frac{h}{D_y^{0.5}} = 0.14\ s$ Along y

Soil profile:
Assumed to be sub soil category III (soft soil site)

Average response acceleration coefficient S a/g
(from soil type and time period T)

Along x	Along y
$0\ s \le T_x \le 0.1\ s = 0$	$0\ s \le T_y \le 0.1\ s = 0$
$0.1\ s \le T_x \le 0.67\ s = 1$	$0.1\ s \le T_y \le 0.67\ s = 1$
$0.67\ s \le T_x \le 4\ s = 0$	$0.67\ s \le T_y \le 4\ s = 0$



$S_{ax} = 2.5$ $g_x = 1$ $S_{ay} = 2.5$ $g_y = 1$

$\frac{S_{ax}}{g_x} = 2.5$ $\frac{S_{ay}}{g_y} = 2.5$

Hand caluculations for load considerations
Single storey - 2 rooms + kitchen

The design horizontal seismic coefficient A_h

$$A_{hx} = \frac{Z \cdot I \cdot S_{ax}}{2 \cdot R \cdot g_x} = 0.15$$ if T<= 0.1 s -> A_h=Z/2

$$A_{hy} = \frac{Z \cdot I \cdot S_{ay}}{2 \cdot R \cdot g_y} = 0.15$$

Seismic weight of the building W as per 7.4.2

" *The seismic weight of each floor is its full dead load plus appropriate amount of imposed load, as specified in 7.3.1 and 7.3.2. While computing the seismic weight of each floor, the weight of columns and walls in any storey shall be equally distributed to the floors above and below the storey.* "

Treat each of roof, lintel and sill RCC bands as one storey and apply earthquake load in proportion to tributary area and weight of each "floor":

$W_{ave} = 4.1 \ \frac{kN}{m^2}$ Adjusted estimated value to get closer to total seaismic weight from SAP

$h_{sill} = 0.780 \ m$ $h_{lintel} = 2.03 \ m$ $h_{roof} = 2.545 \ m$

Rough estimation of the roof weight :

$$W_{rafter} = (76.97 \cdot 6.6 \cdot 10^{-4} \cdot (1.65 + 3.5)) \cdot 7 \ kN = 1.831 \cdot 10^{3} \ N$$

$$W_{purlin} = (76.97 \cdot 4.45 \cdot 10^{-4} \cdot (6.15)) \cdot 6 \ kN = 1.264 \cdot 10^{3} \ N$$

$$W_{ridge} = 76.97 \cdot 1.1 \cdot 10^{-3} \cdot 6.15 \ kN = 520.702 \ N$$

$$W_{r.p.r} = W_{rafter} + W_{purlin} + W_{ridge} = 3.616 \cdot 10^{3} \ N$$

$$A_{roof} = 1.65 \ m \cdot 6.15 \ m + 3.5 \ m \cdot 6.15 \ m = 31.673 \ m^2$$

$$W_{rafters} = \frac{W_{r.p.r}}{A_{roof}} = 0.114 \ \frac{kN}{m^2}$$

Weight of CGI and roof rafters:

$$W_{CGI} = 0.043164 \ \frac{kN}{m^2}$$

$$W_{CGI.rafters} = W_{CGI} + W_{rafters} = 0.157 \ \frac{kN}{m^2}$$

$$W_{CGI.rafters} \cdot A_{roof} = 4.983 \ kN$$

Hand caluculations for load considerations
Single storey - 2 rooms + kitchen

Roof RCC band:

$$A_{roof} = \begin{vmatrix} 0.484 \\ 0.461 \\ 0.461 \\ 0.484 \\ 0.839 \\ 0.839 \\ 0.839 \\ 0.815 \\ 0.815 \\ 0.839 \\ 0.484 \\ 0.461 \\ 0.461 \\ 0.484 \\ 0.535 \end{vmatrix} \ m^2 \qquad W_{roof\_i1} = A_{roof} \cdot W_{ave} = \begin{vmatrix} 1.984 \\ 1.89 \\ 1.89 \\ 1.984 \\ 3.44 \\ 3.44 \\ 3.44 \\ 3.342 \\ 3.342 \\ 3.44 \\ 1.984 \\ 1.89 \\ 1.89 \\ 1.984 \\ 2.194 \end{vmatrix} \ kN$$

Areas and weights from rafters added to roof RCC band:

$$A_{only\_roof} = \begin{vmatrix} 1.479 \\ 1.443 \\ 1.449 \\ 1.470 \\ 1.470 \\ 2.250 \\ 1.470 \\ 1.447 \\ 2.250 \\ 1.460 \\ 1.822 \\ 1.823 \\ 1.823 \\ 1.849 \\ 1.714 \end{vmatrix} \ m^2 \qquad W_{only\_roof} = A_{only\_roof} \cdot W_{CGI.rafters} = \begin{vmatrix} 0.233 \\ 0.227 \\ 0.228 \\ 0.231 \\ 0.231 \\ 0.354 \\ 0.231 \\ 0.228 \\ 0.354 \\ 0.23 \\ 0.287 \\ 0.287 \\ 0.287 \\ 0.291 \\ 0.27 \end{vmatrix} \ kN$$

Hand caluculations for load considerations
Single storey - 2 rooms + kitchen

Total seismic weight at roof RCC band

$$W_{roof\_i} = W_{roof\_i1} + W_{only\_roof} = \begin{vmatrix} 2.217 \\ 2.117 \\ 2.118 \\ 2.216 \\ 3.671 \\ 3.794 \\ 3.671 \\ 3.569 \\ 3.695 \\ 3.67 \\ 2.271 \\ 2.177 \\ 2.177 \\ 2.275 \\ 2.463 \end{vmatrix} \text{ kN}$$

Lintel RCC band:

$$A_{lintel} = \begin{vmatrix} 1.088 \\ 1.108 \\ 1.108 \\ 1.088 \\ 1.174 \\ 1.261 \\ 1.390 \\ 1.022 \\ 0.936 \\ 1.390 \\ 1.390 \\ 1.324 \\ 1.022 \\ 1.088 \\ 1.769 \\ 0.758 \\ 0.761 \\ 0.847 \\ 0.579 \\ 1.010 \end{vmatrix} \text{ m}^2 \qquad W_{lintel\_i1} = A_{lintel} \cdot W_{ave} = \begin{vmatrix} 4.461 \\ 4.543 \\ 4.543 \\ 4.461 \\ 4.813 \\ 5.17 \\ 5.699 \\ 4.19 \\ 3.838 \\ 5.699 \\ 5.699 \\ 5.428 \\ 4.19 \\ 4.461 \\ 7.253 \\ 3.108 \\ 3.12 \\ 3.473 \\ 2.374 \\ 4.141 \end{vmatrix} \text{ kN}$$

Areas and weights from rafters added to lintel RCC band:

$$A_{only\_lintel} = \begin{vmatrix} 0 \\ 0 \\ 0 \\ 0 \\ 0 \\ 0 \\ 0 \\ 0 \\ \vdots \end{vmatrix} \text{ m}^2 \qquad W_{only\_lintel} = A_{only\_lintel} \cdot W_{CGI.rafters} = \begin{array}{c} 0 \\ 1 \\ 2 \\ 3 \\ 4 \\ 5 \\ \vdots \\ 19 \end{array} \begin{vmatrix} 0 \\ 0 \\ 0 \\ 0 \\ 0 \\ 0 \\ \vdots \\ \end{vmatrix} \text{ kN}$$

Hand caluculations for load considerations
Single storey - 2 rooms + kitchen

Total seismic weight at lintel RCC band:

$$W_{lintel\_i} = W_{lintel\_i1} + W_{only\_lintel} = \begin{vmatrix} 4.461 \\ 4.543 \\ 4.543 \\ 4.461 \\ 4.813 \\ 5.17 \\ 5.699 \\ 4.19 \\ 3.838 \\ 5.699 \\ 5.699 \\ 5.428 \\ 4.19 \\ 4.461 \\ 7.458 \\ 3.272 \\ 3.283 \\ 3.836 \\ 2.594 \\ 4.349 \end{vmatrix} \, kN$$

Total seismic weight at sill RCC band:

$$A_{sill} = \begin{vmatrix} 1.336 \\ 1.344 \\ 1.337 \\ 1.336 \\ 1.422 \\ 1.404 \\ 1.638 \\ 0.999 \\ 0.858 \\ 1.638 \\ 1.638 \\ 1.560 \\ 1.014 \\ 1.092 \\ 1.950 \\ 0.546 \\ 1.336 \\ 1.422 \\ 0.624 \\ 1.456 \end{vmatrix} \, m^2 \qquad W_{sill\_i} = A_{sill} \cdot W_{ave} = \begin{vmatrix} 5.478 \\ 5.51 \\ 5.482 \\ 5.478 \\ 5.83 \\ 5.756 \\ 6.716 \\ 4.096 \\ 3.518 \\ 6.716 \\ 6.716 \\ 6.396 \\ 4.157 \\ 4.477 \\ 7.995 \\ 2.239 \\ 5.478 \\ 5.83 \\ 2.558 \\ 5.97 \end{vmatrix} \, kN$$

Hand caluculations for load considerations
Single storey - 2 rooms + kitchen

Tot weight from SAP for comparison
$G = 232.071$ kN   From SAP

Total weight of each "floor":

$$W_{sill} = \left(\sum_{i=1}^{19} W_{sill\_i}(i)\right) = 100.917 \text{ kN}$$

$$W_{lintel} = \left(\sum_{i=1}^{19} W_{lintel\_i}(i)\right) = 87.526 \text{ kN}$$

$$W_{roof} = \left(\sum_{i=1}^{14} W_{roof\_i}(i)\right) = 39.885 \text{ kN}$$

$$W_{tot} = W_{sill} + W_{lintel} + W_{roof} = 228.328 \text{ kN}$$

Total seismic weight of building
(compare to SAP value)

Seismic base shear V_base as per 7.5.3

$V_{bx} = A_{hx} \cdot W_{tot} = 34.249$ kN $D_x = 6.15$ m

$V_{by} = A_{hy} \cdot W_{tot} = 34.249$ kN $D_y = 4.95$ m

Distribution of design force as per 7.7.1

$$Q_i = V_B \frac{W_i h_i^2}{\sum_{j=1}^{n} W_j h_j^2}$$

Q_i = Design lateral force at floor i
W_i = Seismic weight of floor i
h_i = Height of floor i measured from base
n = numer of storeys

Hand caluculations for load considerations
Single storey - 2 rooms + kitchen

Proportion of base shear applied on sill band

$$Q_{sill\_x} = \frac{V_{bx} \cdot W_{sill\_i} \cdot h_{sill}^{\;2}}{W_{sill} \cdot h_{sill}^{\;2} + W_{lintel} \cdot h_{lintel}^{\;2} + W_{roof} \cdot h_{roof}^{\;2}} = \begin{vmatrix} 0.168 \\ 0.169 \\ 0.168 \\ 0.168 \\ 0.179 \\ 0.176 \\ 0.206 \\ 0.125 \\ 0.108 \\ 0.206 \\ 0.206 \\ 0.196 \\ 0.127 \\ 0.137 \\ 0.245 \\ 0.069 \\ 0.168 \\ 0.179 \\ 0.078 \\ 0.183 \end{vmatrix} \; kN$$

$$Q_{sill\_y} = \frac{V_{by} \cdot W_{sill\_i} \cdot h_{sill}^{\;2}}{W_{sill} \cdot h_{sill}^{\;2} + W_{lintel} \cdot h_{lintel}^{\;2} + W_{roof} \cdot h_{roof}^{\;2}} = \begin{vmatrix} 0.168 \\ 0.169 \\ 0.168 \\ 0.168 \\ 0.179 \\ 0.176 \\ 0.206 \\ 0.125 \\ 0.108 \\ 0.206 \\ 0.206 \\ 0.196 \\ 0.127 \\ 0.137 \\ 0.245 \\ 0.069 \\ 0.168 \\ 0.179 \\ 0.078 \\ 0.183 \end{vmatrix} \; kN$$

Hand caluculations for load considerations
Single storey - 2 rooms + kitchen

Proportion of base shear applied on lintel band

$$Q_{lintel\_x} = \frac{V_{bx} \cdot W_{lintel\_i} \cdot h_{lintel}^{\ 2}}{W_{sill} \cdot h_{sill}^{\ 2} + W_{lintel} \cdot h_{lintel}^{\ 2} + W_{roof} \cdot h_{roof}^{\ 2}} = \left| \begin{matrix} 0.925 \\ 0.942 \\ 0.942 \\ 0.925 \\ 0.998 \\ 1.072 \\ 1.182 \\ 0.869 \\ 0.796 \\ 1.182 \\ 1.182 \\ 1.126 \\ 0.869 \\ 0.925 \\ 1.547 \\ 0.679 \\ 0.681 \\ 0.796 \\ 0.538 \\ 0.902 \end{matrix} \right| \ \mathrm{kN}$$

$$Q_{lintel\_y} = \frac{V_{by} \ W_{lintel\_i} \cdot h_{lintel}^{\ 2}}{W_{sill} \cdot h_{sill}^{\ 2} + W_{lintel} \cdot h_{lintel}^{\ 2} + W_{roof} \cdot h_{roof}^{\ 2}} = \left| \begin{matrix} 0.925 \\ 0.942 \\ 0.942 \\ 0.925 \\ 0.998 \\ 1.072 \\ 1.182 \\ 0.869 \\ 0.796 \\ 1.182 \\ 1.182 \\ 1.126 \\ 0.869 \\ 0.925 \\ 1.547 \\ 0.679 \\ 0.681 \\ 0.796 \\ 0.538 \\ 0.902 \end{matrix} \right| \ \mathrm{kN}$$

Hand caluculations for load considerations
Single storey - 2 rooms + kitchen

Proportion of base shear applied on roof band

$$Q_{roof\_x} = \frac{V_{bx} \cdot W_{roof\_i} \cdot h_{roof}^{2}}{W_{sill} \cdot h_{sill}^{2} + W_{lintel} \cdot h_{lintel}^{2} + W_{roof} \cdot h_{roof}^{2}} = \begin{vmatrix} 0.723 \\ 0.69 \\ 0.691 \\ 0.722 \\ 1.197 \\ 1.237 \\ 1.197 \\ 1.164 \\ 1.205 \\ 1.196 \\ 0.74 \\ 0.71 \\ 0.71 \\ 0.742 \\ 0.803 \end{vmatrix} \; \text{kN}$$

$$Q_{roof\_y} = \frac{V_{by} \cdot W_{roof\_i} \cdot h_{roof}^{2}}{W_{sill} \cdot h_{sill}^{2} + W_{lintel} \cdot h_{lintel}^{2} + W_{roof} \cdot h_{roof}^{2}} = \begin{vmatrix} 0.723 \\ 0.69 \\ 0.691 \\ 0.722 \\ 1.197 \\ 1.237 \\ 1.197 \\ 1.164 \\ 1.205 \\ 1.196 \\ 0.74 \\ 0.71 \\ 0.71 \\ 0.742 \\ 0.803 \end{vmatrix} \; \text{kN}$$

$$Q_{sill} = \left(\sum_{i=1}^{19} Q_{sill\_x}(i)\right) = 3.091 \; \text{kN} \qquad Q_{lintel} = \left(\sum_{i=1}^{19} Q_{lintel\_x}(i)\right) = 18.155 \; \text{kN}$$

$$Q_{roof} = \left(\sum_{i=1}^{14} Q_{roof\_x}(i)\right) = 13.003 \; \text{kN} \qquad Q_{tot} = Q_{sill} + Q_{lintel} + Q_{roof} = 34.249 \; \text{kN}$$

Table I.1: Earthquake Loads

Wall Part	**Roof Part**	**Wall Part**	**Roof Part**
Roof 1	1	Roof 12	12
Roof 2	2	Roof 13	13
Roof 3	3	Roof 14	14
Roof 4	4	Lintel 15	15
Roof 5	5	Lintel 16	16
Roof 6	6	Lintel 17	17
Roof 7	7	Lintel 18	18
Roof 8	8	Lintel 19	19
Roof 9	9	Lintel 20	20
Roof 10	10	Lintel 21	21
Roof 11	11		

Table I.2: Earthquake Loads

Part	**x-dir. [kN]**	**y-dir. [kN]**	**Part**	**x-dir. [kN]**	**y-dir. [kN]**
Roof 1	0.723	0.723	Roof 8	1.164	1.164
Roof 2	0.690	0.690	Roof 9	1.205	1.205
Roof 3	0.691	0.691	Roof 10	1.196	1.196
Roof 4	0.711	0.711	Roof 11	0.740	0.740
Roof 5	1.197	1.197	Roof 12	0.710	0.710
Roof 6	1.237	1.237	Roof 13	0.710	0.710
Roof 7	1.197	1.197	Roof 14	0.742	0.742
Lintel 1	0.925	0.925	Lintel 12	1.126	1.126
Lintel 2	0.942	0.942	Lintel 13	0.869	0.869
Lintel 3	0.942	0.942	Lintel 14	0.925	0.925
Lintel 4	0.925	0.925	Lintel 15	1.547	1.547
Lintel 5	0.998	0.998	Lintel 16	0.679	0.679
Lintel 6	1.072	1.072	Lintel 17	0.681	0.681
Lintel 7	1.182	1.182	Lintel 18	0.796	0.796
Lintel 8	0.869	0.869	Lintel 19	0.538	0.538
Lintel 9	0.796	0.796	Lintel 20	0.902	0.902
Lintel 10	1.182	1.182	Lintel 21	0.803	0.803
Lintel 11	1.182	1.182			
Sill 1	0.168	0.168	Sill 11	0.206	0.206
Sill 2	0.169	0.169	Sill 12	0.196	0.196
Sill 3	0.168	0.168	Sill 13	0.127	0.127
Sill 4	0.168	0.168	Sill 14	0.137	0.137
Sill 5	0.179	0.179	Sill 15	0.245	0.245
Sill 6	0.176	0.176	Sill 16	0.069	0.069
Sill 7	0.206	0.206	Sill 17	0.168	0.168
Sill 8	0.125	0.125	Sill 18	0.179	0.179
Sill 9	0.108	0.108	Sill 19	0.078	0.078
Sill 10	0.206	0.206	Sill 20	0.183	0.183

Hand calculations for load considerations

Geometrical inputs:

$l = 6.15\ m$	Length of building	t_{brick} 0.15 m
$h = (3.472 + 0.5)\ m$	Heigth of building	
$w = 4.95\ m$	Width of building	
$\alpha = 28\ °$	Angle of roof slope	
$alpha = 28$	Angle of roos for imposed loads	
$D_x = 6.15\ m$	Dimensions of the building along x -axis	
$D_y = 4.95\ m$	Dimensions of the building along y -axis	

Material properties of CSEB:

$f_{ck\_CSEB} = 5\ \frac{N}{mm^2}$ $\quad E_{CSEB} = 700\ MPa$

$\rho_{CSEB} = 19\ \frac{kN}{m^3}$ $\quad v_{CSEB} = 0.15$

$f_{y\_CSEB} = 500\ \frac{N}{mm^2}$ $\quad W_{CSEBm} = 19.0\ \frac{kN}{m^3}$

Material properties of structural steel (roof):

$\rho_{steel} = 78.5\ \frac{kN}{m^3}$ $\quad E_{steel} = 200000\ MPa$

IS 800 : 2007
Fe 250

$f_{y\_strc\_steel} = 250\ \frac{N}{mm^2}$ $\quad v_{steel} = 0.3$

Material poperties of reinforcing steel:

$f_{y\_steel} = 500\ \frac{N}{mm^2}$

IS 1786 : 2008
Fe 500

Material poperties of concret

$f_{ck\_concret} = 20\ \frac{N}{mm^2}$

Ordinary concrete M 20
IS 456:2000

Material poperties of mortar

$f_{ck\_mortar} = 2.5\ \frac{N}{mm^2}$

EKS 10, Table H -2

Hand caluculations for load considerations
Single storey - 2 rooms + kitchen

LOADS:
- Dead loads: as per IS 875 : 1984 Part I
- Live loads: as per IS 875 : 1984 Part II
- Wind loads: as per NBC 104 : 1994 and IS 875 : 1984 Part III
- Earthquake load: as per IS 1893 : 2002 (Or NBC 105:1994?)

Dead load:

$G = 232.071 \ kN$ From SAP without false ceiling

Imposed load:

Table 2: Imposed loads on various types of roofs
ii) sloping roof with slope greater than 10 degreees

$$q_r = 0.75 \ \frac{kN}{m^2} - \left(0.02 \ \frac{kN}{m^2} \left(alpha - 10\right)\right) = 0.39 \ \frac{kN}{m^2}$$

Divide the imposed load over the roof rafters

$$q_{r1} = q_r \cdot 31.673 \ m^2 = 12.352 \ kN$$

$$l_r = 8 \cdot 6.15 \ m = 49.2 \ m$$

$$q_{rdist} = \frac{q_{r1}}{l_r} = 0.251 \ \frac{kN}{m}$$

Hand caluculations for load considerations
Single storey - 2 rooms + kitchen

Wind loads:

$V_b = 50\ \frac{m}{s}$ Basic wind speed

$k_1 = 1$ Probability factor (risk coeff.) from Table 1

$k_2 = 1.05$ Terrain, height and structure size factor from Table 2 Category 1 Class A?

$k_3 = 1$ Topography factor

$V_z = V_b \cdot k_1 \cdot k_2 \cdot k_3 = 52.5\ \frac{m}{s}$ Design wind speed

$p_z = 0.6\ \frac{Pa \cdot s^2}{m^2} \cdot V_z^{\ 2} = (1.654 \cdot 10^3)\ \frac{N}{m^2}$ Design wind pressure, accor. to 5.4 " the coeff. 0.6 SI units depends on a number of factors..."

$\frac{h}{w} = 0.802$

$\frac{l}{w} = 1.242$

Wind angle tetha = 0

WALLS:
Decide external pressure coefficients (C_pe) from ratios (Table 4 - Rectangular clad buildings):

$\frac{h}{w} \le \frac{1}{2} = 0$

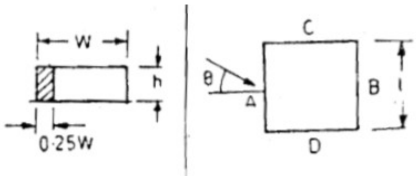
$\frac{1}{2} < \frac{h}{w} \le \frac{3}{2} = 1$

$\frac{3}{2} < \frac{h}{w} \le 6 = 0$

$1 < \frac{l}{w} \le \frac{3}{2} = 1$

$\frac{3}{2} < \frac{l}{w} < 4 = 0$

These coefficent used from second h/w condition and first l/w condition

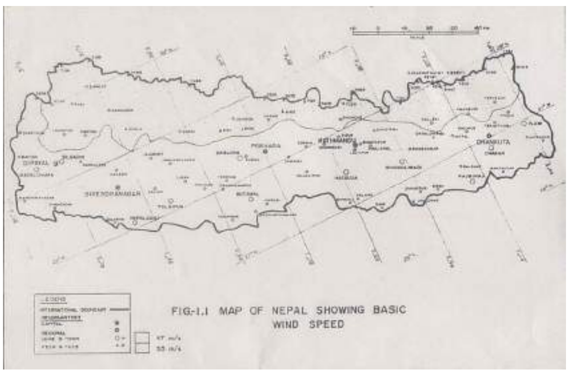


$C_{pe\_A} = 0.7$ $C_{pe\_C} = -0.6$

$C_{pe\_B} = -0.25$ $C_{pe\_D} = -0.6$

W

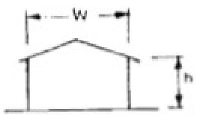
$C_{pe\_W} = -1.1$

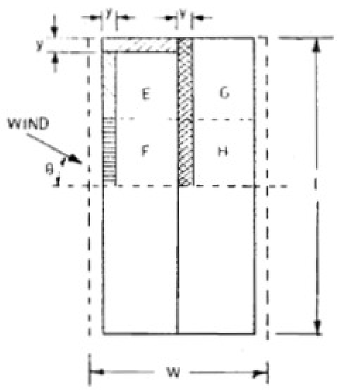


Hand caluculations for load considerations
Single storey - 2 rooms + kitchen

<u>ROOF:</u>
Ratios for C_pe on the roof (Table 5)

$$\frac{h}{w} \leq \frac{1}{2} = 0$$





KEY PLAN

$y = h$ or $0{\cdot}15\,w$, whichever is the lesser.

$$\frac{1}{2} < \frac{h}{w} \leq \frac{3}{2} = 1$$

$$\frac{3}{2} < \frac{h}{w} \leq 6 = 0$$

$x = 28$ -> Linear interpolation between 20 -30 degrees

$x_0 = 20$
$x_1 = 30$

EF Tetha = 0 degrees

$y_0 = -0.7$
$y_1 = -0.2$

$$C_{pe\_EF0} = y_0 + (x - x_0) \cdot \frac{(y_1 - y_0)}{(x_1 - x_0)} = -0.3$$

GH Tetha = 0 degrees
$y_0 = -0.5$
$y_1 = -0.5$

$$C_{pe\_GH0} = y_0 + (x - x_0) \cdot \frac{(y_1 - y_0)}{(x_1 - x_0)} = -0.5$$

EG Tetha = 90 degrees
$y_0 = -0.8$
$y_1 = -0.8$

$$C_{pe\_EG90} = y_0 + (x - x_0) \cdot \frac{(y_1 - y_0)}{(x_1 - x_0)} = -0.8$$

FH Tetha = 90 degrees
$y_0 = -0.6$
$y_1 = -0.8$

$$C_{pe\_FH90} = y_0 + (x - x_0) \cdot \frac{(y_1 - y_0)}{(x_1 - x_0)} = -0.76$$

Hand caluculations for load considerations
Single storey - 2 rooms + kitchen

walls:

$$C_{pe\_max\_wx} = \max(C_{pe\_A}, C_{pe\_B}, C_{pe\_W}) = 0.7$$

$$C_{pe\_min\_wx} = \min(C_{pe\_A}, C_{pe\_B}, C_{pe\_W}) = -1.1$$

$$C_{pe\_max\_wy} = \max(C_{pe\_C}, C_{pe\_D}, C_{pe\_W}) = -0.6$$

$$C_{pe\_min\_wy} = \min(C_{pe\_C}, C_{pe\_D}, C_{pe\_W}) = -1.1$$

Roof:

$$C_{pe\_max\_rx} = \max(C_{pe\_EG90}, C_{pe\_FH90}) = -0.76$$

$$C_{pe\_min\_rx} = \min(C_{pe\_EG90}, C_{pe\_FH90}) = -0.8$$

$$C_{pe\_max\_ry} = \max(C_{pe\_EF0}, C_{pe\_GH0}) = -0.3$$

$$C_{pe\_min\_ry} = \min(C_{pe\_EF0}, C_{pe\_GH0}) = -0.5$$

Internal pressure coefficents

$C_{pi\_neg} = -0.2$ $\qquad$ $C_{pi\_pos} = 0.2$

Design load in x direction for the walls:

$$F1x = (C_{pe\_min\_wx} - C_{pi\_neg}) \cdot p_z = -1.488 \ \frac{kN}{m^2}$$

$$F2x = (C_{pe\_min\_wx} - C_{pi\_pos}) \cdot p_z = -2.15 \ \frac{kN}{m^2}$$

$$F3x = (C_{pe\_max\_wx} - C_{pi\_neg}) \cdot p_z = 1.488 \ \frac{kN}{m^2}$$

$$F4x = (C_{pe\_max\_wx} - C_{pi\_pos}) \cdot p_z = 0.827 \ \frac{kN}{m^2}$$

$$F_{wx} = F2x = -2.15 \ \frac{kN}{m^2}$$

Design load in y direction for the walls:

$$F1y = (C_{pe\_min\_wy} - C_{pi\_neg}) \cdot p_z = -1.488 \ \frac{kN}{m^2}$$

$$F2y = (C_{pe\_min\_wy} - C_{pi\_pos}) \cdot p_z = -2.15 \ \frac{kN}{m^2}$$

Hand caluculations for load considerations
Single storey - 2 rooms + kitchen

$$F3y = (C_{pe\_max\_wy} - C_{pi\_neg}) \cdot p_z = -0.662 \ \frac{kN}{m^2}$$

$$F4y = (C_{pe\_max\_wy} - C_{pi\_pos}) \cdot p_z = -1.323 \ \frac{kN}{m^2}$$

$$F_{wy} = F2y = -2.15 \ \frac{kN}{m^2}$$

Design load in x direction for the roof:

$$F1x = (C_{pe\_min\_rx} - C_{pi\_neg}) \cdot p_z = -0.992 \ \frac{kN}{m^2} \qquad F3x = (C_{pe\_max\_rx} - C_{pi\_neg}) \cdot p_z = -0.926 \ \frac{kN}{m^2}$$

$$F2x = (C_{pe\_min\_rx} - C_{pi\_pos}) \cdot p_z = -1.654 \ \frac{kN}{m^2} \qquad F4x = (C_{pe\_max\_rx} - C_{pi\_pos}) \cdot p_z = -1.588 \ \frac{kN}{m^2}$$

$$F_{rx} = F2x = -1.654 \ \frac{kN}{m^2}$$

Design load in y direction for the roof:

$$F1y = (C_{pe\_min\_ry} - C_{pi\_neg}) \cdot p_z = -0.496 \ \frac{kN}{m^2} \qquad F3y = (C_{pe\_max\_ry} - C_{pi\_neg}) \cdot p_z = -0.165 \ \frac{kN}{m^2}$$

$$F2y = (C_{pe\_min\_ry} - C_{pi\_pos}) \cdot p_z = -1.158 \ \frac{kN}{m^2} \qquad F4y = (C_{pe\_max\_ry} - C_{pi\_pos}) \cdot p_z = -0.827 \ \frac{kN}{m^2}$$

$$F_{ry} = F2y = -1.158 \ \frac{kN}{m^2}$$

WL to apply on walls:

$$WL_{wall} = \begin{vmatrix} F_{wx} \\ F_{wy} \end{vmatrix} = \begin{vmatrix} -2.15 \\ -2.15 \end{vmatrix} \ \frac{kN}{m^2}$$

WL to apply on roof:

$$WL_{roof} = \begin{vmatrix} F_{rx} \\ F_{ry} \end{vmatrix} = \begin{vmatrix} -1.654 \\ -1.158 \end{vmatrix} \ \frac{kN}{m^2}$$

Divide the WL on the roof over the rafters:

$$q_{rwx} = F_{rx} \cdot 31.673 \ m^2 = -52.379 \ kN \qquad q_{rwy} = F_{ry} \cdot 31.673 \ m^2 = -36.665 \ kN$$

$$l_r = 8 \cdot 6.15 \ m = 49.2 \ m \qquad l_r = 8 \cdot 6.15 = 49.2 \ m$$

$$q_{rdistx} = \frac{q_{rwx}}{l_r} = -1.065 \ \frac{kN}{m} \qquad q_{rdisty} = \frac{q_{rwy}}{l_r} = -0.745 \ \frac{kN}{m}$$

Hand caluculations for load considerations
Single storey - 2 rooms + kitchen

Earthquake load

Classification of building:
- General provisions and buildings

Zone factor Z Table 2 (Clause 6.4.2)

$Z = 0.36$ Very severe

Importance factor I Table 6

$I = 1.0$

Response reduction factor R as per Table 7 and clause 6.4.2)

$R = 3$ Category v) b) Reinf h RC bands
Should be v) c) Reinf h RC + vertical? = 3.0, Resilient 2.5

$\frac{I}{R} < 1 = 1$

Time period of the building as per 7.6.2

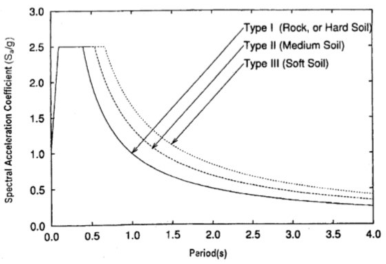
$T_x = 0.09 \frac{s}{m^{0.5}} \frac{h}{D_x^{0.5}} = 0.144\ s$ Along x

$T_y = 0.09 \frac{s}{m^{0.5}} \frac{h}{D_y^{0.5}} = 0.161\ s$ Along y

Soil profile:
Assumed to be sub soil category III (soft soil site)

Average response acceleration coefficient S a/g
(from soil type and time period T)

Along x	Along y
$0\ s \le T_x \le 0.1\ s = 0$	$0\ s \le T_y \le 0.1\ s = 0$
$0.1\ s \le T_x \le 0.67\ s = 1$	$0.1\ s \le T_y \le 0.67\ s = 1$
$0.67\ s \le T_x \le 4\ s = 0$	$0.67\ s \le T_y \le 4\ s = 0$



$S_{ax} = 2.5$ $g_x = 1$ $S_{ay} = 2.5$ $g_y = 1$

$\frac{S_{ax}}{g_x} = 2.5$ $\frac{S_{ay}}{g_y} = 2.5$

Hand caluculations for load considerations
Single storey - 2 rooms + kitchen

The design horizontal seismic coefficient A_h

$$A_{hx} = \frac{Z \cdot I \cdot S_{ax}}{2 \cdot R \cdot g_x} = 0.15$$ if T<= 0.1 s -> A_h=Z/2

$$A_{hy} = \frac{Z \cdot I \cdot S_{ay}}{2 \cdot R \cdot g_y} = 0.15$$

Seismic weight of the building W as per 7.4.2

" *The seismic weight of each floor is its full dead load plus appropriate amount of imposed load, as specified in 7.3.1 and 7.3.2. While computing the seismic weight of each floor, the weight of columns and walls in any storey shall be equally distributed to the floors above and below the storey.* "

Treat each of roof, lintel and sill RCC bands as one storey and apply earthquake load in proportion to tributary area and weight of each "floor":

$W_{ave} = 4.1 \ \frac{kN}{m^2}$ Adjusted estimated value to get closer to total seaismic weight from SAP

$h_{sill} = 0.780 \ m$ $h_{lintel} = 2.03 \ m$ $h_{roof} = 2.545 \ m$

Rough estimation of the roof weight :

$$W_{rafter} = (76.97 \cdot 6.6 \cdot 10^{-4} \cdot (1.65 + 3.5)) \cdot 7 \ kN = 1.831 \cdot 10^3 \ N$$

$$W_{purlin} = (76.97 \cdot 4.45 \cdot 10^{-4} \cdot (6.15)) \cdot 6 \ kN = 1.264 \cdot 10^3 \ N$$

$$W_{ridge} = 76.97 \cdot 1.1 \cdot 10^{-3} \cdot 6.15 \ kN = 520.702 \ N$$

$$W_{r.p.r} = W_{rafter} + W_{purlin} + W_{ridge} = 3.616 \cdot 10^3 \ N$$

$$A_{roof} = 1.65 \ m \cdot 6.15 \ m + 3.5 \ m \cdot 6.15 \ m = 31.673 \ m^2$$

$$W_{rafters} = \frac{W_{r.p.r}}{A_{roof}} = 0.114 \ \frac{kN}{m^2}$$

Weight of CGI and roof rafters:

$$W_{CGI} = 0.043164 \ \frac{kN}{m^2}$$

$$W_{CGI.rafters} = W_{CGI} + W_{rafters} = 0.157 \ \frac{kN}{m^2}$$

$$W_{CGI.rafters} \cdot A_{roof} = 4.983 \ kN$$

New roof area from SAP:

$A_{gabble} = 1.3923$ $A_{gabbleold} = 0.6423$

$$ratio_{gabble} = \frac{A_{gabble}}{A_{gabbleold}} = 2.168$$

Hand caluculations for load considerations
Single storey - 2 rooms + kitchen

Roof RCC band:

$$A_{roof} = ratio_{gabble} \cdot \begin{vmatrix} 0.484 \\ 0.461 \\ 0.461 \\ 0.484 \\ 0.839 \\ 0.839 \\ 0.839 \\ 0.815 \\ 0.815 \\ 0.839 \\ 0.484 \\ 0.461 \\ 0.461 \\ 0.484 \\ 0.535 \end{vmatrix} m^2 \qquad W_{roof\_i1} = A_{roof} \cdot W_{ave} = \begin{vmatrix} 4.302 \\ 4.097 \\ 4.097 \\ 4.302 \\ 7.457 \\ 7.457 \\ 7.457 \\ 7.243 \\ 7.243 \\ 7.457 \\ 4.302 \\ 4.097 \\ 4.097 \\ 4.302 \\ 4.755 \end{vmatrix} kN$$

Areas and weights from rafters added to roof RCC band:

$$A_{only\_roof} = \begin{vmatrix} 1.479 \\ 1.443 \\ 1.449 \\ 1.470 \\ 1.470 \\ 2.250 \\ 1.470 \\ 1.447 \\ 2.250 \\ 1.460 \\ 1.822 \\ 1.823 \\ 1.823 \\ 1.849 \\ \vdots \end{vmatrix} m^2 \qquad W_{only\_roof} = A_{only\_roof} \cdot W_{CGI.rafters} = \begin{vmatrix} 0.233 \\ 0.227 \\ 0.228 \\ 0.231 \\ 0.231 \\ 0.354 \\ 0.231 \\ 0.228 \\ 0.354 \\ 0.23 \\ 0.287 \\ 0.287 \\ 0.287 \\ 0.291 \\ 0.27 \end{vmatrix} kN$$

Hand caluculations for load considerations
Single storey - 2 rooms + kitchen

Total seismic weight at roof RCC band

$$W_{roof\_i} = W_{roof\_i1} + W_{only\_roof} = \begin{vmatrix} 4.534 \\ 4.324 \\ 4.325 \\ 4.533 \\ 7.688 \\ 7.811 \\ 7.688 \\ 7.471 \\ 7.597 \\ 7.686 \\ 4.588 \\ 4.384 \\ 4.384 \\ 4.592 \\ 5.024 \end{vmatrix} \ \mathbf{kN}$$

Lintel RCC band:

$$A_{lintel} = \begin{vmatrix} 1.088 \\ 1.108 \\ 1.108 \\ 1.088 \\ 1.174 \\ 1.261 \\ 1.390 \\ 1.022 \\ 0.936 \\ 1.390 \\ 1.390 \\ 1.324 \\ 1.022 \\ 1.088 \\ 1.769 \\ 0.758 \\ 0.761 \\ 0.847 \\ 0.579 \\ 1.010 \end{vmatrix} \ \mathbf{m}^2 \qquad W_{lintel\_i1} = A_{lintel} \cdot W_{ave} = \begin{vmatrix} 4.461 \\ 4.543 \\ 4.543 \\ 4.461 \\ 4.813 \\ 5.17 \\ 5.699 \\ 4.19 \\ 3.838 \\ 5.699 \\ 5.699 \\ 5.428 \\ 4.19 \\ 4.461 \\ 7.253 \\ 3.108 \\ 3.12 \\ 3.473 \\ 2.374 \\ 4.141 \end{vmatrix} \ \mathbf{kN}$$

Areas and
weights from
rafters added to
lintel RCC band:

$$A_{only\_lintel} = \begin{vmatrix} 0 \\ 0 \\ 0 \\ 0 \\ 0 \\ 0 \\ 0 \\ 0 \\ \vdots \end{vmatrix} \ \mathbf{m}^2 \qquad W_{only\_lintel} = A_{only\_lintel} \cdot W_{CGI.rafters} = \begin{array}{c} 0 \\ 1 \\ 2 \\ 3 \\ 4 \\ 5 \\ \vdots \\ 19 \end{array} \begin{vmatrix} 0 \\ 0 \\ 0 \\ 0 \\ 0 \\ 0 \\ \vdots \\ \ \end{vmatrix} \ \mathbf{kN}$$

Hand caluculations for load considerations
Single storey - 2 rooms + kitchen

Total seismic weight at lintel RCC band:

$$W_{lintel\_i} = W_{lintel\_i1} + W_{only\_lintel} = \begin{vmatrix} 4.461 \\ 4.543 \\ 4.543 \\ 4.461 \\ 4.813 \\ 5.17 \\ 5.699 \\ 4.19 \\ 3.838 \\ 5.699 \\ 5.699 \\ 5.428 \\ 4.19 \\ 4.461 \\ 7.458 \\ 3.272 \\ 3.283 \\ 3.836 \\ 2.594 \\ 4.349 \end{vmatrix} \text{kN}$$

Total seismic weight at sill RCC band:

$$A_{sill} = \begin{vmatrix} 1.336 \\ 1.344 \\ 1.337 \\ 1.336 \\ 1.422 \\ 1.404 \\ 1.638 \\ 0.999 \\ 0.858 \\ 1.638 \\ 1.638 \\ 1.560 \\ 1.014 \\ 1.092 \\ 1.950 \\ 0.546 \\ 1.336 \\ 1.422 \\ 0.624 \\ 1.456 \end{vmatrix} m^2 \qquad W_{sill\_i} = A_{sill} \cdot W_{ave} = \begin{vmatrix} 5.478 \\ 5.51 \\ 5.482 \\ 5.478 \\ 5.83 \\ 5.756 \\ 6.716 \\ 4.096 \\ 3.518 \\ 6.716 \\ 6.716 \\ 6.396 \\ 4.157 \\ 4.477 \\ 7.995 \\ 2.239 \\ 5.478 \\ 5.83 \\ 2.558 \\ 5.97 \end{vmatrix} \text{kN}$$

Hand caluculations for load considerations
Single storey - 2 rooms + kitchen

Tot weight from SAP for comparison

$G = 232.071$ kN From SAP

Total weight of each "floor":

$$W_{sill} = \left(\sum_{i=1}^{19} W_{sill\_i}(i)\right) = 100.917 \text{ kN}$$

$$W_{lintel} = \left(\sum_{i=1}^{19} W_{lintel\_i}(i)\right) = 87.526 \text{ kN}$$

$$W_{roof} = \left(\sum_{i=1}^{14} W_{roof\_i}(i)\right) = 82.096 \text{ kN}$$

$$W_{tot} = W_{sill} + W_{lintel} + W_{roof} = 270.539 \text{ kN}$$

Total seismic weight of building
(compare to SAP value)

$$Part_{sill} = \frac{W_{sill}}{W_{tot}} = 0.373 \qquad Part_{lintel} = \frac{W_{lintel}}{W_{tot}} = 0.324 \qquad Part_{roof} = \frac{W_{roof}}{W_{tot}} = 0.303$$

$$Part_{tot} = Part_{sill} + Part_{lintel} + Part_{roof} = 1$$

<u>Seismic base shear V_base</u> as per 7.5.3

$$V_{bx} = A_{hx} \cdot W_{tot} = 40.581 \text{ kN} \qquad D_x = 6.15 \text{ m}$$

$$V_{by} = A_{hy} \cdot W_{tot} = 40.581 \text{ kN} \qquad D_y = 4.95 \text{ m}$$

<u>Distribution of design force</u> as per 7.7.1

$$Q_i = V_B \frac{W_i h_i^2}{\sum_{j=1}^{n} W_j h_j^2}$$

Q_i = Design lateral force at floor i
W_i = Seismic weight of floor i
h_i = Height of floor i measured from base
n = numer of storeys

Hand caluculatic ns
Single storey - .

$$Q_i = V_B \frac{W_i h_i^2}{\sum_{j=1}^{n} W_j h_j^2}$$

Proportion of base shear applied on sill band

$$Q_{sill\_x} = \frac{V_{bx} \cdot W_{sill\_i} \cdot h_{sill}^{\ 2}}{W_{sill} \cdot h_{sill}^{\ 2} + W_{lintel} \cdot h_{lintel}^{\ 2} + W_{roof} \cdot h_{roof}^{\ 2}} = \begin{vmatrix} 0.142 \\ 0.143 \\ 0.142 \\ 0.142 \\ 0.151 \\ 0.149 \\ 0.174 \\ 0.106 \\ 0.091 \\ 0.174 \\ 0.174 \\ 0.166 \\ 0.108 \\ 0.116 \\ 0.207 \\ 0.058 \\ 0.142 \\ 0.151 \\ 0.066 \\ 0.155 \end{vmatrix} \ \mathbf{kN}$$

$$Q_{sill\_y} = \frac{V_{by} \cdot W_{sill\_i} \cdot h_{sill}^{\ 2}}{W_{sill} \cdot h_{sill}^{\ 2} + W_{lintel} \cdot h_{lintel}^{\ 2} + W_{roof} \cdot h_{roof}^{\ 2}} = \begin{vmatrix} 0.142 \\ 0.143 \\ 0.142 \\ 0.142 \\ 0.151 \\ 0.149 \\ 0.174 \\ 0.106 \\ 0.091 \\ 0.174 \\ 0.174 \\ 0.166 \\ 0.108 \\ 0.116 \\ 0.207 \\ 0.058 \\ 0.142 \\ 0.151 \\ 0.066 \\ 0.155 \end{vmatrix} \ \mathbf{kN}$$

Hand caluculations for load considerations
Single storey - 2 rooms + kitchen

Proportion of base shear applied on lintel band

$$Q_{lintel\_x} = \frac{V_{bx} \cdot W_{lintel\_i} \cdot h_{lintel}^{\ 2}}{W_{sill} \cdot h_{sill}^{\ 2} + W_{lintel} \cdot h_{lintel}^{\ 2} + W_{roof} \cdot h_{roof}^{\ 2}} = \begin{vmatrix} 0.782 \\ 0.796 \\ 0.796 \\ 0.782 \\ 0.844 \\ 0.906 \\ 0.999 \\ 0.735 \\ 0.673 \\ 0.999 \\ 0.999 \\ 0.952 \\ 0.735 \\ 0.782 \\ 1.308 \\ 0.574 \\ 0.576 \\ 0.673 \\ 0.455 \\ 0.762 \end{vmatrix} \ \mathbf{kN}$$

$$Q_{lintel\_y} = \frac{V_{by} \ W_{lintel\_i} \cdot h_{lintel}^{\ 2}}{W_{sill} \cdot h_{sill}^{\ 2} + W_{lintel} \cdot h_{lintel}^{\ 2} + W_{roof} \cdot h_{roof}^{\ 2}} = \begin{vmatrix} 0.782 \\ 0.796 \\ 0.796 \\ 0.782 \\ 0.844 \\ 0.906 \\ 0.999 \\ 0.735 \\ 0.673 \\ 0.999 \\ 0.999 \\ 0.952 \\ 0.735 \\ 0.782 \\ 1.308 \\ 0.574 \\ 0.576 \\ 0.673 \\ 0.455 \\ 0.762 \end{vmatrix} \ \mathbf{kN}$$

Hand caluculations for load considerations
Single storey - 2 rooms + kitchen

Proportion of base shear applied on roof band

$$Q_{roof\_x} = \frac{V_{bx} \cdot W_{roof\_i} \cdot h_{roof}^{\ 2}}{W_{sill} \cdot h_{sill}^{\ 2} + W_{lintel} \cdot h_{lintel}^{\ 2} + W_{roof} \cdot h_{roof}^{\ 2}} = \begin{vmatrix} 1.249 \\ 1.192 \\ 1.192 \\ 1.249 \\ 2.119 \\ 2.152 \\ 2.119 \\ 2.059 \\ 2.094 \\ 2.118 \\ 1.264 \\ 1.208 \\ 1.208 \\ 1.266 \\ 1.385 \end{vmatrix} \ \mathrm{kN}$$

$$Q_{roof\_y} = \frac{V_{by} \cdot W_{roof\_i} \cdot h_{roof}^{\ 2}}{W_{sill} \cdot h_{sill}^{\ 2} + W_{lintel} \cdot h_{lintel}^{\ 2} + W_{roof} \cdot h_{roof}^{\ 2}} = \begin{vmatrix} 1.249 \\ 1.192 \\ 1.192 \\ 1.249 \\ 2.119 \\ 2.152 \\ 2.119 \\ 2.059 \\ 2.094 \\ 2.118 \\ 1.264 \\ 1.208 \\ 1.208 \\ 1.266 \\ 1.385 \end{vmatrix} \ \mathrm{kN}$$

$$Q_{sill} = \left(\sum_{i=1}^{19} Q_{sill\_x}(i)\right) = 2.612 \ \mathrm{kN} \qquad Q_{lintel} = \left(\sum_{i=1}^{19} Q_{lintel\_x}(i)\right) = 15.346 \ \mathrm{kN}$$

$$Q_{roof} = \left(\sum_{i=1}^{14} Q_{roof\_x}(i)\right) = 22.623 \ \mathrm{kN} \qquad Q_{tot} = Q_{sill} + Q_{lintel} + Q_{roof} = 40.581 \ \mathrm{kN}$$

Hand calculations for load considerations - X3

Geometrical inputs:

$l = 6.15\ \text{m}$	Length of building
$h = (3.472 + 2.23)\ \text{m} = 5.702\ \text{m}$	Heigth of building
$w = 4.95\ \text{m}$	Width of building
$\alpha = 17\ °$	Angle of roof slope
$alpha = 17$	Angle of roos for imposed loads
$D_x = 6.15\ \text{m}$	Dimensions of the building along x -axis
$D_y = 4.95\ \text{m}$	Dimensions of the building along y -axis

$t_{brick} = 0.15\ \text{m}$

Material properties of CSEB:

$f_{ck\_CSEB} = 5\ \dfrac{\text{N}}{\text{mm}^2}$ $\qquad E_{CSEB} = 700\ \text{MPa}$

$\rho_{CSEB} = 19\ \dfrac{\text{kN}}{\text{m}^3}$ $\qquad v_{CSEB} = 0.15$

$f_{y\_CSEB} = 500\ \dfrac{\text{N}}{\text{mm}^2}$ $\qquad W_{CSEBm} = 19.0\ \dfrac{\text{kN}}{\text{m}^3}$

Material properties of structural steel (roof):

$\rho_{steel} = 78.5\ \dfrac{\text{kN}}{\text{m}^3}$ $\qquad E_{steel} = 200000\ \text{MPa}$

IS 800 : 2007
Fe 250

$f_{y\_strc\_steel} = 250\ \dfrac{\text{N}}{\text{mm}^2}$ $\qquad v_{steel} = 0.3$

Material poperties of reinforcing steel:

$f_{y\_steel} = 500\ \dfrac{\text{N}}{\text{mm}^2}$

IS 1786 : 2008
Fe 500

Material poperties of concret

$f_{ck\_concret} = 20\ \dfrac{\text{N}}{\text{mm}^2}$

Ordinary concrete M 20
IS 456:2000

Material poperties of mortar

$f_{ck\_mortar} = 2.5\ \dfrac{\text{N}}{\text{mm}^2}$

EKS 10, Table H -2

Hand caluculations for load considerations
Two storey

LOADS:

- Dead loads: as per IS 875 : 1984 Part I
- Live loads: as per IS 875 : 1984 Part II
- Wind loads: as per NBC 104 : 1994 and IS 875 : 1984 Part III
- Earthquake load: as per IS 1893 : 2002 (Or NBC 105:1994?)

Dead load:

$G = 608.43\ \mathrm{kN}$ From SAP without false ceiling

Imposed load:

$q_{floor} = 2.0\ \frac{\mathrm{kN}}{\mathrm{m}^2}$ As per Table 1 - Residential building all floors

Table 2: Imposed loads on various types of roofs
ii) sloping roof with slope greater than 10 degreees

$q_r = 0.75\ \frac{\mathrm{kN}}{\mathrm{m}^2} - \left(0.02\ \frac{\mathrm{kN}}{\mathrm{m}^2}\left(alpha - 10\right)\right) = 0.61\ \frac{\mathrm{kN}}{\mathrm{m}^2}$

Lateral load reduction per floor as per 3.2
-10%

$q_{d.floor} = 0.9 \cdot q_{floor} = 1.8\ \frac{\mathrm{kN}}{\mathrm{m}^2}$

$q_{d.r} = 0.9 \cdot q_r = 0.549\ \frac{\mathrm{kN}}{\mathrm{m}^2}$

Divide the imposed load over the roof rafters

$q_{r1} = q_{d.r} \cdot 31.673\ \mathrm{m}^2 = 17.388\ \mathrm{kN}$

$l_r = 8 \cdot 6.15\ \mathrm{m} = 49.2\ \mathrm{m}$

$q_{rdist} = \frac{q_{r1}}{l_r} = 0.353\ \frac{\mathrm{kN}}{\mathrm{m}}$

Hand caluculations for load considerations
Two storey

Wind loads:

$V_b = 50\ \frac{m}{s}$ — Basic wind speed

$k_1 = 1$ — Probability factor (risk coeff.) from Table 1

$k_2 = 1.05$ — Terrain, height and structure size factor from Table 2 Category 1 Class A?

$k_3 = 1$ — Topography factor

$V_z = V_b \cdot k_1 \cdot k_2 \cdot k_3 = 52.5\ \frac{m}{s}$ — Design wind speed

$p_z = 0.6\ \frac{Pa \cdot s^2}{m^2} \cdot V_z^{\ 2} = (1.654 \cdot 10^3)\ \frac{N}{m^2}$ — Design wind pressure, accor. to 5.4 " the coeff. 0.6 SI units depends on a number of factors..."

$\frac{h}{w} = 1.152$ $\quad \frac{l}{w} = 1.242$ — Wind angle tetha = 0

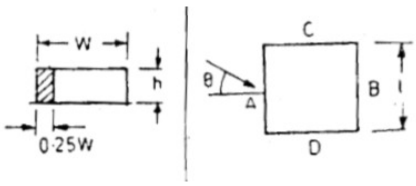
WALLS:
Decide external pressure coefficients (C_pe) from ratios (Table 4 - Rectangular clad buildings):

$\frac{h}{w} \le \frac{1}{2} = 0$ $\quad 1 < \frac{l}{w} \le \frac{3}{2} = 1$

$\frac{1}{2} < \frac{h}{w} \le \frac{3}{2} = 1$ $\quad \frac{3}{2} < \frac{l}{w} < 4 = 0$

$\frac{3}{2} < \frac{h}{w} \le 6 = 0$

These coefficent used from second h/w condition and first l/w condition



$C_{pe\_A} = 0.7$ $\quad C_{pe\_C} = -0.6$

$C_{pe\_B} = -0.25$ $\quad C_{pe\_D} = -0.6$

W

$C_{pe\_W} = -1.1$

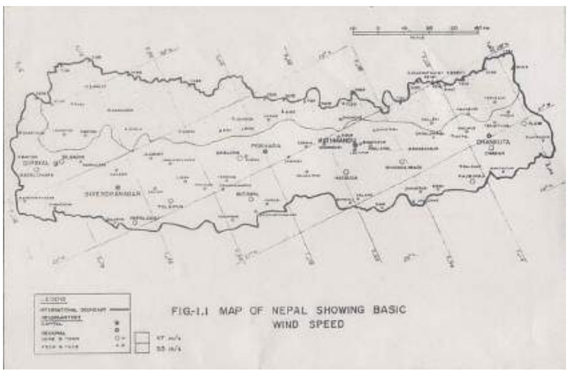


FIG-1.1 MAP OF NEPAL SHOWING BASIC WIND SPEED

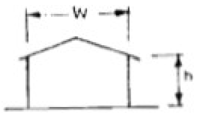
Hand caluculations for load considerations
Two storey

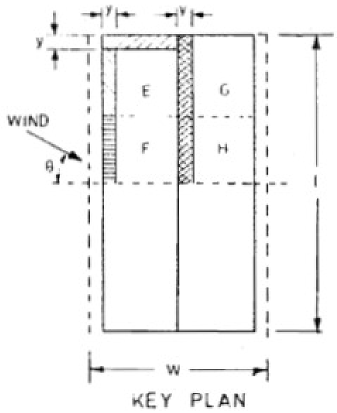
ROOF:
Ratios for C_pe on the roof (Table 5)

$$\frac{h}{w} \le \frac{1}{2} = 0$$

$$\frac{1}{2} < \frac{h}{w} \le \frac{3}{2} = 1$$

$$\frac{3}{2} < \frac{h}{w} \le 6 = 0$$





$y = h$ or $0 \cdot 15\, w$, whichever is the lesser.

$x = 17$ -> Linear interpolation between 10 -20 degrees

$x_0 = 10$
$x_1 = 20$

EF Tetha = 0 degrees

$y_0 = -1.1$
$y_1 = -0.7$

$$C_{pe\_EF0} = y_0 + (x - x_0) \cdot \frac{(y_1 - y_0)}{(x_1 - x_0)} = -0.82$$

GH Tetha = 0 degrees

$y_0 = -0.6$
$y_1 = -0.5$

$$C_{pe\_GH0} = y_0 + (x - x_0) \cdot \frac{(y_1 - y_0)}{(x_1 - x_0)} = -0.53$$

EG Tetha = 90 degrees

$y_0 = -0.8$
$y_1 = -0.8$

$$C_{pe\_EG90} = y_0 + (x - x_0) \cdot \frac{(y_1 - y_0)}{(x_1 - x_0)} = -0.8$$

FH Tetha = 90 degrees

$y_0 = -0.6$
$y_1 = -0.6$

$$C_{pe\_FH90} = y_0 + (x - x_0) \cdot \frac{(y_1 - y_0)}{(x_1 - x_0)} = -0.6$$

Hand caluculations for load considerations
Two storey

walls:

$C_{pe\_max\_wx} = \max(C_{pe\_A}, C_{pe\_B}, C_{pe\_W}) = 0.7$

$C_{pe\_min\_wx} = \min(C_{pe\_A}, C_{pe\_B}, C_{pe\_W}) = -1.1$

$C_{pe\_max\_wy} = \max(C_{pe\_C}, C_{pe\_D}, C_{pe\_W}) = -0.6$

$C_{pe\_min\_wy} = \min(C_{pe\_C}, C_{pe\_D}, C_{pe\_W}) = -1.1$

Roof:

$C_{pe\_max\_rx} = \max(C_{pe\_EG90}, C_{pe\_FH90}) = -0.6$

$C_{pe\_min\_rx} = \min(C_{pe\_EG90}, C_{pe\_FH90}) = -0.8$

$C_{pe\_max\_ry} = \max(C_{pe\_EF0}, C_{pe\_GH0}) = -0.53$

$C_{pe\_min\_ry} = \min(C_{pe\_EF0}, C_{pe\_GH0}) = -0.82$

Internal pressure coefficents

$C_{pi\_neg} = -0.2$ $\qquad$ $C_{pi\_pos} = 0.2$

<u>Design load in x direction for the walls:</u>

$F1x = (C_{pe\_min\_wx} - C_{pi\_neg}) \cdot p_z = -1.488 \ \frac{kN}{m^2}$

$F2x = (C_{pe\_min\_wx} - C_{pi\_pos}) \cdot p_z = -2.15 \ \frac{kN}{m^2}$

$F3x = (C_{pe\_max\_wx} - C_{pi\_neg}) \cdot p_z = 1.488 \ \frac{kN}{m^2}$

$F4x = (C_{pe\_max\_wx} - C_{pi\_pos}) \cdot p_z = 0.827 \ \frac{kN}{m^2}$

$F_{wx} = F2x = -2.15 \ \frac{kN}{m^2}$

<u>Design load in y direction for the walls:</u>

$F1y = (C_{pe\_min\_wy} - C_{pi\_neg}) \cdot p_z = -1.488 \ \frac{kN}{m^2}$

$F2y = (C_{pe\_min\_wy} - C_{pi\_pos}) \cdot p_z = -2.15 \ \frac{kN}{m^2}$

Hand caluculations for load considerations
Two storey

$$F3y = (C_{pe\_max\_wy} - C_{pi\_neg}) \cdot p_z = -0.662 \; \frac{kN}{m^2}$$

$$F4y = (C_{pe\_max\_wy} - C_{pi\_pos}) \cdot p_z = -1.323 \; \frac{kN}{m^2}$$

$$F_{wy} = F2y = -2.15 \; \frac{kN}{m^2}$$

Design load in x direction for the roof:

$$F1x = (C_{pe\_min\_rx} - C_{pi\_neg}) \cdot p_z = -0.992 \; \frac{kN}{m^2} \qquad F3x = (C_{pe\_max\_rx} - C_{pi\_neg}) \cdot p_z = -0.662 \; \frac{kN}{m^2}$$

$$F2x = (C_{pe\_min\_rx} - C_{pi\_pos}) \cdot p_z = -1.654 \; \frac{kN}{m^2} \qquad F4x = (C_{pe\_max\_rx} - C_{pi\_pos}) \cdot p_z = -1.323 \; \frac{kN}{m^2}$$

$$F_{rx} = F2x = -1.654 \; \frac{kN}{}$$

Design load in y direction for the roof:

$$F1y = (C_{pe\_min\_ry} - C_{pi\_neg}) \cdot p_z = -1.025 \; \frac{kN}{m^2} \qquad F3y = (C_{pe\_max\_ry} - C_{pi\_neg}) \cdot p_z = -0.546 \; \frac{kN}{m^2}$$

$$F2y = (C_{pe\_min\_ry} - C_{pi\_pos}) \cdot p_z = -1.687 \; \frac{kN}{m^2} \qquad F4y = (C_{pe\_max\_ry} - C_{pi\_pos}) \cdot p_z = -1.207 \; \frac{kN}{m^2}$$

$$F_{ry} = F2y = -1.687 \; \frac{kN}{m^2}$$

WL to apply on walls:

$$WL_{wall} = \begin{vmatrix} F_{wx} \\ F_{wy} \end{vmatrix} = \begin{vmatrix} -2.15 \\ -2.15 \end{vmatrix} \; \frac{kN}{m^2}$$

WL to apply on roof:

$$WL_{roof} = \begin{vmatrix} F_{rx} \\ F_{ry} \end{vmatrix} = \begin{vmatrix} -1.654 \\ -1.687 \end{vmatrix} \; \frac{kN}{m^2}$$

Divide WL on roof on rafters:

$$q_{rwx} = F_{rx} \cdot 31.673 \; m^2 = -52.379 \; kN \qquad q_{rwy} = F_{ry} \cdot 31.673 \; m^2 = -53.427 \; kN$$

$$l_r = 8 \cdot 6.15 \; m = 49.2 \; m \qquad l_r = 8 \cdot 6.15 \; m = 49.2 \; m$$

$$q_{rdistx} = \frac{q_{rwx}}{l_r} = -1.065 \; \frac{kN}{m} \qquad q_{rdisty} = \frac{q_{rwy}}{l_r} = -1.086 \; \frac{kN}{m}$$

Hand caluculations for load considerations
Two storey

Earthquake load

<u>Classification of building:</u>
- General provisions and buildings

<u>Zone factor Z</u> Table 2 (Clause 6.4.2)

$Z = 0.36$ Very severe

<u>Importance factor I</u> Table 6

$I = 1.0$

<u>Response reduction factor R</u> as per Table 7 and clause 6.4.2)

$R = 3$ Category v) b) Reinf h RC bands
Should be v) c) Reinf h RC + vertical? = 3.0, Resilient 2.5

$\frac{I}{R} < 1 = 1$

<u>Time period of the building</u> as per 7.6.2

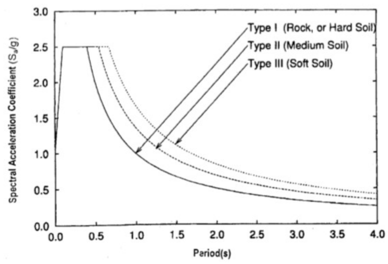
$T_x = 0.09 \frac{s}{m^{0.5}} \frac{h}{D_x^{\,0.5}} = 0.207\ s$ Along x

$T_y = 0.09 \frac{s}{m^{0.5}} \frac{h}{D_y^{\,0.5}} = 0.231\ s$ Along y

<u>Soil profile:</u>
Assumed to be sub soil category III (soft soil site)

<u>Average response acceleration coefficient S a/g</u>
(from soil type and time period T)

Along x	Along y
$0\ s \le T_x \le 0.1\ s = 0$	$0\ s \le T_y \le 0.1\ s = 0$
$0.1\ s \le T_x \le 0.67\ s = 1$	$0.1\ s \le T_y \le 0.67\ s = 1$
$0.67\ s \le T_x \le 4\ s = 0$	$0.67\ s \le T_y \le 4\ s = 0$



$S_{ax} = 2.5$ $g_x = 1$ $S_{ay} = 2.5$ $g_y = 1$

$\frac{S_{ax}}{g_x} = 2.5$ $\frac{S_{ay}}{g_y} = 2.5$

Hand caluculations for load considerations
Two storey

The design horizontal seismic coefficient A_h

$$A_{hx} = \frac{Z \cdot I \cdot S_{ax}}{2 \cdot R \cdot g_x} = 0.15$$ if T<= 0.1 s -> A_h=Z/2

$$A_{hy} = \frac{Z \cdot I \cdot S_{ay}}{2 \cdot R \cdot g_y} = 0.15$$

Seismic weight of the building W as per 7.4.2

"*The seismic weight of each floor is its full dead load plus appropriate amount of imposed load, as specified in 7.3.1 and 7.3.2. While computing the seismic weight of each floor, the weight of columns and walls in any storey shall be equally distributed to the floors above and below the storey.* "

Treat each of roof, lintel and sill RCC bands as one storey and apply earthquake load in proportion to tributary area and weight of each "floor":

$W_{ave} = 4.1 \; \frac{kN}{m^2}$ Adjusted estimated value to get closer to total seaismic weight from SAP

$h_{sill0} = 0.780 \; m$ $h_{lintel0} = 2.03 \; m$ $h_{floor} = 2.23 \; m$

$h_{sill1} = 3.01 \; m$ $h_{lintel1} = 4.25 \; m$ $h_{roof} = 4.775 \; m$

Rough estimation of the roof weight :

$$W_{rafter} = (76.97 \cdot 6.6 \cdot 10^{-4} \cdot (1.65 + 3.5)) \cdot 7 \; kN = 1.831 \cdot 10^3 \; N$$

$$W_{purlin} = (76.97 \cdot 4.45 \cdot 10^{-4} \cdot (6.15)) \cdot 6 \; kN = 1.264 \cdot 10^3 \; N$$

$$W_{ridge} = 76.97 \cdot 1.1 \cdot 10^{-3} \cdot 6.15 \; kN = 520.702 \; N$$

$$W_{r.p.r} = W_{rafter} + W_{purlin} + W_{ridge} = 3.616 \cdot 10^3 \; N$$

$$A_{roof} = 1.65 \; m \cdot 6.15 \; m + 3.5 \; m \cdot 6.15 \; m = 31.673 \; m^2$$

$$W_{rafters} = \frac{W_{r.p.r}}{A_{roof}} = 0.114 \; \frac{kN}{m^2}$$

Weight of CGI and roof rafters:

$$W_{CGI} = 0.043164 \; \frac{kN}{m^2}$$

$$W_{CGI.rafters} = W_{CGI} + W_{rafters} = 0.157 \; \frac{kN}{m^2}$$

$$W_{CGI.rafters} \cdot A_{roof} = 4.983 \; kN$$

Hand caluculations for load considerations
Two storey

Roof RCC band:

$$A_{roof} = \begin{vmatrix} 0.484 \\ 0.461 \\ 0.461 \\ 0.484 \\ 0.839 \\ 0.839 \\ 0.839 \\ 0.815 \\ 0.815 \\ 0.839 \\ 0.484 \\ 0.461 \\ 0.461 \\ 0.484 \\ 0.535 \end{vmatrix} m^2 \qquad W_{roof\_i1} = A_{roof} \cdot W_{ave} = \begin{vmatrix} 1.984 \\ 1.89 \\ 1.89 \\ 1.984 \\ 3.44 \\ 3.44 \\ 3.44 \\ 3.342 \\ 3.342 \\ 3.44 \\ 1.984 \\ 1.89 \\ 1.89 \\ 1.984 \\ 2.194 \end{vmatrix} kN$$

Areas and weights from rafters added to roof RCC band:

$$A_{only\_roof} = \begin{vmatrix} 1.479 \\ 1.443 \\ 1.449 \\ 1.470 \\ 1.470 \\ 2.250 \\ 1.470 \\ 1.447 \\ 2.250 \\ 1.460 \\ 1.822 \\ 1.823 \\ 1.823 \\ 1.849 \\ 1.714 \end{vmatrix} m^2 \qquad W_{only\_roof} = A_{only\_roof} \cdot W_{CGI.rafters} = \begin{vmatrix} 0.233 \\ 0.227 \\ 0.228 \\ 0.231 \\ 0.231 \\ 0.354 \\ 0.231 \\ 0.228 \\ 0.354 \\ 0.23 \\ 0.287 \\ 0.287 \\ 0.287 \\ 0.291 \\ 0.27 \end{vmatrix} kN$$

Hand caluculations for load considerations
Two storey

Total seismic weight at roof RCC band
1st floor

$$W_{roof\_i} = W_{roof\_i1} + W_{only\_roof} = \begin{vmatrix} 2.217 \\ 2.117 \\ 2.118 \\ 2.216 \\ 3.671 \\ 3.794 \\ 3.671 \\ 3.569 \\ 3.695 \\ 3.67 \\ 2.271 \\ 2.177 \\ 2.177 \\ 2.275 \\ 2.463 \end{vmatrix} \ kN$$

Lintel RCC band:
1st floor

$$A_{lintel1} = \begin{vmatrix} 1.088 \\ 1.108 \\ 1.108 \\ 1.088 \\ 1.174 \\ 1.261 \\ 1.390 \\ 1.022 \\ 0.936 \\ 1.390 \\ 1.390 \\ 1.324 \\ 1.022 \\ 1.088 \\ 1.769 \\ 0.758 \\ 0.761 \\ 0.847 \\ 0.579 \\ 1.931 \\ 1.265 \end{vmatrix} \ m^2 \qquad W_{lintel\_i1} = A_{lintel1} \cdot W_{ave} = \begin{vmatrix} 4.461 \\ 4.543 \\ 4.543 \\ 4.461 \\ 4.813 \\ 5.17 \\ 5.699 \\ 4.19 \\ 3.838 \\ 5.699 \\ 5.699 \\ 5.428 \\ 4.19 \\ 4.461 \\ 7.253 \\ 3.108 \\ 3.12 \\ 3.473 \\ 2.374 \\ 7.917 \\ 5.187 \end{vmatrix} \ kN$$

Hand caluculations for load considerations
Two storey

Areas and weights from rafters added to lintel RCC band: 1st floor

$$A_{roof\_lintel} = \begin{vmatrix} 0 \\ 0 \\ 0 \\ 0 \\ 0 \\ 0 \\ 0 \\ 0 \\ 0 \\ 0 \\ 0 \\ 0 \\ 0 \\ 0 \\ 1.305 \\ 1.043 \\ 1.033 \\ 2.312 \\ 1.462 \\ 1.5 \\ 1.478 \end{vmatrix} m^2 \qquad W_{roof\_lintel} = A_{roof\_lintel} \cdot W_{CGI.rafters} = \begin{vmatrix} 0 \\ 0 \\ 0 \\ 0 \\ 0 \\ 0 \\ 0 \\ 0 \\ 0 \\ 0 \\ 0 \\ 0 \\ 0 \\ 0 \\ 0.205 \\ 0.164 \\ 0.163 \\ 0.364 \\ 0.23 \\ 0.236 \\ 0.233 \end{vmatrix} kN$$

Total seismic weight at lintel RCC band: 1st floor

$$W_{lintel\_i1} = W_{lintel\_i1} + W_{roof\_lintel} = \begin{vmatrix} 4.461 \\ 4.543 \\ 4.543 \\ 4.461 \\ 4.813 \\ 5.17 \\ 5.699 \\ 4.19 \\ 3.838 \\ 5.699 \\ 5.699 \\ 5.428 \\ 4.19 \\ 4.461 \\ 7.458 \\ 3.272 \\ 3.283 \\ 3.836 \\ 2.604 \\ 8.153 \\ 5.419 \end{vmatrix} kN$$

Hand caluculations for load considerations
Two storey

Total seismic weight at
sill RCC band:
first floor

$$A_{sill1} = \begin{vmatrix} 1.336 \\ 1.344 \\ 1.337 \\ 1.336 \\ 1.422 \\ 1.404 \\ 1.638 \\ 0.999 \\ 0.858 \\ 1.638 \\ 1.638 \\ 1.560 \\ 1.014 \\ 1.092 \\ 1.950 \\ 0.546 \\ 1.336 \\ 1.422 \\ 0.624 \\ 2.974 \\ 1.9498 \end{vmatrix} m^2 \qquad W_{sill\_i1} = A_{sill1} \cdot W_{ave} = \begin{vmatrix} 5.478 \\ 5.51 \\ 5.482 \\ 5.478 \\ 5.83 \\ 5.756 \\ 6.716 \\ 4.096 \\ 3.518 \\ 6.716 \\ 6.716 \\ 6.396 \\ 4.157 \\ 4.477 \\ 7.995 \\ 2.239 \\ 5.478 \\ 5.83 \\ 2.558 \\ 12.193 \\ 7.994 \end{vmatrix} kN$$

Areas and seismic weights of ground floor
GROUND FLOOR - Floor RCC band

$$t_{slab} = 0.2\ m \qquad W_{C20} = 24.99\ \frac{kN}{m^3}$$

$$A_{floorslab} = \begin{vmatrix} 1.24 \\ 1.18 \\ 1.18 \\ 1.24 \\ 1.24 \\ 2.36 \\ 1.24 \\ 1.18 \\ 2.25 \\ 1.18 \\ 1.889 \\ 1.8 \\ 1.8 \\ 1.889 \\ 1.477 \\ 1.35 \\ 0.768 \\ 0.731 \\ 1.462 \\ 1.5 \\ 1.478 \end{vmatrix} m^2 \qquad V_{slab} = A_{floorslab} \cdot t_{slab} \qquad W_{slab\_i} = V_{slab} \cdot W_{C20} = \begin{vmatrix} 6.198 \\ 5.898 \\ 5.898 \\ 6.198 \\ 6.198 \\ 11.795 \\ 6.198 \\ 5.898 \\ 11.246 \\ 5.898 \\ 9.441 \\ 8.996 \\ 8.996 \\ 9.441 \\ 7.382 \\ 6.747 \\ 3.838 \\ 3.654 \\ 7.307 \\ 7.497 \\ 7.387 \end{vmatrix} kN$$

Hand caluculations for load considerations
Two storey

25% of the imposed loads added to the seismic weigth of the building as per 7.3.1:

$$W_{imp\_floor} = A_{floorslab} \cdot 0.25 \cdot q_{d.floor} = \begin{vmatrix} 0.558 \\ 0.531 \\ 0.531 \\ 0.558 \\ 0.558 \\ 1.062 \\ 0.558 \\ 0.531 \\ 1.013 \\ 0.531 \\ 0.85 \\ 0.81 \\ 0.81 \\ 0.85 \\ 0.665 \\ 0.608 \\ 0.346 \\ 0.329 \\ 0.658 \\ 0.675 \\ 0.665 \end{vmatrix} \mathbf{kN}$$

$$A_{floor} = \begin{vmatrix} 0.889 \\ 0.847 \\ 0.847 \\ 0.889 \\ 0.889 \\ 0.808 \\ 0.889 \\ 0.848 \\ 0.601 \\ 0.848 \\ 0.889 \\ 0.847 \\ 0.656 \\ 0.699 \\ 1.059 \\ 0.371 \\ 0.889 \\ 0.884 \\ 0.414 \\ 1.69 \\ 1.06 \end{vmatrix} \mathbf{m^2} \qquad W_{floor\_walls} = A_{floor} \cdot W_{ave} = \begin{vmatrix} 3.645 \\ 3.473 \\ 3.473 \\ 3.645 \\ 3.645 \\ 3.313 \\ 3.645 \\ 3.477 \\ 2.464 \\ 3.477 \\ 3.645 \\ 3.473 \\ 2.69 \\ 2.866 \\ 4.342 \\ 1.521 \\ 3.645 \\ 3.624 \\ 1.697 \\ 6.929 \\ 4.346 \end{vmatrix} \mathbf{kN}$$

Hand caluculations for load considerations
Two storey

$$W_{floor\_i} = W_{floor\_walls} + W_{imp\_floor} + W_{slab\_i} = \begin{vmatrix} 10.4 \\ 9.901 \\ 9.901 \\ 10.4 \\ 10.4 \\ 16.17 \\ 10.4 \\ 9.905 \\ 14.722 \\ 9.905 \\ 13.936 \\ 13.279 \\ 12.496 \\ 13.157 \\ 12.389 \\ 8.876 \\ 7.829 \\ 7.607 \\ 9.662 \\ 15.101 \\ 12.398 \end{vmatrix} \ kN$$

GROUND FLOOR - Lintel RCC band:

$$A_{lintel0} = \begin{vmatrix} 1.088 \\ 1.108 \\ 1.108 \\ 1.088 \\ 1.174 \\ 1.261 \\ 1.390 \\ 1.022 \\ 0.936 \\ 1.390 \\ 1.390 \\ 1.324 \\ 1.022 \\ 1.088 \\ 1.769 \\ 0.758 \\ 0.761 \\ 0.847 \\ 0.579 \\ 1.010 \end{vmatrix} \ m^2 \qquad W_{lintel\_i0} = A_{lintel0} \cdot W_{ave} = \begin{vmatrix} 4.461 \\ 4.543 \\ 4.543 \\ 4.461 \\ 4.813 \\ 5.17 \\ 5.699 \\ 4.19 \\ 3.838 \\ 5.699 \\ 5.699 \\ 5.428 \\ 4.19 \\ 4.461 \\ 7.253 \\ 3.108 \\ 3.12 \\ 3.473 \\ 2.374 \\ 4.141 \end{vmatrix} \ kN$$

Hand caluculations for load considerations
Two storey

GROUND FLOOR - Sill RCC band

$$A_{sill0} = \begin{vmatrix} 1.336 \\ 1.344 \\ 1.337 \\ 1.336 \\ 1.422 \\ 1.404 \\ 1.638 \\ 0.999 \\ 0.858 \\ 1.638 \\ 1.638 \\ 1.560 \\ 1.014 \\ 1.092 \\ 1.950 \\ 0.546 \\ 1.336 \\ 1.422 \\ 0.624 \\ 1.456 \end{vmatrix} \; m^2 \qquad W_{sill\_i0} = A_{sill0} \cdot W_{ave} = \begin{vmatrix} 5.478 \\ 5.51 \\ 5.482 \\ 5.478 \\ 5.83 \\ 5.756 \\ 6.716 \\ 4.096 \\ 3.518 \\ 6.716 \\ 6.716 \\ 6.396 \\ 4.157 \\ 4.477 \\ 7.995 \\ 2.239 \\ 5.478 \\ 5.83 \\ 2.558 \\ 5.97 \end{vmatrix} \; kN$$

Tot weight from SAP for comparison

$G = 608.43$ kN From SAP

Total weight of each "floor":

$$W_{sill} = \left(\sum_{i=1}^{19} W_{sill\_i}(i)\right) = 100.917 \; kN \qquad W_{sill1} = \left(\sum_{i=1}^{20} W_{sill\_i1}(i)\right) = 115.135 \; kN$$

$$W_{lintel} = \left(\sum_{i=1}^{19} W_{lintel\_i}(i)\right) = 86.203 \; kN \qquad W_{lintel1} = \left(\sum_{i=1}^{20} W_{lintel\_i1}(i)\right) = 96.759 \; kN$$

$$W_{floor} = \left(\sum_{i=1}^{20} W_{floor}(i)\right) = 228.437 \; kN \qquad W_{roof} = \left(\sum_{i=1}^{14} W_{roof\_i}(i)\right) = 39.885 \; kN$$

$$W_{tot} = W_{sill0} + W_{lintel0} + W_{floor} + W_{sill1} + W_{lintel1} + W_{roof} = 667.337 \; kN$$

Total seismic weight of building
(compare to SAP value)

Hand caluculations for load considerations
Two storey

Seismic base shear V_base as per 7.5.3

$V_{bx} = A_{hx} \cdot W_{tot} = 100.1$ kN $D_x = 6.15$ m

$V_{by} = A_{hy} \cdot W_{tot} = 100.1$ kN $D_y = 4.95$ m

Distribution of design force as per 7.7.1

$$Q_i = V_B \frac{W_i h_i^2}{\sum_{j=1}^{n} W_j h_j^2}$$

Q_i = Design lateral force at floor i
W_i = Seismic weight of floor i
h_i = Height of floor i measured from base
n = numer of storeys

Proportion of base shear applied on sill band

$$Q_{sill\_x0} = \frac{V_{bx} \cdot W_{sill\_i0} \cdot h_{sill0}^{2}}{W_{sill0} \cdot h_{sill0}^{2} + W_{lintel0} \cdot h_{lintel0}^{2} + W_{floor} \cdot h_{floor}^{2} + W_{sill1} \cdot h_{sill1}^{2} + W_{lintel1} \cdot h_{lintel1}^{2} + W_{roof} \cdot h_{roof}^{2}} = \begin{vmatrix} 0.064 \\ 0.064 \\ 0.064 \\ 0.064 \\ 0.068 \\ 0.067 \\ 0.078 \\ 0.047 \\ 0.041 \\ 0.078 \\ 0.078 \\ 0.074 \\ 0.048 \\ 0.052 \\ 0.093 \\ 0.026 \\ 0.064 \\ 0.068 \\ 0.03 \\ 0.069 \end{vmatrix} \text{ kN}$$

$$Q_{sill\_y0} = \frac{V_{by} \cdot W_{sill\_i0} \cdot h_{sill0}^{2}}{W_{sill0} \cdot h_{sill0}^{2} + W_{lintel0} \cdot h_{lintel0}^{2} + W_{floor} \cdot h_{floor}^{2} + W_{sill1} \cdot h_{sill1}^{2} + W_{lintel1} \cdot h_{lintel1}^{2} + W_{roof} \cdot h_{roof}^{2}} = \begin{vmatrix} 0.064 \\ 0.064 \\ 0.064 \\ 0.064 \\ 0.068 \\ 0.067 \\ 0.078 \\ 0.047 \\ 0.041 \\ 0.078 \\ 0.078 \\ 0.074 \\ 0.048 \\ 0.052 \\ 0.093 \\ 0.026 \\ 0.064 \\ 0.068 \\ 0.03 \\ 0.069 \end{vmatrix} \text{ kN}$$

Hand caluculations for load considerations
Two storey

Proportion of base shear applied on ground floor's lintel band

$$Q_{lintel\_x0} = \frac{V_{bx} \cdot W_{lintel\_i0} \cdot h_{lintel0}^{\ 2}}{W_{sill0} \cdot h_{sill0}^{\ 2} + W_{lintel0} \cdot h_{lintel0}^{\ 2} + W_{floor} \cdot h_{floor}^{\ 2} + W_{sill1} \cdot h_{sill1}^{\ 2} + W_{lintel1} \cdot h_{lintel1}^{\ 2} + W_{roof} \cdot h_{roof}^{\ 2}} = \begin{vmatrix} 0.35 \\ 0.357 \\ 0.357 \\ 0.35 \\ 0.378 \\ 0.406 \\ 0.448 \\ 0.329 \\ 0.301 \\ 0.448 \\ 0.448 \\ 0.426 \\ 0.329 \\ 0.35 \\ 0.57 \\ 0.244 \\ 0.245 \\ 0.273 \\ 0.186 \\ 0.325 \end{vmatrix} \ \mathbf{kN}$$

$$Q_{lintel\_y0} = \frac{V_{by} \cdot W_{lintel\_i0} \cdot h_{lintel0}^{\ 2}}{W_{sill0} \cdot h_{sill0}^{\ 2} + W_{lintel0} \cdot h_{lintel0}^{\ 2} + W_{floor} \cdot h_{floor}^{\ 2} + W_{sill1} \cdot h_{sill1}^{\ 2} + W_{lintel1} \cdot h_{lintel1}^{\ 2} + W_{roof} \cdot h_{roof}^{\ 2}} = \begin{vmatrix} 0.35 \\ 0.357 \\ 0.357 \\ 0.35 \\ 0.378 \\ 0.406 \\ 0.448 \\ 0.329 \\ 0.301 \\ 0.448 \\ 0.448 \\ 0.426 \\ 0.329 \\ 0.35 \\ 0.57 \\ 0.244 \\ 0.245 \\ 0.273 \\ 0.186 \\ 0.325 \end{vmatrix} \ \mathbf{kN}$$

Hand caluculations for load considerations
Two storey

Proportion of base shear applied on floor band

$$Q_{floor\_x} = \frac{V_{bx} \bullet W_{floor\_i} \bullet h_{floor}^{\ 2}}{W_{sill0} \bullet h_{sill0}^{\ 2} + W_{lintel0} \bullet h_{lintel0}^{\ 2} + W_{floor} \bullet h_{floor}^{\ 2} + W_{sill1} \bullet h_{sill1}^{\ 2} + W_{lintel1} \bullet h_{lintel1}^{\ 2} + W_{roof} \bullet h_{roof}^{\ 2}} = \begin{vmatrix} 0.986 \\ 0.938 \\ 0.938 \\ 0.986 \\ 0.986 \\ 1.532 \\ 0.986 \\ 0.939 \\ 1.395 \\ 0.939 \\ 1.321 \\ 1.258 \\ 1.184 \\ 1.247 \\ 1.174 \\ 0.841 \\ 0.742 \\ 0.721 \\ 0.916 \\ 1.431 \\ 1.175 \end{vmatrix} \ \mathbf{kN}$$

$$Q_{floor\_\_y} = \frac{V_{by} \bullet W_{floor\_i} \bullet h_{floor}^{\ 2}}{W_{sill0} \bullet h_{sill0}^{\ 2} + W_{lintel0} \bullet h_{lintel0}^{\ 2} + W_{floor} \bullet h_{floor}^{\ 2} + W_{sill1} \bullet h_{sill1}^{\ 2} + W_{lintel1} \bullet h_{lintel1}^{\ 2} + W_{roof} \bullet h_{roof}^{\ 2}} = \begin{vmatrix} 0.986 \\ 0.938 \\ 0.938 \\ 0.986 \\ 0.986 \\ 1.532 \\ 0.986 \\ 0.939 \\ 1.395 \\ 0.939 \\ 1.321 \\ 1.258 \\ 1.184 \\ 1.247 \\ 1.174 \\ 0.841 \\ 0.742 \\ 0.721 \\ 0.916 \\ 1.431 \\ 1.175 \end{vmatrix} \ \mathbf{kN}$$

Hand caluculations for load considerations
Two storey

Proportion of base shear applied on first floor's sill band

$$Q_{sill\_x1} = \frac{V_{bx} \cdot W_{sill\_i1} \cdot {h_{sill1}}^2}{W_{sill0} \cdot {h_{sill0}}^2 + W_{lintel0} \cdot {h_{lintel0}}^2 + W_{floor} \cdot {h_{floor}}^2 + W_{sill1} \cdot {h_{sill1}}^2 + W_{lintel1} \cdot {h_{lintel1}}^2 + W_{roof} \cdot {h_{roof}}^2} = \begin{vmatrix} 0.946 \\ 0.951 \\ 0.946 \\ 0.946 \\ 1.007 \\ 0.994 \\ 1.159 \\ 0.707 \\ 0.607 \\ 1.159 \\ 1.159 \\ 1.104 \\ 0.718 \\ 0.773 \\ 1.38 \\ 0.386 \\ 0.946 \\ 1.007 \\ 0.442 \\ 2.105 \\ 1.38 \end{vmatrix} \text{ kN}$$

$$Q_{sill\_\_y1} = \frac{V_{by} \cdot W_{sill\_i1} \cdot {h_{sill1}}^2}{W_{sill0} \cdot {h_{sill0}}^2 + W_{lintel0} \cdot {h_{lintel0}}^2 + W_{floor} \cdot {h_{floor}}^2 + W_{sill1} \cdot {h_{sill1}}^2 + W_{lintel1} \cdot {h_{lintel1}}^2 + W_{roof} \cdot {h_{roof}}^2} = \begin{vmatrix} 0.946 \\ 0.951 \\ 0.946 \\ 0.946 \\ 1.007 \\ 0.994 \\ 1.159 \\ 0.707 \\ 0.607 \\ 1.159 \\ 1.159 \\ 1.104 \\ 0.718 \\ 0.773 \\ 1.38 \\ 0.386 \\ 0.946 \\ 1.007 \\ 0.442 \\ 2.105 \\ 1.38 \end{vmatrix} \text{ kN}$$

Hand caluculations for load considerations
Two storey

Proportion of base shear applied on first floor's lintel band

$$Q_{lintel\_x1} = \frac{V_{bx} \cdot W_{lintel\_i1} \cdot h_{lintel1}^{\ 2}}{W_{sill0} \cdot h_{sill0}^{\ 2} + W_{lintel0} \cdot h_{lintel0}^{\ 2} + W_{floor} \cdot h_{floor}^{\ 2} + W_{sill1} \cdot h_{sill1}^{\ 2} + W_{lintel1} \cdot h_{lintel1}^{\ 2} + W_{roof} \cdot h_{roof}^{\ 2}} = \begin{vmatrix} 1.535 \\ 1.564 \\ 1.564 \\ 1.535 \\ 1.657 \\ 1.78 \\ 1.962 \\ 1.442 \\ 1.321 \\ 1.962 \\ 1.962 \\ 1.868 \\ 1.442 \\ 1.535 \\ 2.567 \\ 1.126 \\ 1.13 \\ 1.321 \\ 0.896 \\ 2.806 \\ 1.865 \end{vmatrix} \ kN$$

$$Q_{lintel\_\_y1} = \frac{V_{by} \cdot W_{lintel\_i1} \cdot h_{lintel1}^{\ 2}}{W_{sill0} \cdot h_{sill0}^{\ 2} + W_{lintel0} \cdot h_{lintel0}^{\ 2} + W_{floor} \cdot h_{floor}^{\ 2} + W_{sill1} \cdot h_{sill1}^{\ 2} + W_{lintel1} \cdot h_{lintel1}^{\ 2} + W_{roof} \cdot h_{roof}^{\ 2}} = \begin{vmatrix} 1.535 \\ 1.564 \\ 1.564 \\ 1.535 \\ 1.657 \\ 1.78 \\ 1.962 \\ 1.442 \\ 1.321 \\ 1.962 \\ 1.962 \\ 1.868 \\ 1.442 \\ 1.535 \\ 2.567 \\ 1.126 \\ 1.13 \\ 1.321 \\ 0.896 \\ 2.806 \\ 1.865 \end{vmatrix} \ kN$$

Hand caluculations for load considerations
Two storey

Proportion of base shear applied on roof band

$$Q_{roof\_x} = \frac{V_{bx} \cdot W_{roof\_i} \cdot h_{roof}^{\ 2}}{W_{sill0} \cdot h_{sill0}^{\ 2} + W_{lintel0} \cdot h_{lintel0}^{\ 2} + W_{floor} \cdot h_{floor}^{\ 2} + W_{sill1} \cdot h_{sill1}^{\ 2} + W_{lintel1} \cdot h_{lintel1}^{\ 2} + W_{roof} \cdot h_{roof}^{\ 2}} = \begin{vmatrix} 0.963 \\ 0.92 \\ 0.92 \\ 0.963 \\ 1.595 \\ 1.648 \\ 1.595 \\ 1.551 \\ 1.606 \\ 1.594 \\ 0.987 \\ 0.946 \\ 0.946 \\ 0.989 \\ 1.07 \end{vmatrix} \ \mathbf{kN}$$

$$Q_{roof\_\_y} = \frac{V_{by} \cdot W_{roof\_i} \cdot h_{roof}^{\ 2}}{W_{sill0} \cdot h_{sill0}^{\ 2} + W_{lintel0} \cdot h_{lintel0}^{\ 2} + W_{floor} \cdot h_{floor}^{\ 2} + W_{sill1} \cdot h_{sill1}^{\ 2} + W_{lintel1} \cdot h_{lintel1}^{\ 2} + W_{roof} \cdot h_{roof}^{\ 2}} = \begin{vmatrix} 0.963 \\ 0.92 \\ 0.92 \\ 0.963 \\ 1.595 \\ 1.648 \\ 1.595 \\ 1.551 \\ 1.606 \\ 1.594 \\ 0.987 \\ 0.946 \\ 0.946 \\ 0.989 \\ 1.07 \end{vmatrix} \ \mathbf{kN}$$

Hand caluculations for load considerations
Two storey

$Q_{sill0} = \left(\sum_{i=1}^{19} Q_{sill\_x0}(i)\right) = 1.17$ **kN**

$Q_{lintel0} = \left(\sum_{i=1}^{19} Q_{lintel\_x0}(i)\right) = 6.769$ **kN**

$Q_{floor} = \left(\sum_{i=1}^{20} Q_{floor\_x}(i)\right) = 21.648$ **kN**

$Q_{sill1} = \left(\sum_{i=1}^{20} Q_{sill\_x1}(i)\right) = 19.878$ **kN**

$Q_{lintel1} = \left(\sum_{i=1}^{20} Q_{lintel\_x1}(i)\right) = 33.305$ **kN**

$Q_{roof} = \left(\sum_{i=1}^{14} Q_{roof\_x}(i)\right) = 17.33$ **kN**

$Q_{tot} = Q_{sill0} + Q_{sill1} + Q_{lintel0} + Q_{lintel1} + Q_{floor} + Q_{roof} = 100.1$ **kN**

$V_{bx} = 100.1$ **kN** OK!

Hand caluculations for load considerations
Single storey - 2 rooms + kitchen (X5 - Roof band made all around the building)

Hand calculations for load considerations - X5

Geometrical inputs:

$l = 6.15\ m$	Length of building	$t_{brick} = 0.15\ m$
$h = 3.472\ m$	Heigth of building	
$w = 4.95\ m$	Width of building	
$\alpha = 17\ °$	Angle of roof slope	
$alpha = 17$	Angle of roos for imposed loads	
$D_x = 6.15\ m$	Dimensions of the building along x -axis	
$D_y = 4.95\ m$	Dimensions of the building along y -axis	

Material properties of CSEB:

$f_{ck\_CSEB} = 5\ \frac{N}{mm^2}$ $E_{CSEB} = 700\ MPa$

$\rho_{CSEB} = 19\ \frac{kN}{m^3}$ $v_{CSEB} = 0.15$

$f_{y\_CSEB} = 500\ \frac{N}{mm^2}$ $W_{CSEBm} = 19.0\ \frac{kN}{m^3}$

Material properties of structural steel (roof):

$\rho_{steel} = 78.5\ \frac{kN}{m^3}$ $E_{steel} = 200000\ MPa$

IS 800 : 2007
Fe 250

$f_{y\_strc\_steel} = 250\ \frac{N}{mm^2}$ $v_{steel} = 0.3$

Material poperties of reinforcing steel:

$f_{y\_steel} = 500\ \frac{N}{mm^2}$

IS 1786 : 2008
Fe 500

Material poperties of concret

$f_{ck\_concret} = 20\ \frac{N}{mm^2}$

Ordinary concrete M 20
IS 456:2000

Material poperties of mortar

$f_{ck\_mortar} = 2.5\ \frac{N}{mm^2}$

EKS 10, Table H -2

Hand caluculations for load considerations
Single storey - 2 rooms + kitchen (X5 - Roof band made all around the building)

LOADS:

- Dead loads: as per IS 875 : 1984 Part I
- Live loads: as per IS 875 : 1984 Part II
- Wind loads: as per NBC 104 : 1994 and IS 875 : 1984 Part III
- Earthquake load: as per IS 1893 : 2002 (and NBC 105:1994)

Dead load:

$G = 232.071\ \mathrm{kN}$ From SAP without false ceiling

Imposed load:

Table 2: Imposed loads on various types of roofs
ii) sloping roof with slope greater than 10 degreees

$$q_r = 0.75\ \frac{\mathrm{kN}}{\mathrm{m}^2} - \left(0.02\ \frac{\mathrm{kN}}{\mathrm{m}^2}\left(alpha - 10\right)\right) = 0.75\ \frac{\mathrm{kN}}{\mathrm{m}^2}$$

Divide the imposed load over the roof rafters

$$q_{r1} = q_r \cdot 31.673\,\mathrm{m}^2 = 23.755\ \mathrm{kN}$$

$$l_r = 8 \cdot 6.15\,\mathrm{m} = 49.2\ \mathrm{m}$$

$$q_{rdist} = \frac{q_{r1}}{l_r} = 0.483\ \frac{\mathrm{kN}}{\mathrm{m}}$$

Hand caluculations for load considerations
Single storey - 2 rooms + kitchen (X5 - Roof band made all around the building)

<u>Wind loads:</u>

$V_b = 50 \, \frac{m}{s}$ Basic wind speed

$k_1 = 1$ Probability factor (risk coeff.) from Table 1

$k_2 = 1.05$ Terrain, height and structure size factor from Table 2 Category 1 Class A?

$k_3 = 1$ Topography factor

$V_z = V_b \cdot k_1 \cdot k_2 \cdot k_3 = 52.5 \, \frac{m}{}$ Design wind speed

$p_z = 0.6 \, \frac{Pa \cdot s^2}{m^2} \cdot V_z^{\,2} = (1.654 \cdot 10^3) \, \frac{N}{m^2}$ Design wind pressure, accor. to 5.4 " the coeff. 0.6 SI units depends on a number of factors..."

$\frac{h}{w} = 0.701$

Wind angle tetha = 0

$\frac{l}{w} = 1.242$

<u>WALLS:</u>
Decide external pressure coefficients (C_pe) from ratios (Table 4 - Rectangular clad buildings):

$\frac{h}{w} \le \frac{1}{2} = 0$

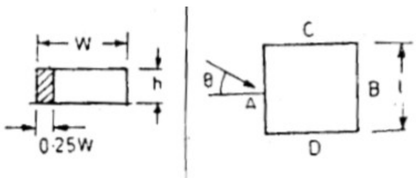
$1 < \frac{l}{w} \le \frac{3}{2} = 1$

These coefficent used from second h/w condition and first l/w condition

$\frac{1}{2} < \frac{h}{w} \le \frac{3}{2} = 1$

$\frac{3}{2} < \frac{l}{w} < 4 = 0$

$\frac{3}{2} < \frac{h}{w} \le 6 = 0$



$C_{pe\_A} = 0.7$ $C_{pe\_C} = -0.6$

$C_{pe\_B} = -0.25$ $C_{pe\_D} = -0.6$

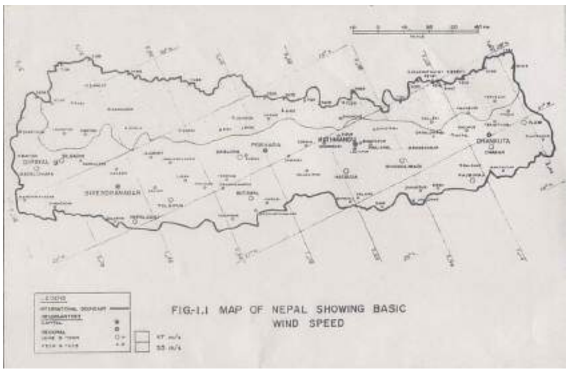


FIG-1.1 MAP OF NEPAL SHOWING BASIC WIND SPEED

W

$C_{pe\_W} = -1.1$

Hand caluculations for load considerations
Single storey - 2 rooms + kitchen (X5   - Roof band made all around the building)

ROOF:

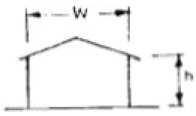
Ratios for C_pe on the roof (Table 5)

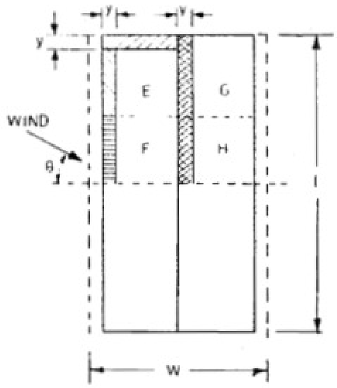
$$\frac{h}{w} \le \frac{1}{2} = 0$$

$$\frac{1}{2} < \frac{h}{w} \le \frac{3}{2} = 1$$

$$\frac{3}{2} < \frac{h}{w} \le 6 = 0$$

$x = 10$





KEY PLAN

$y = h$ or 0·15 w,
whichever is the lesser.

EF Tetha = 0 degrees

$C_{pe\_EF0} = -1.1$

GH Tetha = 0 degrees

C_{pe_GH0} -0.6

EG Tetha = 90 degrees

C_{pe_EG90} -0.8

FH Tetha = 90 degrees

C_{pe_FH90} -0.6

Hand caluculations for load considerations
Single storey - 2 rooms + kitchen (X5 - Roof band made all around the building)

walls:

$$C_{pe\_max\_wx} = \max\left(C_{pe\_A}, C_{pe\_B}, C_{pe\_W}\right) = 0.7$$

$$C_{pe\_min\_wx} = \min\left(C_{pe\_A}, C_{pe\_B}, C_{pe\_W}\right) = -1.1$$

$$C_{pe\_max\_wy} = \max\left(C_{pe\_C}, C_{pe\_D}, C_{pe\_W}\right) = -0.6$$

$$C_{pe\_min\_wy} = \min\left(C_{pe\_C}, C_{pe\_D}, C_{pe\_W}\right) = -1.1$$

Roof:

$$C_{pe\_max\_rx} = \max\left(C_{pe\_EG90}, C_{pe\_FH90}\right) = -0.6$$

$$C_{pe\_min\_rx} = \min\left(C_{pe\_EG90}, C_{pe\_FH90}\right) = -0.8$$

$$C_{pe\_max\_ry} = \max\left(C_{pe\_EF0}, C_{pe\_GH0}\right) = -0.6$$

$$C_{pe\_min\_ry} = \min\left(C_{pe\_EF0}, C_{pe\_GH0}\right) = -1.1$$

Internal pressure coefficents

$$C_{pi\_neg} = -0.2 \qquad C_{pi\_pos} = 0.2$$

<u>Design load in x direction for the walls:</u>

$$F1x = \left(C_{pe\_min\_wx} - C_{pi\_neg}\right) \cdot p_z = -1.488\ \frac{kN}{m^2}$$

$$F2x = \left(C_{pe\_min\_wx} - C_{pi\_pos}\right) \cdot p_z = -2.15\ \frac{kN}{m^2}$$

$$F3x = \left(C_{pe\_max\_wx} - C_{pi\_neg}\right) \cdot p_z = 1.488\ \frac{kN}{m^2}$$

$$F4x = \left(C_{pe\_max\_wx} - C_{pi\_pos}\right) \cdot p_z = 0.827\ \frac{kN}{m^2}$$

$$F_{wx} = F2x = -2.15\ \frac{kN}{m^2}$$

<u>Design load in y direction for the walls:</u>

$$F1y = \left(C_{pe\_min\_wy} - C_{pi\_neg}\right) \cdot p_z = -1.488\ \frac{kN}{m^2}$$

$$F2y = \left(C_{pe\_min\_wy} - C_{pi\_pos}\right) \cdot p_z = -2.15\ \frac{kN}{m^2}$$

Hand caluculations for load considerations
Single storey - 2 rooms + kitchen (X5 - Roof band made all around the building)

$$F3y = (C_{pe\_max\_wy} - C_{pi\_neg}) \cdot p_z = -0.662 \ \frac{kN}{m^2}$$

$$F4y = (C_{pe\_max\_wy} - C_{pi\_pos}) \cdot p_z = -1.323 \ \frac{kN}{m^2}$$

$$F_{wy} = F2y = -2.15 \ \frac{kN}{m^2}$$

Design load in x direction for the roof:

$$F1x = (C_{pe\_min\_rx} - C_{pi\_neg}) \cdot p_z = -0.992 \ \frac{kN}{m^2}$$

$$F2x = (C_{pe\_min\_rx} - C_{pi\_pos}) \cdot p_z = -1.654 \ \frac{kN}{m^2}$$

$$F3x = (C_{pe\_max\_rx} - C_{pi\_neg}) \cdot p_z = -0.662 \ \frac{kN}{m^2}$$

$$F4x = (C_{pe\_max\_rx} - C_{pi\_pos}) \cdot p_z = -1.323 \ \frac{kN}{m^2}$$

$$F_{rx} = F2x = -1.654 \ \frac{kN}{m^2}$$

Design load in y direction for the roof:

$$F1y = (C_{pe\_min\_ry} - C_{pi\_neg}) \cdot p_z = -1.488 \ \frac{kN}{m^2}$$

$$F2y = (C_{pe\_min\_ry} - C_{pi\_pos}) \cdot p_z = -2.15 \ \frac{kN}{m^2}$$

$$F3y = (C_{pe\_max\_ry} - C_{pi\_neg}) \cdot p_z = -0.662 \ \frac{kN}{m^2}$$

$$F4y = (C_{pe\_max\_ry} - C_{pi\_pos}) \cdot p_z = -1.323 \ \frac{kN}{m^2}$$

$$F_{ry} = F2y = -2.15 \ \frac{kN}{m^2}$$

WL to apply on walls:

$$WL_{wall} = \left| \begin{matrix} F_{wx} \\ F_{wy} \end{matrix} \right| = \left| \begin{matrix} -2.15 \\ -2.15 \end{matrix} \right| \ \frac{kN}{m^2}$$

WL to apply on roof:

$$WL_{roof} = \left| \begin{matrix} F_{rx} \\ F_{ry} \end{matrix} \right| = \left| \begin{matrix} -1.654 \\ -2.15 \end{matrix} \right| \ \frac{kN}{m^2}$$

Divide the WL on roof on the rafters:

$$q_{rwx} = F_{rx} \cdot 31.673 \ m^2 = -52.379 \ kN$$

$$l_r = 8 \cdot 6.15 \ m = 49.2 \ m$$

$$q_{rdistx} = \frac{q_{rwx}}{l_r} = -1.065 \ \frac{kN}{m}$$

$$q_{rwy} = F_{ry} \cdot 31.673 \ m^2 = -68.093 \ kN$$

$$l_r = 8 \cdot 6.15 \ m = 49.2 \ m$$

$$q_{rdisty} = \frac{q_{rwy}}{l_r} = -1.384 \ \frac{kN}{m}$$

Hand caluculations for load considerations
Single storey - 2 rooms + kitchen (X5 - Roof band made all around the building)

Earthquake load

Classification of building:
- General provisions and buildings

Zone factor Z Table 2 (Clause 6.4.2)

$Z = 0.36$ Very severe

Importance factor I Table 6

$I = 1.0$

Response reduction factor R as per Table 7 and clause 6.4.2)

$R = 3$ Category v) b) Reinf h RC bands
Should be v) c) Reinf h RC + vertical? = 3.0, Resilient 2.5

$\frac{I}{R} < 1 = 1$

Time period of the building as per 7.6.2

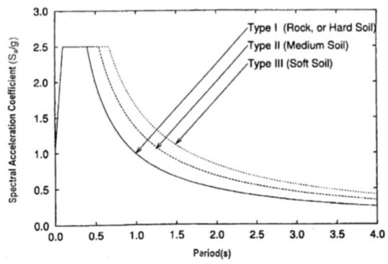
$T_x = 0.09 \frac{s}{m^{0.5}} \frac{h}{D_x^{0.5}} = 0.126\ s$ Along x

$T_y = 0.09 \frac{s}{m^{0.5}} \frac{h}{D_y^{0.5}} = 0.14\ s$ Along y

Soil profile:
Assumed to be sub soil category III (soft soil site)

Average response acceleration coefficient S a/g
(from soil type and time period T)

Along x	Along y
$0\ s \le T_x \le 0.1\ s = 0$	$0\ s \le T_y \le 0.1\ s = 0$
$0.1\ s \le T_x \le 0.67\ s = 1$	$0.1\ s \le T_y \le 0.67\ s = 1$
$0.67\ s \le T_x \le 4\ s = 0$	$0.67\ s \le T_y \le 4\ s = 0$



$S_{ax} = 2.5$ $g_x = 1$ $S_{ay} = 2.5$ $g_y = 1$

$\frac{S_{ax}}{g_x} = 2.5$ $\frac{S_{ay}}{g_y} = 2.5$

Hand caluculations for load considerations
Single storey - 2 rooms + kitchen (X5 - Roof band made all around the building)

The design horizontal seismic coefficient A_h

$$A_{hx} = \frac{Z \cdot I \cdot S_{ax}}{2 \cdot R \cdot g_x} = 0.15$$ if T<= 0.1 s -> A_h=Z/2

$$A_{hy} = \frac{Z \cdot I \cdot S_{ay}}{2 \cdot R \cdot g_y} = 0.15$$

Seismic weight of the building W as per 7.4.2

" *The seismic weight of each floor is its full dead load plus appropriate amount of imposed load, as specified in 7.3.1 and 7.3.2. While computing the seismic weight of each floor, the weight of columns and walls in any storey shall be equally distributed to the floors above and below the storey.* "

Treat each of roof, lintel and sill RCC bands as one storey and apply earthquake load in proportion to tributary area and weight of each "floor":

$W_{ave} = 4.1 \ \frac{kN}{m^2}$ Adjusted estimated value to get closer to total seaismic weight from SAP

$h_{sill} = 0.780 \ m$ $h_{lintel} = 2.03 \ m$ $h_{roof} = 2.545 \ m$

Rough estimation of the roof weight :

$$W_{rafter} = (76.97 \cdot 6.6 \cdot 10^{-4} \cdot (1.65 + 3.5)) \cdot 7 \ kN = 1.831 \cdot 10^3 \ N$$

$$W_{purlin} = (76.97 \cdot 4.45 \cdot 10^{-4} \cdot (6.15)) \cdot 6 \ kN = 1.264 \cdot 10^3 \ N$$

$$W_{ridge} = 76.97 \cdot 1.1 \cdot 10^{-3} \cdot 6.15 \ kN = 520.702 \ N$$

$$W_{r.p.r} = W_{rafter} + W_{purlin} + W_{ridge} = 3.616 \cdot 10^3 \ N$$

$$A_{roof} = 1.65 \ m \cdot 6.15 \ m + 3.5 \ m \cdot 6.15 \ m = 31.673 \ m^2$$

$$W_{rafters} = \frac{W_{r.p.r}}{A_{roof}} = 0.114 \ \frac{kN}{m^2}$$

Weight of CGI and roof rafters:

$$W_{CGI} = 0.043164 \ \frac{kN}{m^2}$$

$$W_{CGI.rafters} = W_{CGI} + W_{rafters} = 0.157 \ \frac{kN}{m^2}$$

$$W_{CGI.rafters} \cdot A_{roof} = 4.983 \ kN$$

New roof area from SAP:

$A_{gabble} = 1.3923$ $A_{gabbleold} = 0.6423$

$$ratio_{gabble} = \frac{A_{gabble}}{A_{gabbleold}} = 2.168$$

Hand caluculations for load considerations
Single storey - 2 rooms + kitchen (X5 - Roof band made all around the building)

Roof RCC band:

$$A_{roof} = ratio_{gabble} \cdot \begin{vmatrix} 0.484 \\ 0.461 \\ 0.461 \\ 0.484 \\ 0.839 \\ 0.839 \\ 0.839 \\ 0.815 \\ 0.815 \\ 0.839 \\ 0.484 \\ 0.461 \\ 0.461 \\ 0.484 \\ 0.535 \end{vmatrix} m^2 \qquad W_{roof\_i1} = A_{roof} \cdot W_{ave} \begin{vmatrix} 4.302 \\ 4.097 \\ 4.097 \\ 4.302 \\ 7.457 \\ 7.457 \\ 7.457 \\ 7.243 \\ 7.243 \\ 7.457 \\ 4.302 \\ 4.097 \\ 4.097 \\ 4.302 \\ 4.755 \end{vmatrix} kN$$

Areas and weights from rafters added to roof RCC band:

$$A_{only\_roof} = \begin{vmatrix} 1.479 \\ 1.443 \\ 1.449 \\ 1.470 \\ 1.470 \\ 2.250 \\ 1.470 \\ 1.447 \\ 2.250 \\ 1.460 \\ 1.822 \\ 1.823 \\ 1.823 \\ 1.849 \\ 1.714 \end{vmatrix} m^2 \qquad W_{only\_roof} = A_{only\_roof} \cdot W_{CGI.rafters} = \begin{vmatrix} 0.233 \\ 0.227 \\ 0.228 \\ 0.231 \\ 0.231 \\ 0.354 \\ 0.231 \\ 0.228 \\ 0.354 \\ 0.23 \\ 0.287 \\ 0.287 \\ 0.287 \\ 0.291 \\ 0.27 \end{vmatrix} kN$$

Hand caluculations for load considerations
Single storey - 2 rooms + kitchen (X5 - Roof band made all around the building)

Total seismic weight at roof RCC band

$$W_{roof\_i} = W_{roof\_i1} + W_{only\_roof} = \begin{vmatrix} 4.534 \\ 4.324 \\ 4.325 \\ 4.533 \\ 7.688 \\ 7.811 \\ 7.688 \\ 7.471 \\ 7.597 \\ 7.686 \\ 4.588 \\ 4.384 \\ 4.384 \\ 4.592 \\ 5.024 \end{vmatrix} \, kN$$

Lintel RCC band:

$$A_{lintel} = \begin{vmatrix} 1.088 \\ 1.108 \\ 1.108 \\ 1.088 \\ 1.174 \\ 1.261 \\ 1.390 \\ 1.022 \\ 0.936 \\ 1.390 \\ 1.390 \\ 1.324 \\ 1.022 \\ 1.088 \\ 1.769 \\ 0.758 \\ 0.761 \\ 0.847 \\ 0.579 \\ 1.010 \end{vmatrix} \, m^2 \qquad W_{lintel\_i1} = A_{lintel} \cdot W_{ave} = \begin{vmatrix} 4.461 \\ 4.543 \\ 4.543 \\ 4.461 \\ 4.813 \\ 5.17 \\ 5.699 \\ 4.19 \\ 3.838 \\ 5.699 \\ 5.699 \\ 5.428 \\ 4.19 \\ 4.461 \\ 7.253 \\ 3.108 \\ 3.12 \\ 3.473 \\ 2.374 \\ 4.141 \end{vmatrix} \, kN$$

Areas and
weights from
rafters added to
lintel RCC band:

$$A_{only\_lintel} = \begin{vmatrix} 0 \\ 0 \\ 0 \\ 0 \\ 0 \\ 0 \\ 0 \\ 0 \\ \vdots \end{vmatrix} \, m^2 \qquad W_{only\_lintel} = A_{only\_lintel} \cdot W_{CGI.rafters} = \begin{array}{r} 0 \\ 1 \\ 2 \\ 3 \\ 4 \\ 5 \\ \vdots \\ 19 \end{array} \begin{vmatrix} 0 \\ 0 \\ 0 \\ 0 \\ 0 \\ 0 \\ \vdots \\ \; \end{vmatrix} \, kN$$

Hand caluculations for load considerations
Single storey - 2 rooms + kitchen (X5 - Roof band made all around the building)

Total seismic weight at lintel RCC band:

$$W_{lintel\_i} = W_{lintel\_i1} + W_{only\_lintel} = \begin{vmatrix} 4.461 \\ 4.543 \\ 4.543 \\ 4.461 \\ 4.813 \\ 5.17 \\ 5.699 \\ 4.19 \\ 3.838 \\ 5.699 \\ 5.699 \\ 5.428 \\ 4.19 \\ 4.461 \\ 7.458 \\ 3.272 \\ 3.283 \\ 3.836 \\ 2.594 \\ 4.349 \end{vmatrix} \ kN$$

Total seismic weight at sill RCC band:

$$A_{sill} = \begin{vmatrix} 1.336 \\ 1.344 \\ 1.337 \\ 1.336 \\ 1.422 \\ 1.404 \\ 1.638 \\ 0.999 \\ 0.858 \\ 1.638 \\ 1.638 \\ 1.560 \\ 1.014 \\ 1.092 \\ 1.950 \\ 0.546 \\ 1.336 \\ 1.422 \\ 0.624 \\ 1.456 \end{vmatrix} \ m^2 \qquad W_{sill\_i} = A_{sill} \cdot W_{ave} = \begin{vmatrix} 5.478 \\ 5.51 \\ 5.482 \\ 5.478 \\ 5.83 \\ 5.756 \\ 6.716 \\ 4.096 \\ 3.518 \\ 6.716 \\ 6.716 \\ 6.396 \\ 4.157 \\ 4.477 \\ 7.995 \\ 2.239 \\ 5.478 \\ 5.83 \\ 2.558 \\ 5.97 \end{vmatrix} \ kN$$

Hand caluculations for load considerations
Single storey - 2 rooms + kitchen (X5 - Roof band made all around the building)

Tot weight from SAP for comparison
$G = 232.071$ kN From SAP

Total weight of each "floor":

$$W_{sill} = \left(\sum_{i=1}^{19} W_{sill\_i} (i) \right) = 100.917 \text{ kN}$$

$$W_{lintel} = \left(\sum_{i=1}^{19} W_{lintel\_i} (i) \right) = 87.526 \text{ kN}$$

$$W_{roof} = \left(\sum_{i=1}^{14} W_{roof\_i} (i) \right) = 82.096 \text{ kN}$$

$$W_{tot} = W_{sill} + W_{lintel} + W_{roof} = 270.539 \text{ kN}$$

Total seismic weight of building (compare to SAP value)

$$Part_{sill} = \frac{W_{sill}}{W_{tot}} = 0.373 \qquad Part_{lintel} = \frac{W_{lintel}}{W_{tot}} = 0.324 \qquad Part_{roof} = \frac{W_{roof}}{W_{tot}} = 0.303$$

$$Part_{tot} = Part_{sill} + Part_{lintel} + Part_{roof} = 1$$

Seismic base shear V_base as per 7.5.3

$$V_{bx} = A_{hx} \cdot W_{tot} = 40.581 \text{ kN} \qquad D_x = 6.15 \text{ m}$$

$$V_{by} = A_{hy} \cdot W_{tot} = 40.581 \text{ kN} \qquad D_y = 4.95 \text{ m}$$

Distribution of design force as per 7.7.1

$$Q_i = V_B \frac{W_i h_i^2}{\sum_{j=1}^{n} W_j h_j^2}$$

Q_i = Design lateral force at floor i
W_i = Seismic weight of floor i
h_i = Height of floor i measured from base
n = numer of storeys

Hand caluculations for load considerations
Single storey - 2 rooms + kitchen (X5 - Roof band made all around the building)

Proportion of base shear applied on sill band

$$Q_{sill\_x} = \frac{V_{bx} \cdot W_{sill\_i} \cdot h_{sill}^{\ 2}}{W_{sill} \cdot h_{sill}^{\ 2} + W_{lintel} \cdot h_{lintel}^{\ 2} + W_{roof} \cdot h_{roof}^{\ 2}} = \begin{vmatrix} 0.142 \\ 0.143 \\ 0.142 \\ 0.142 \\ 0.151 \\ 0.149 \\ 0.174 \\ 0.106 \\ 0.091 \\ 0.174 \\ 0.174 \\ 0.166 \\ 0.108 \\ 0.116 \\ 0.207 \\ 0.058 \\ 0.142 \\ 0.151 \\ 0.066 \\ 0.155 \end{vmatrix} \ \mathbf{kN}$$

$$Q_{sill\_y} = \frac{V_{by} \cdot W_{sill\_i} \cdot h_{sill}^{\ 2}}{W_{sill} \cdot h_{sill}^{\ 2} + W_{lintel} \cdot h_{lintel}^{\ 2} + W_{roof} \cdot h_{roof}^{\ 2}} = \begin{vmatrix} 0.142 \\ 0.143 \\ 0.142 \\ 0.142 \\ 0.151 \\ 0.149 \\ 0.174 \\ 0.106 \\ 0.091 \\ 0.174 \\ 0.174 \\ 0.166 \\ 0.108 \\ 0.116 \\ 0.207 \\ 0.058 \\ 0.142 \\ 0.151 \\ 0.066 \\ 0.155 \end{vmatrix} \ \mathbf{kN}$$

Hand caluculations for load considerations
Single storey - 2 rooms + kitchen (X5 - Roof band made all around the building)

Proportion of base shear applied on lintel band

$$Q_{lintel\_x} = \frac{V_{bx} \cdot W_{lintel\_i} \cdot h_{lintel}^{2}}{W_{sill} \cdot h_{sill}^{2} + W_{lintel} \cdot h_{lintel}^{2} + W_{roof} \cdot h_{roof}^{2}} = \begin{vmatrix} 0.782 \\ 0.796 \\ 0.796 \\ 0.782 \\ 0.844 \\ 0.906 \\ 0.999 \\ 0.735 \\ 0.673 \\ 0.999 \\ 0.999 \\ 0.952 \\ 0.735 \\ 0.782 \\ 1.308 \\ 0.574 \\ 0.576 \\ 0.673 \\ 0.455 \\ 0.762 \end{vmatrix} \ \mathbf{kN}$$

$$Q_{lintel\_y} = \frac{V_{by} \ W_{lintel\_i} \cdot h_{lintel}^{2}}{W_{sill} \cdot h_{sill}^{2} + W_{lintel} \cdot h_{lintel}^{2} + W_{roof} \cdot h_{roof}^{2}} = \begin{vmatrix} 0.782 \\ 0.796 \\ 0.796 \\ 0.782 \\ 0.844 \\ 0.906 \\ 0.999 \\ 0.735 \\ 0.673 \\ 0.999 \\ 0.999 \\ 0.952 \\ 0.735 \\ 0.782 \\ 1.308 \\ 0.574 \\ 0.576 \\ 0.673 \\ 0.455 \\ 0.762 \end{vmatrix} \ \mathbf{kN}$$

Hand caluculations for load considerations
Single storey - 2 rooms + kitchen (X5 - Roof band made all around the building)

Proportion of base shear applied on roof band

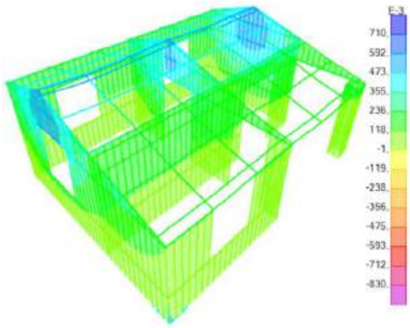
$$Q_{roof\_x} = \frac{V_{bx} \cdot W_{roof\_i} \cdot h_{roof}^{2}}{W_{sill} \cdot h_{sill}^{2} + W_{lintel} \cdot h_{lintel}^{2} + W_{roof} \cdot h_{roof}^{2}} = \begin{vmatrix} 1.249 \\ 1.192 \\ 1.192 \\ 1.249 \\ 2.119 \\ 2.152 \\ 2.119 \\ 2.059 \\ 2.094 \\ 2.118 \\ 1.264 \\ 1.208 \\ 1.208 \\ 1.266 \\ 1.385 \end{vmatrix} \; kN$$

$$Q_{roof\_y} = \frac{V_{by} \cdot W_{roof\_i} \cdot h_{roof}^{2}}{W_{sill} \cdot h_{sill}^{2} + W_{lintel} \cdot h_{lintel}^{2} + W_{roof} \cdot h_{roof}^{2}} = \begin{vmatrix} 1.249 \\ 1.192 \\ 1.192 \\ 1.249 \\ 2.119 \\ 2.152 \\ 2.119 \\ 2.059 \\ 2.094 \\ 2.118 \\ 1.264 \\ 1.208 \\ 1.208 \\ 1.266 \\ 1.385 \end{vmatrix} \; kN$$

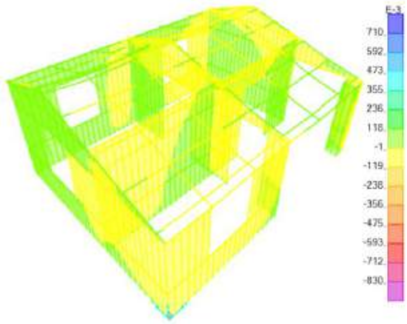
$$Q_{sill} = \left(\sum_{i=1}^{19} Q_{sill\_x}(i)\right) = 2.612 \; kN \qquad Q_{lintel} = \left(\sum_{i=1}^{19} Q_{lintel\_x}(i)\right) = 15.346 \; kN$$

$$Q_{roof} = \left(\sum_{i=1}^{14} Q_{roof\_x}(i)\right) = 22.623 \; kN \qquad Q_{tot} = Q_{sill} + Q_{lintel} + Q_{roof} = 40.581 \; kN$$

J
Displacements - Ideal Model



Load case: 1a
Displacements in x-direction [mm]
Scale factor: 200



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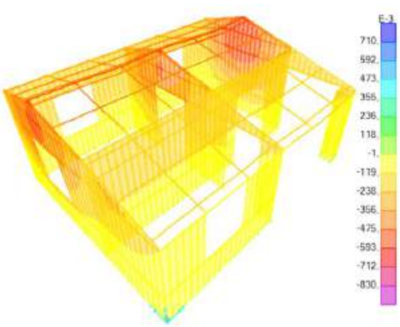
Displacements in y-direction [mm]
Scale factor: 200



Load case: 1b
Displacements in x-direction [mm]
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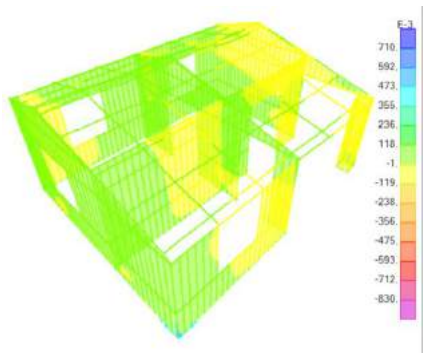
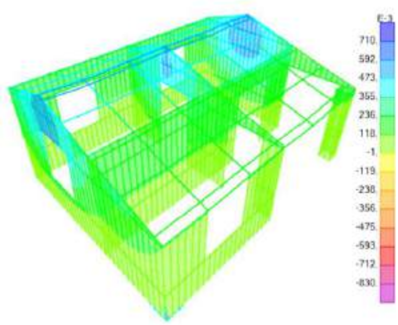


Load case: 1b
Displacements in y-direction [mm]
Scale factor: 200



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Displacements in x-direction [mm]
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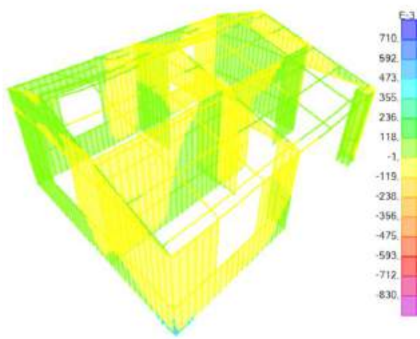
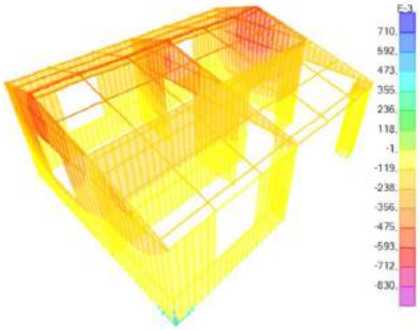


Load case: 2a
Displacements in y-direction [mm]
Scale factor: 200



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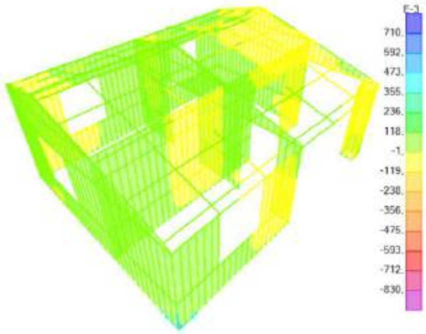

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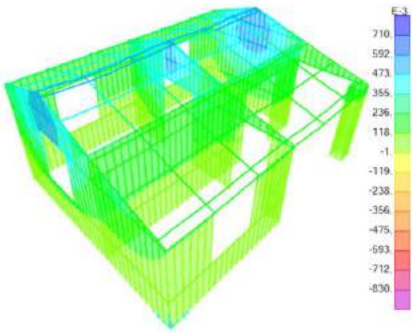
Displacements in y-direction [mm]
Scale factor: 200



Load case: 3a
Displacements in x-direction [mm]
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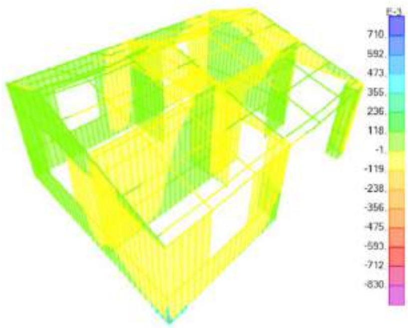
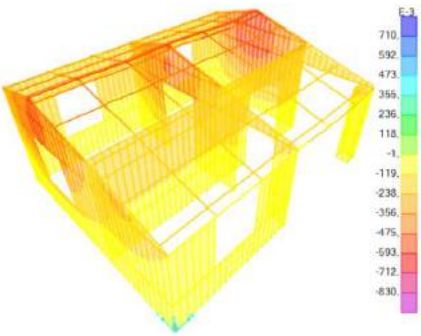
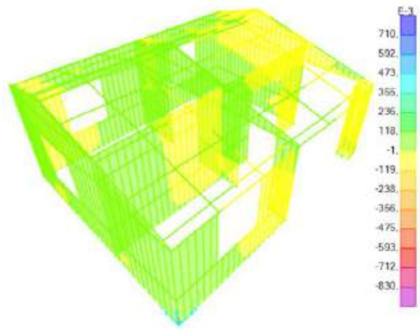


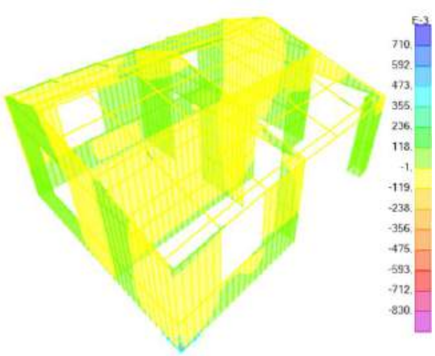
Load case: 3a
Displacements in y-direction [mm]
Scale factor: 200



Load case: 3b
Displacements in x-direction [mm]
Scale factor: 200



Load case: 3b
Displacements in y-direction [mm]
Scale factor: 200



Load case: 4a
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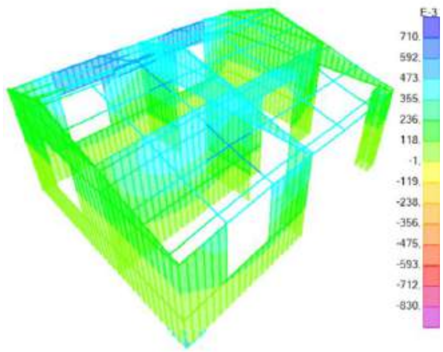
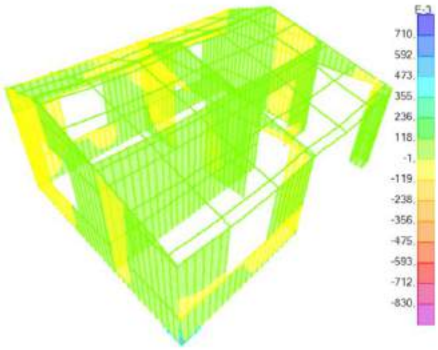
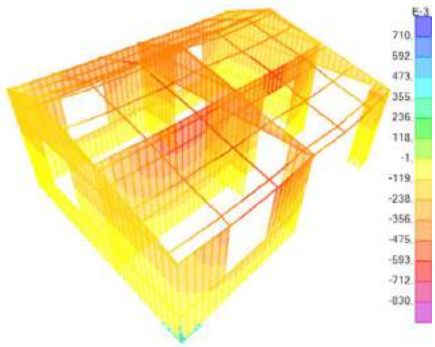


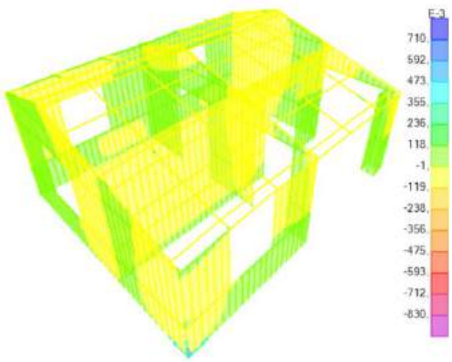
Load case: 4a
Displacements in y-direction [mm]
Scale factor: 200



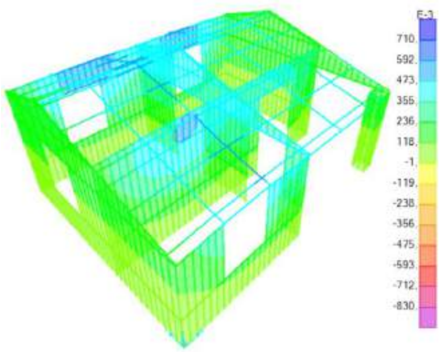
Load case: 4b
Displacements in x-direction [mm]
Scale factor: 200



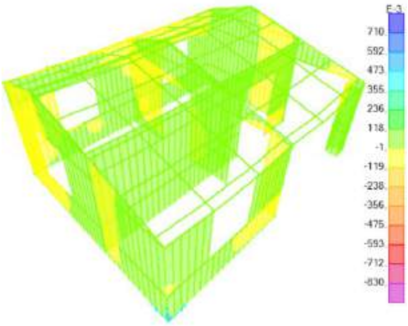
Load case: 4b
Displacements in y-direction [mm]
Scale factor: 200



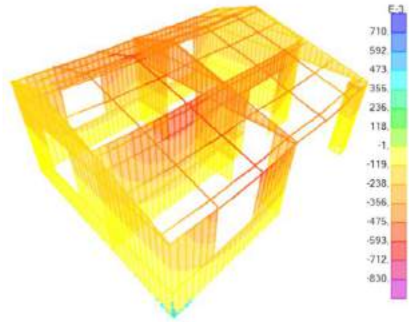
Load case: 5a
Displacements in x-direction [mm]
Scale factor: 200



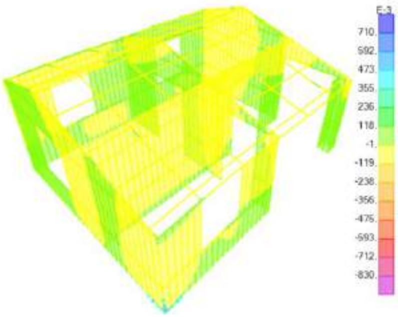
Load case: 5a
Displacements in y-direction [mm]
Scale factor: 200



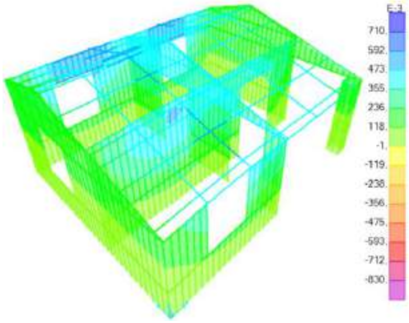
Load case: 5b
Displacements in x-direction [mm]
Scale factor: 200



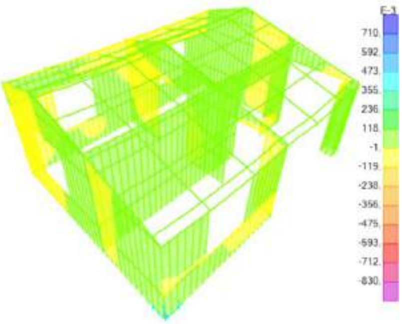
Load case: 5b
Displacements in y-direction [mm]
Scale factor: 200



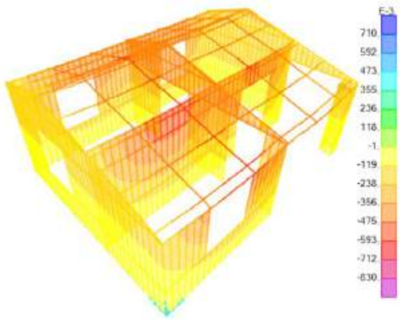
Load case: 6a
Displacements in x-direction [mm]
Scale factor: 200



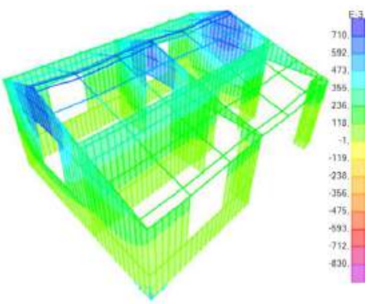
Load case: 6a
Displacements in y-direction [mm]
Scale factor: 200



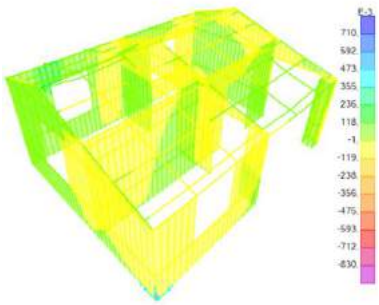
Load case: 6b
Displacements in x-direction [mm]
Scale factor: 200



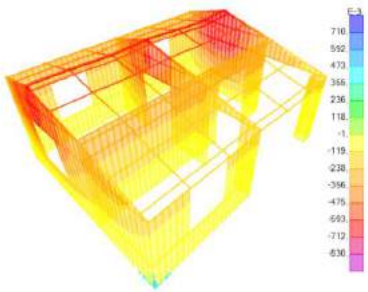
Load case: 6b
Displacements in y-direction [mm]
Scale factor: 200



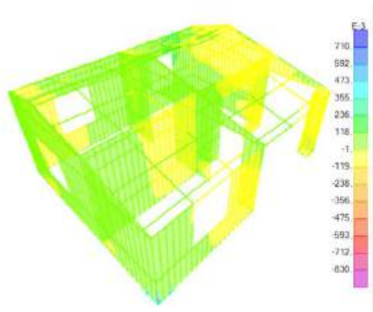
Load case: 7a
Displacements in x-direction [mm]
Scale factor: 200



Load case: 7a
Displacements in y-direction [mm]
Scale factor: 200



Load case: 7b
Displacements in x-direction [mm]
Scale factor: 200



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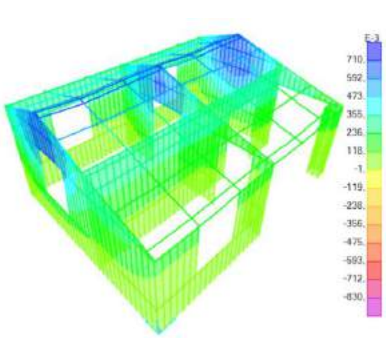
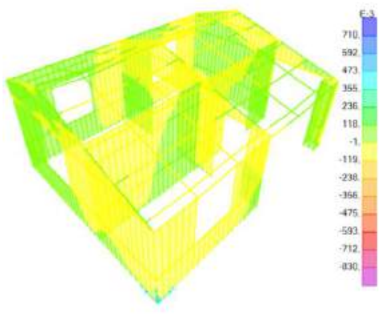
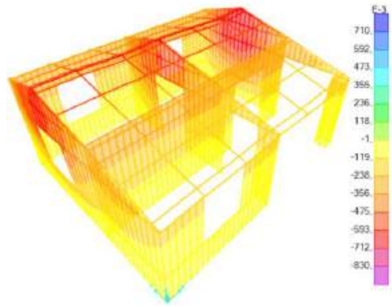
Displacements in y-direction [mm]
Scale factor: 200



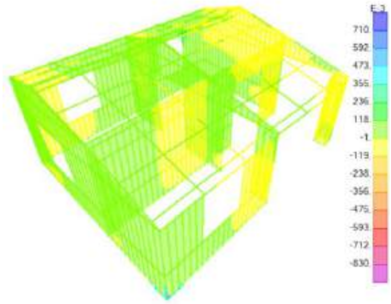
Load case: 8a
Displacements in x-direction [mm]
Scale factor: 200



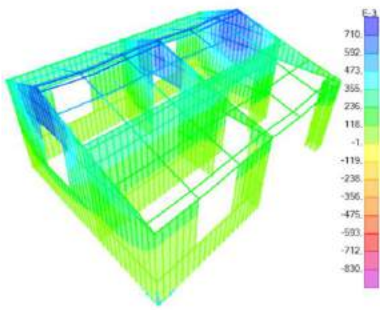
Load case: 8a
Displacements in y-direction [mm]
Scale factor: 200



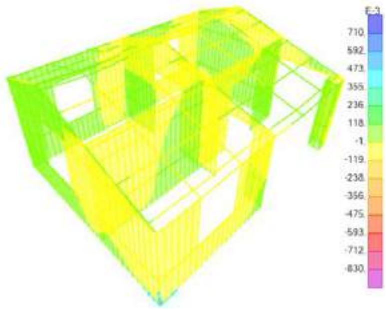
Load case: 8b
Displacements in x-direction [mm]
Scale factor: 200



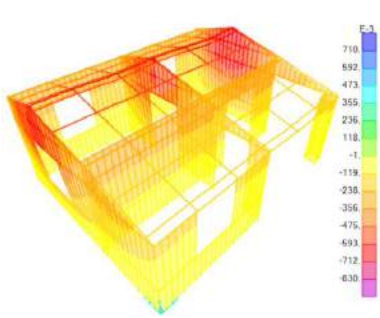
Load case: 8b
Displacements in y-direction [mm]
Scale factor: 200



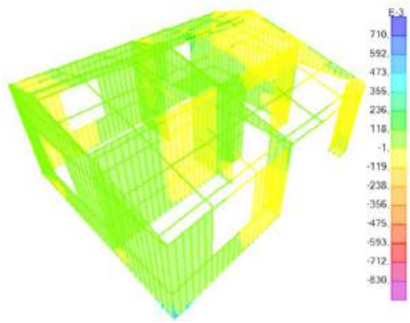
Load case: 9a
Displacements in x-direction [mm]
Scale factor: 200



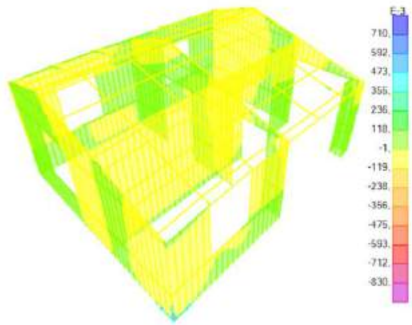
Load case: 9a
Displacements in y-direction [mm]
Scale factor: 200



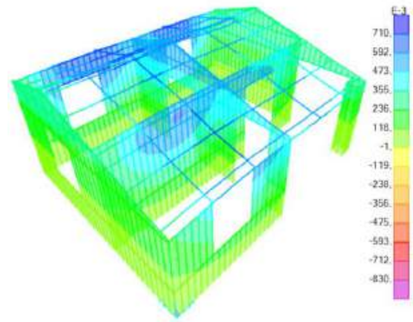
Load case: 9b
Displacements in x-direction [mm]
Scale factor: 200



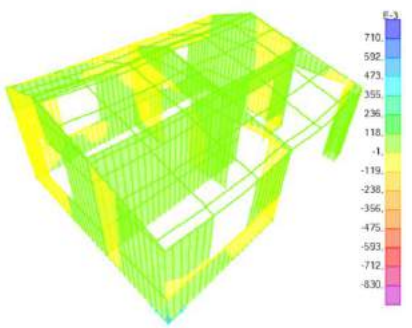
Load case: 9b
Displacements in y-direction [mm]
Scale factor: 200



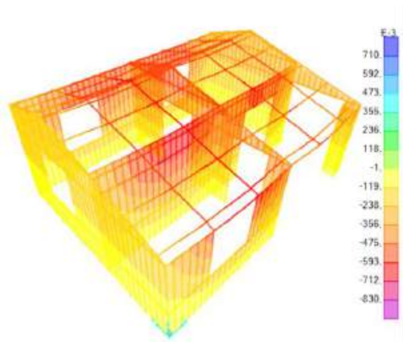
Load case: 10a
Displacements in x-direction [mm]
Scale factor: 200



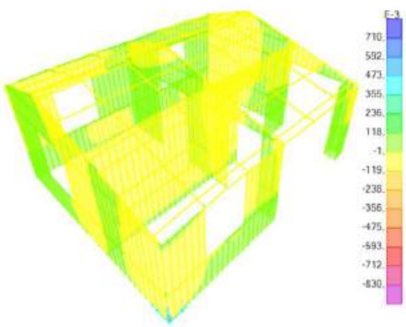
Load case: 10a
Displacements in y-direction [mm]
Scale factor: 200



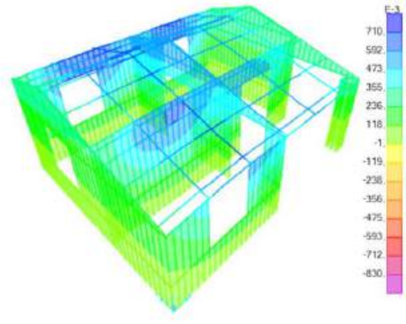
Load case: 10b
Displacements in x-direction [mm]
Scale factor: 200



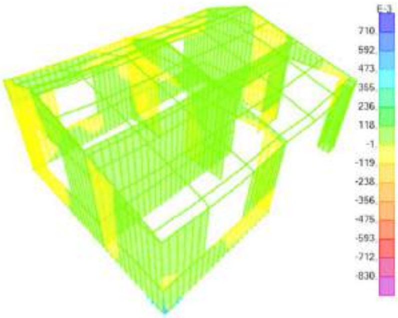
Load case: 10b
Displacements in y-direction [mm]
Scale factor: 200



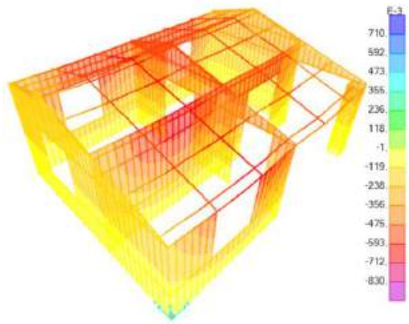
Load case: 11a
Displacements in x-direction [mm]
Scale factor: 200



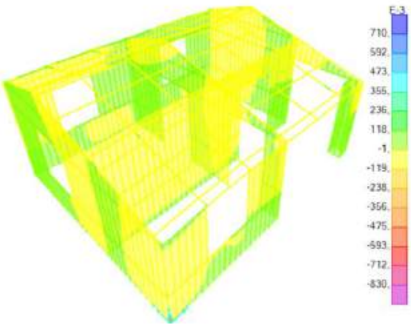
Load case: 11a
Displacements in y-direction [mm]
Scale factor: 200



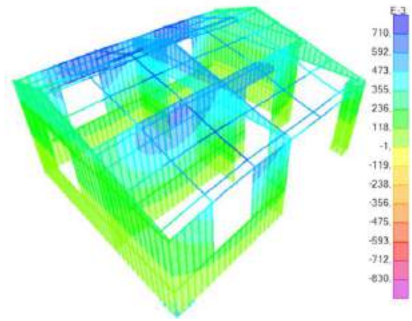
Load case: 11b
Displacements in x-direction [mm]
Scale factor: 200



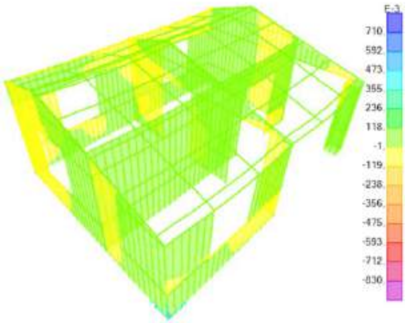
Load case: 11b
Displacements in y-direction [mm]
Scale factor: 200



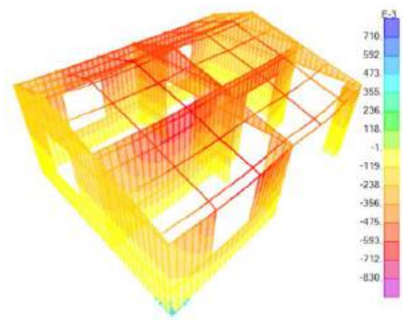
Load case: 12a
Displacements in x-direction [mm]
Scale factor: 200



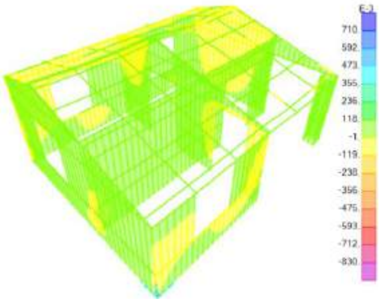
Load case: 12a
Displacements in y-direction [mm]
Scale factor: 200



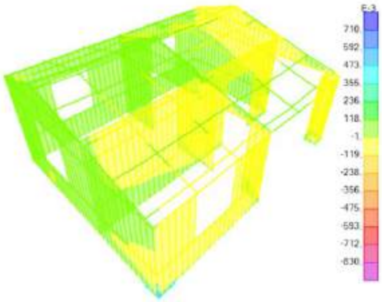
Load case: 13b
Displacements in x-direction [mm]
Scale factor: 200



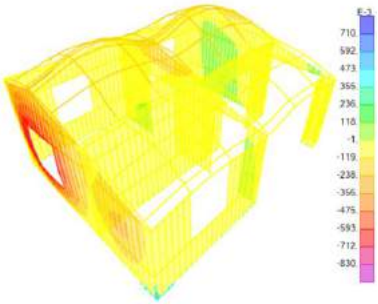
Load case: 13b
Displacements in y-direction [mm]
Scale factor: 200



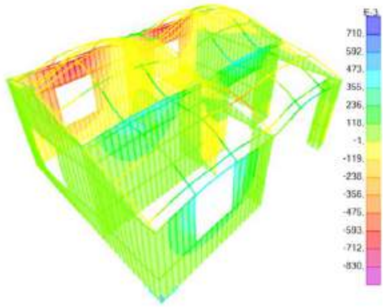
Load case: 13
Displacements in x-direction [mm]
Scale factor: 200



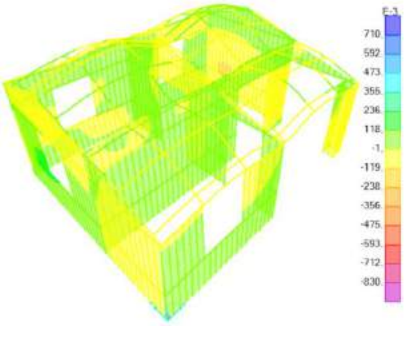
Load case: 13
Displacements in y-direction [mm]
Scale factor: 200



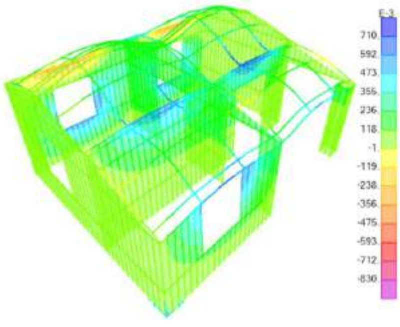
Load case: 14
Displacements in x-direction [mm]
Scale factor: 200



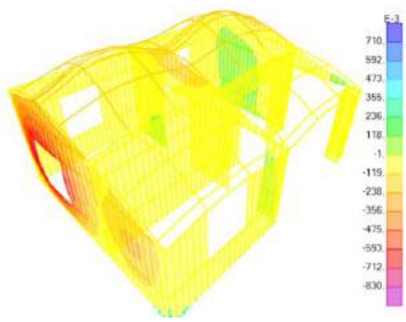
Load case: 14
Displacements in y-direction [mm]
Scale factor: 200



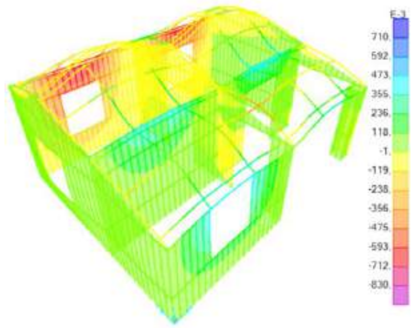
Load case: 15
Displacements in x-direction [mm]
Scale factor: 200



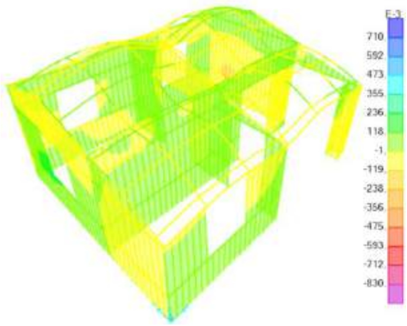
Load case: 15
Displacements in y-direction [mm]
Scale factor: 200



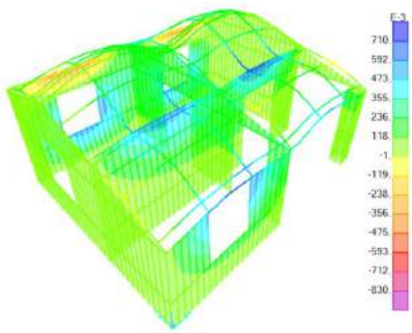
Load case: 16
Displacements in x-direction [mm]
Scale factor: 200



Load case: 16
Displacements in y-direction [mm]
Scale factor: 200



Load case: 17
Displacements in x-direction [mm]
Scale factor: 200



Load case: 17
Displacements in y-direction [mm]
Scale factor: 200

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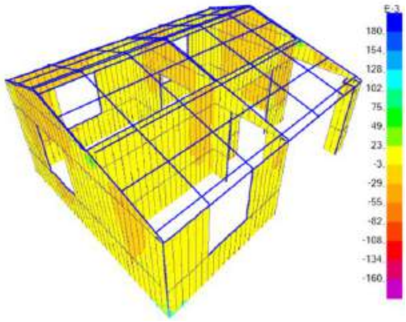
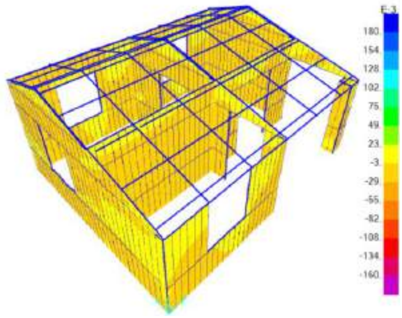
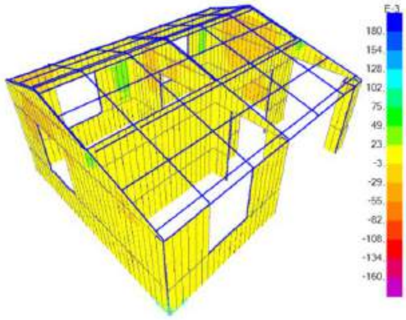
Stresses - Ideal Model



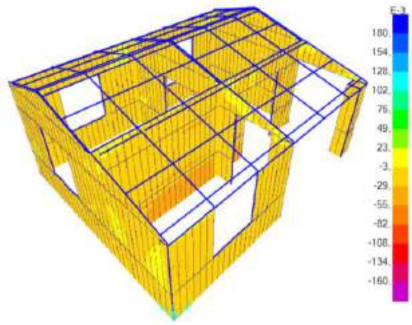
Load case: 1a
S11 [N/mm²]
Visible face



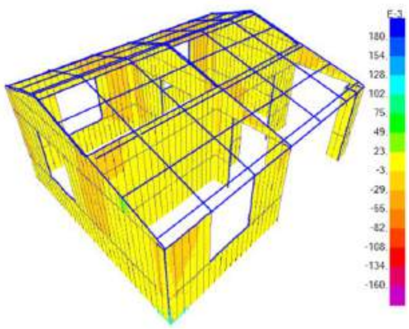
Load case: 1a
S22 [N/mm²]
Visible face



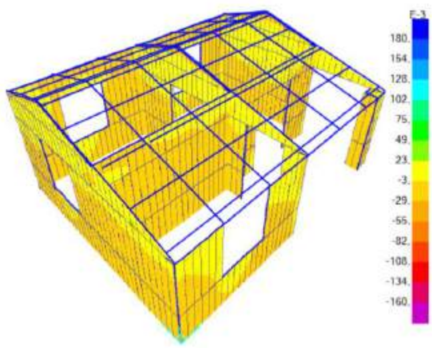
Load case: 1a
SMax [N/mm²]
Visible face



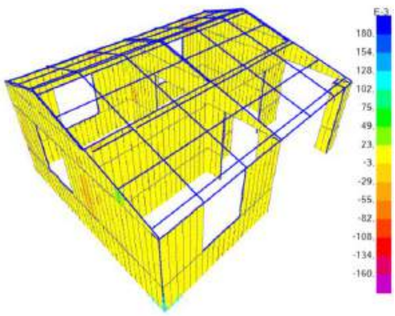
Load case: 1a
SMin [N/mm²]
Visible face



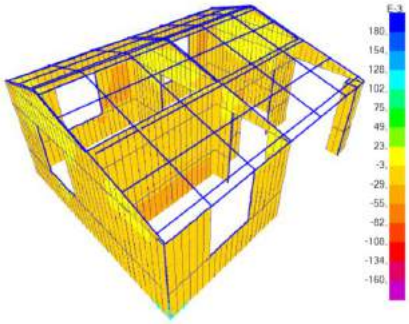
Load case: 1b
S11 [N/mm²]
Visible face



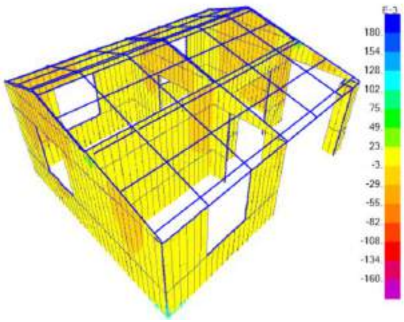
Load case: 1b
S22 [N/mm²]
Visible face



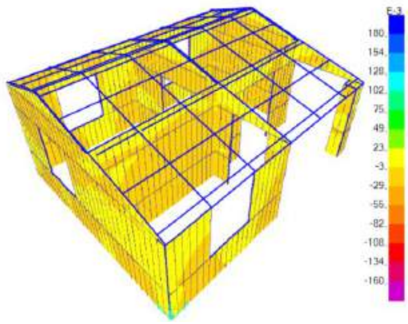
Load case: 1b
SMax [N/mm²]
Visible face



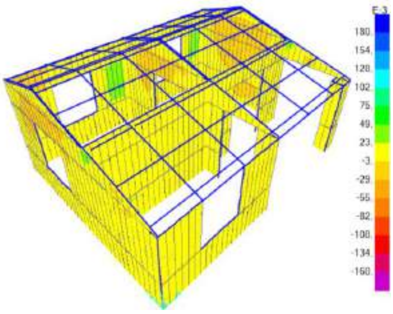
Load case: 1b
SMin [N/mm²]
Visible face



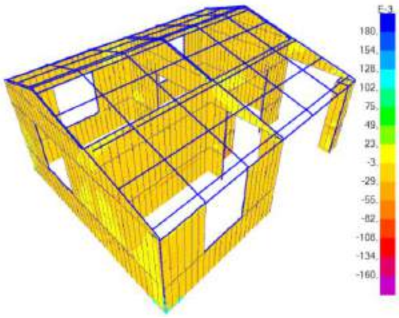
Load case: 2a
S11 [N/mm²]
Visible face



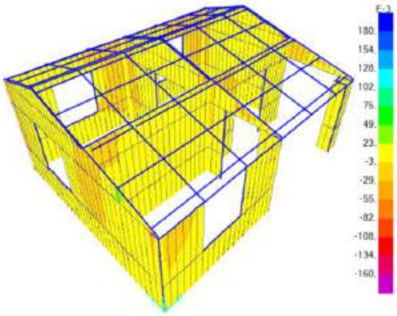
Load case: 2a
S22 [N/mm²]
Visible face



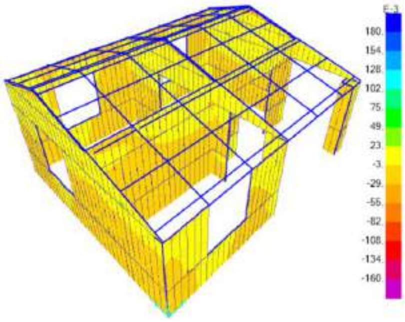
Load case: 2a
SMax [N/mm²]
Visible face



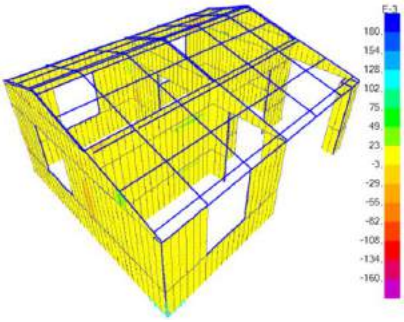
Load case: 2a
SMin [N/mm²]
Visible face



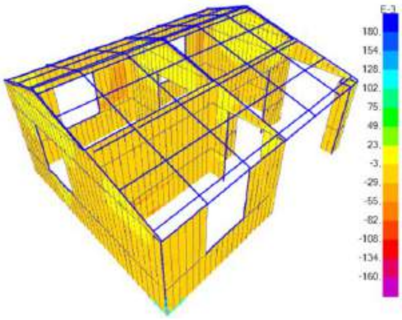
Load case: 2b
S11 [N/mm²]
Visible face



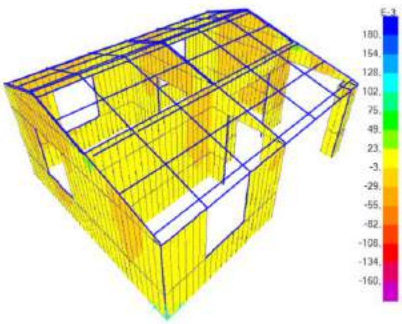
Load case: 2b
S22 [N/mm²]
Visible face



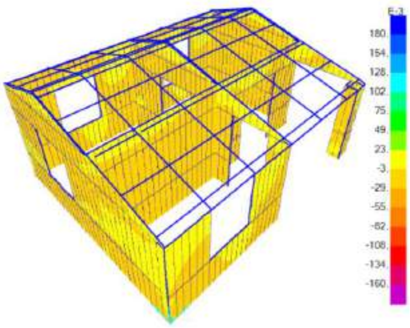
Load case: 2b
SMax [N/mm²]
Visible face



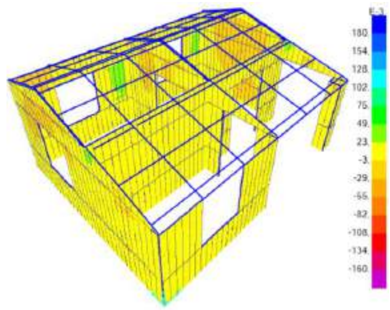
Load case: 2b
SMin [N/mm²]
Visible face



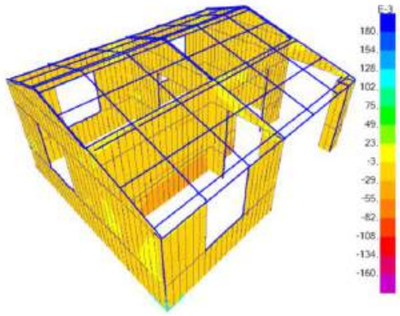
Load case: 3a
S11 [N/mm²]
Visible face



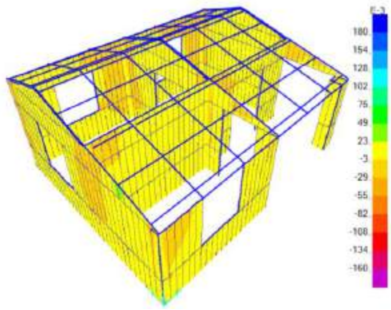
Load case: 3a
S22 [N/mm²]
Visible face



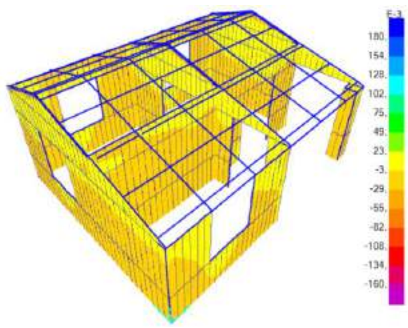
Load case: 3a
SMax [N/mm²]
Visible face



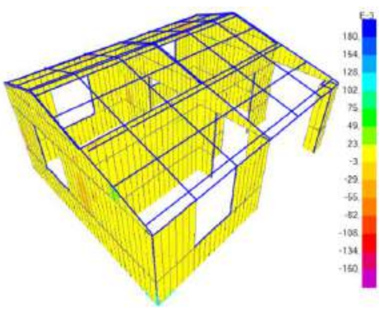
Load case: 3a
SMin [N/mm²]
Visible face



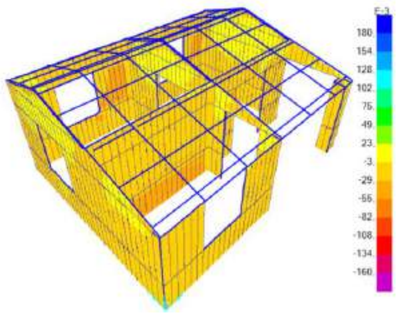
Load case: 3b
S11 [N/mm²]
Visible face



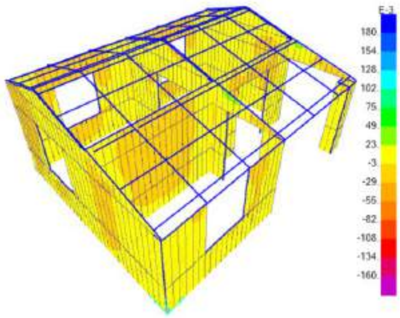
Load case: 3b
S22 [N/mm²]
Visible face



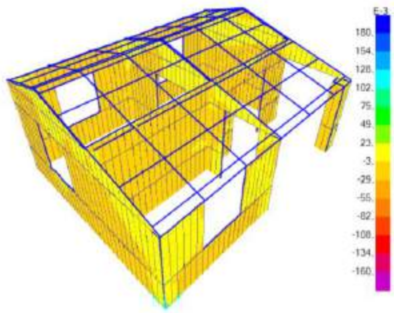
Load case: 3b
SMax [N/mm²]
Visible face



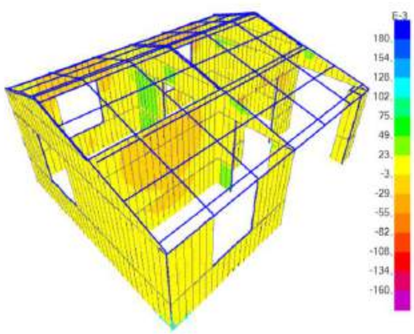
Load case: 3b
SMin [N/mm²]
Visible face



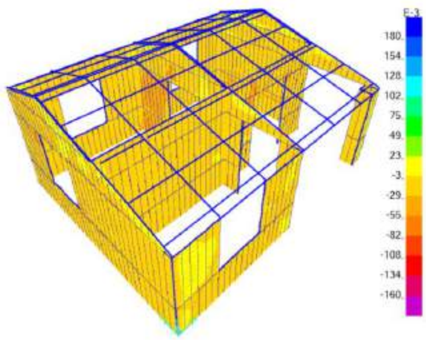
Load case: 4a
S11 [N/mm²]
Visible face



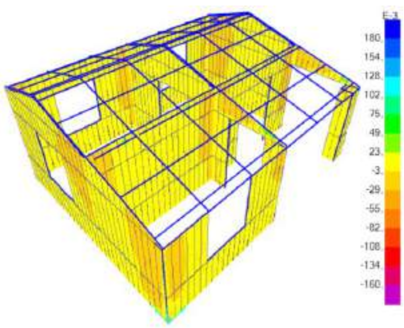
Load case: 4a
S22 [N/mm²]
Visible face



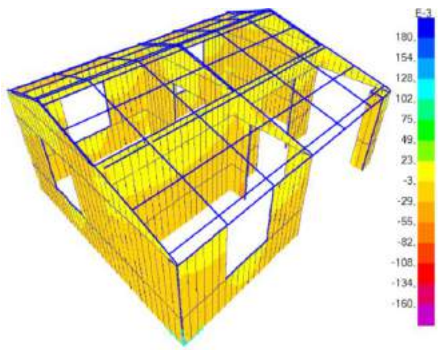
Load case: 4a
SMax [N/mm²]
Visible face



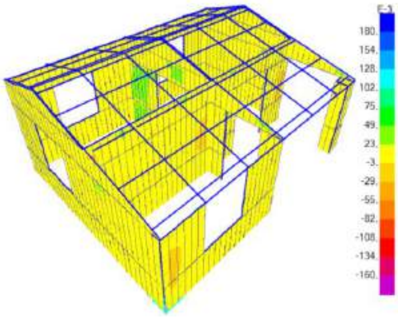
Load case: 4a
SMin [N/mm²]
Visible face



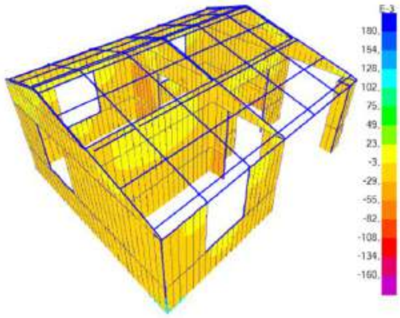
Load case: 4b
S11 [N/mm²]
Visible face



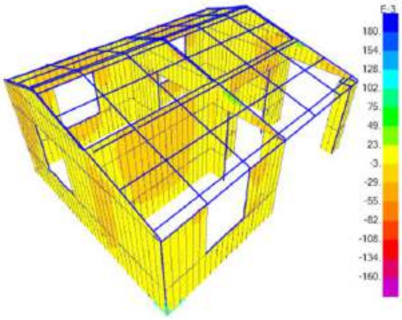
Load case: 4b
S22 [N/mm²]
Visible face



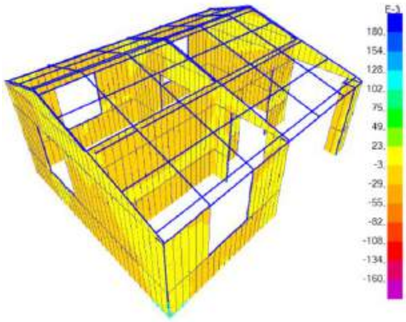
Load case: 4b
SMax [N/mm²]
Visible face



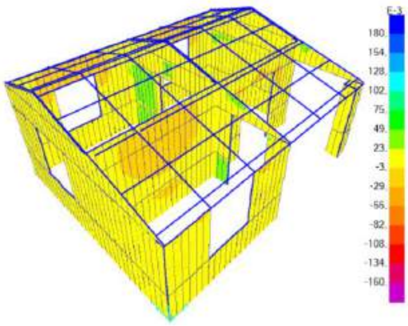
Load case: 4b
SMin [N/mm²]
Visible face



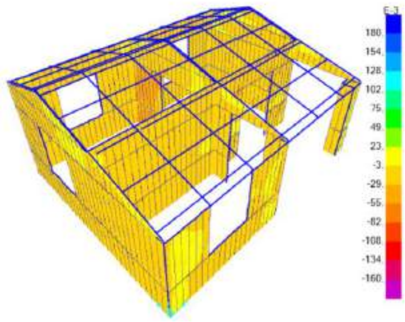
Load case: 5a
S11 [N/mm²]
Visible face



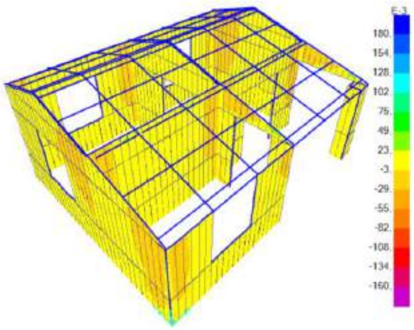
Load case: 5a
S22 [N/mm²]
Visible face



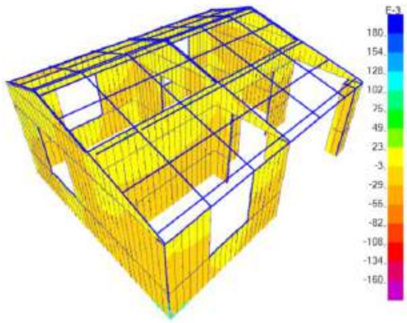
Load case: 5a
SMax [N/mm²]
Visible face



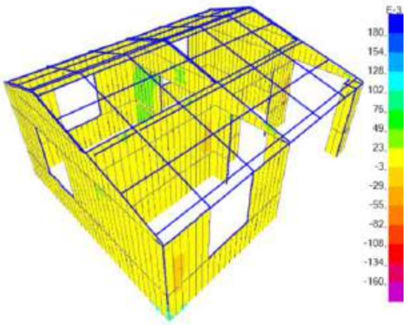
Load case: 5a
SMin [N/mm²]
Visible face



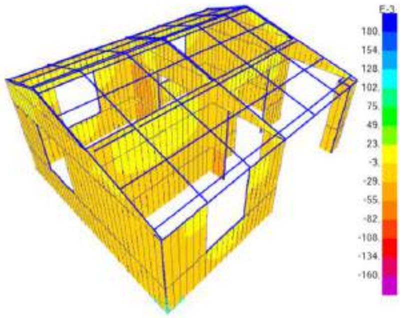
Load case: 5b
S11 [N/mm²]
Visible face



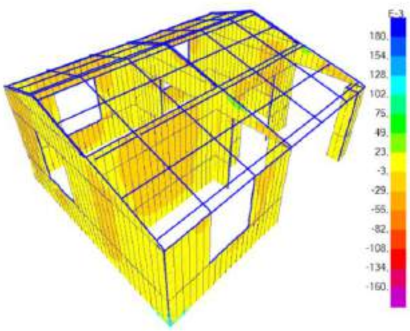
Load case: 5b
S22 [N/mm²]
Visible face



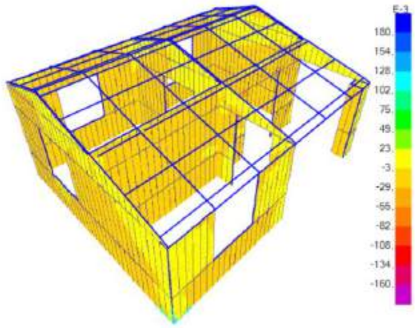
Load case: 5b
SMax [N/mm²]
Visible face



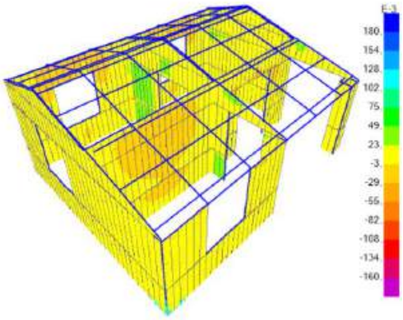
Load case: 5b
SMin [N/mm²]
Visible face



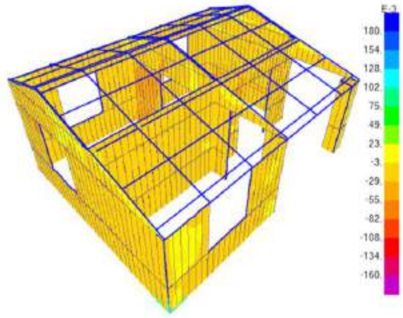
Load case: 6a
S11 [N/mm²]
Visible face



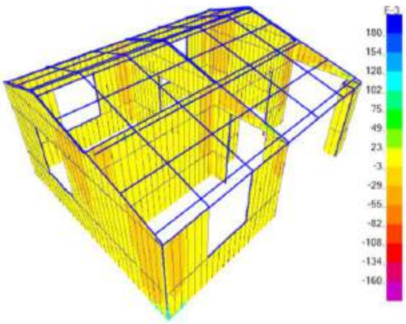
Load case: 6a
S22 [N/mm²]
Visible face



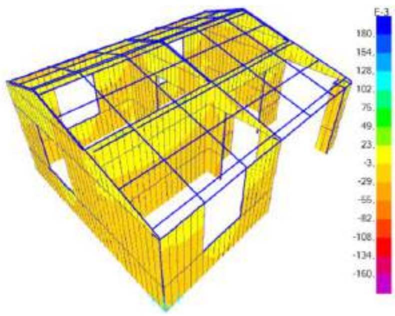
Load case: 6a
SMax [N/mm²]
Visible face



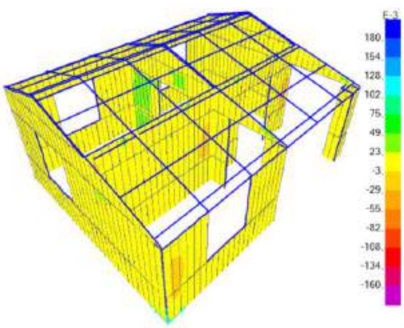
Load case: 6a
SMin [N/mm²]
Visible face



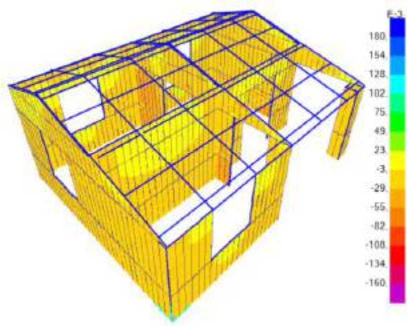
Load case: 6b
S11 [N/mm²]
Visible face



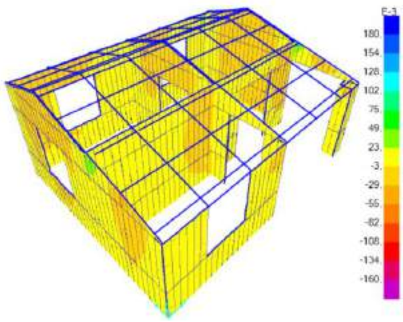
Load case: 6b
S22 [N/mm²]
Visible face



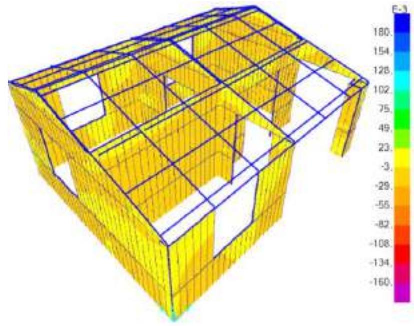
Load case: 6b
SMax [N/mm²]
Visible face



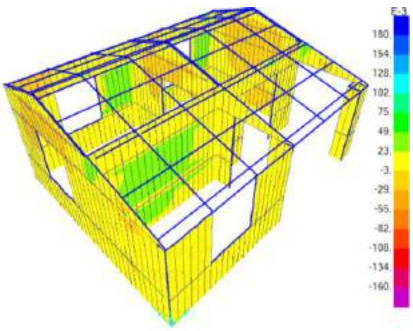
Load case: 6b
SMin [N/mm²]
Visible face



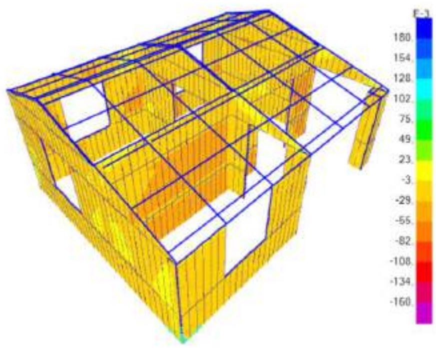
Load case: 7a
S11 [N/mm^2]
Visible face



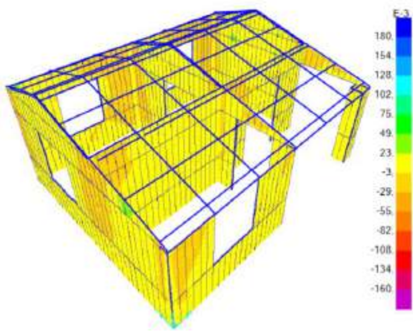
Load case: 7a
S22 [N/mm^2]
Visible face



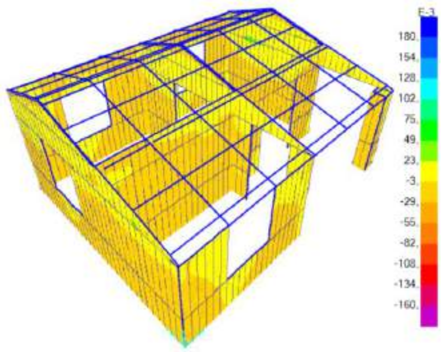
Load case: 7a
SMax [N/mm^2]
Visible face



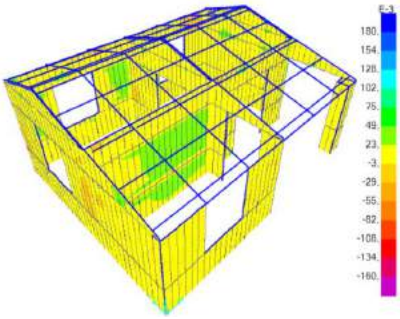
Load case: 7a
SMin [N/mm^2]
Visible face



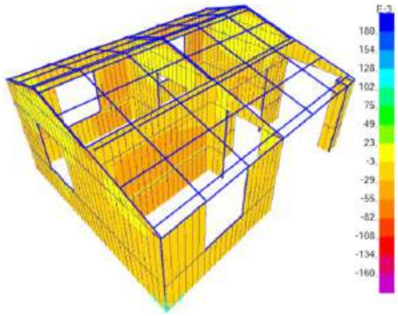
Load case: 7b
S11 [N/mm^2]
Visible face



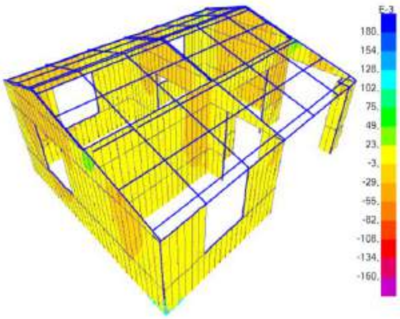
Load case: 7b
S22 [N/mm^2]
Visible face



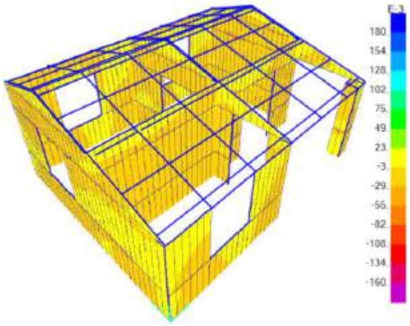
Load case: 7b
SMax [N/mm²]
Visible face



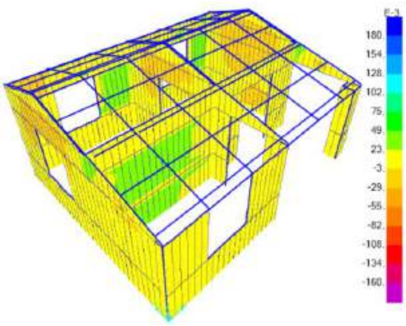
Load case: 7b
SMin [N/mm²]
Visible face



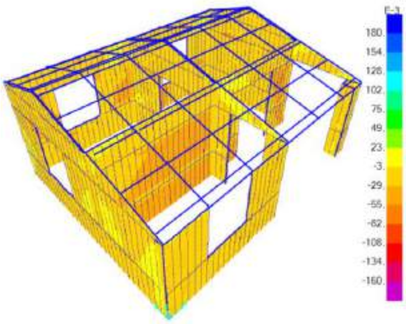
Load case: 8a
S11 [N/mm²]
Visible face



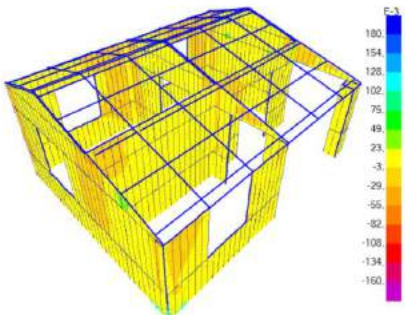
Load case: 8a
S22 [N/mm²]
Visible face



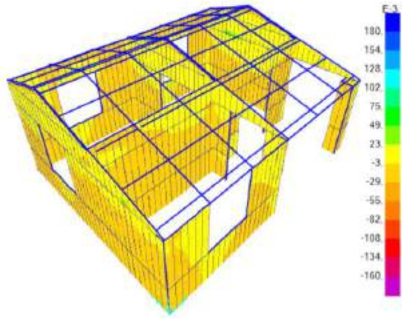
Load case: 8a
SMax [N/mm²]
Visible face



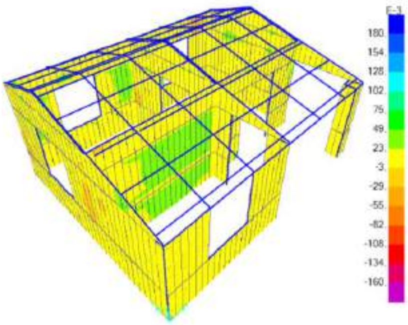
Load case: 8a
SMin [N/mm²]
Visible face



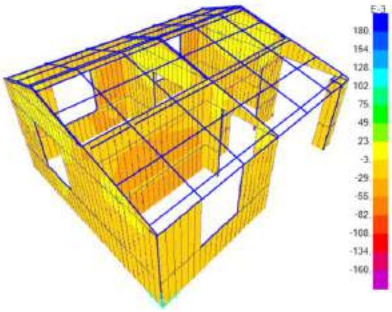
Load case: 8b
S11 [N/mm^2]
Visible face



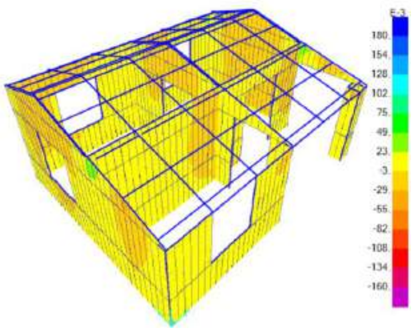
Load case: 8b
S22 [N/mm^2]
Visible face



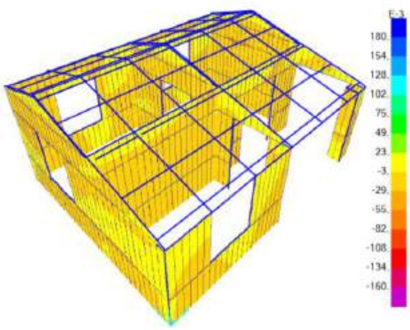
Load case: 8b
SMax [N/mm^2]
Visible face



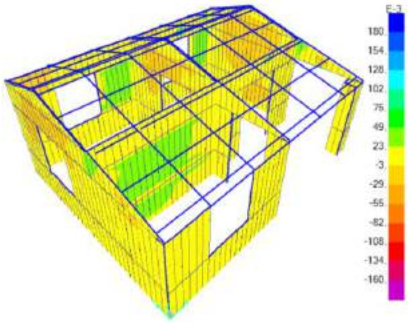
Load case: 8b
SMin [N/mm^2]
Visible face



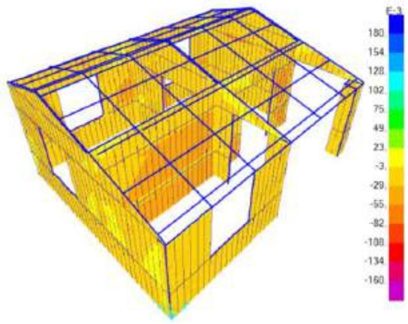
Load case: 9a
S11 [N/mm^2]
Visible face



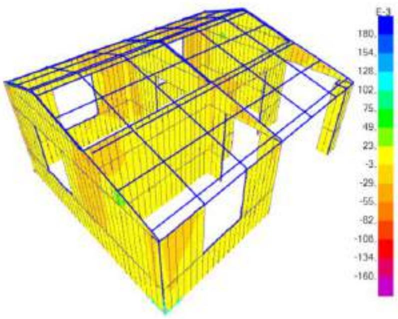
Load case: 9a
S22 [N/mm^2]
Visible face



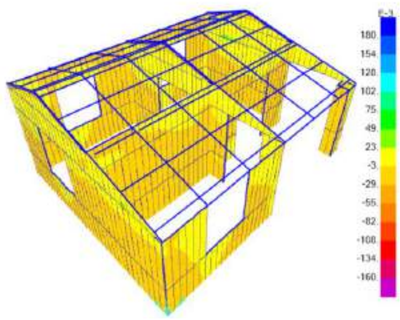
Load case: 9a
SMax [N/mm²]
Visible face



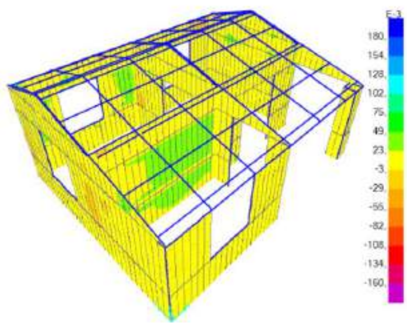
Load case: 9a
SMin [N/mm²]
Visible face



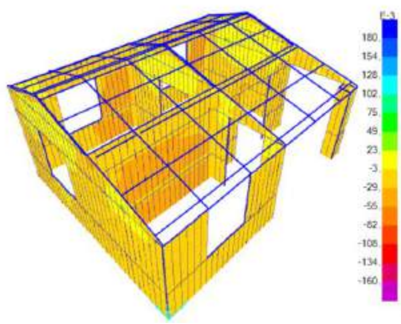
Load case: 9b
S11 [N/mm²]
Visible face



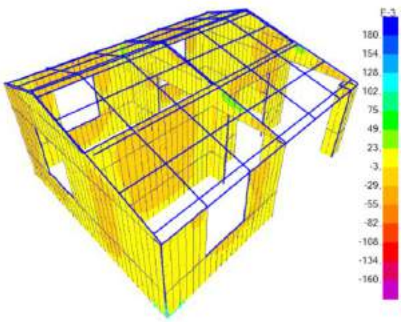
Load case: 9b
S22 [N/mm²]
Visible face



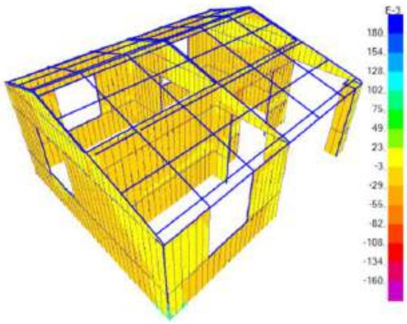
Load case: 9b
SMax [N/mm²]
Visible face



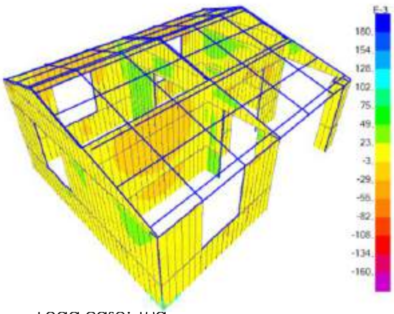
Load case: 9b
SMin [N/mm²]
Visible face



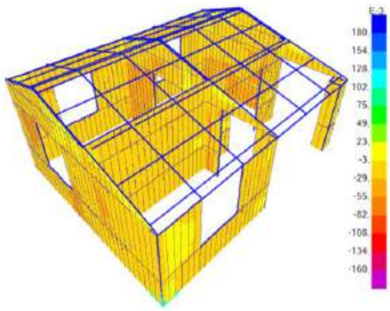
Load case: 10a
S11 [N/mm^2]
Visible face



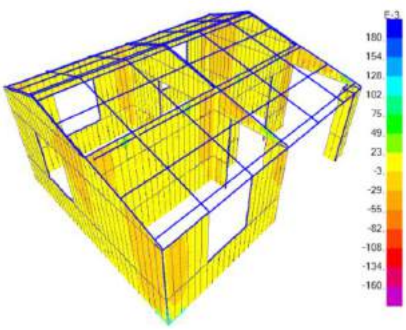
Load case: 10a
S22 [N/mm^2]
Visible face



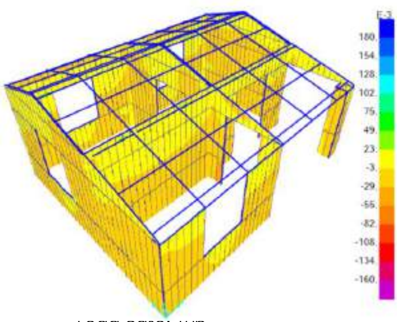
Load case: 10a
SMax [N/mm^2]
Visible face



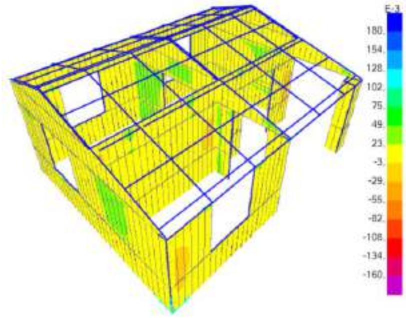
Load case: 10a
SMin [N/mm^2]
Visible face



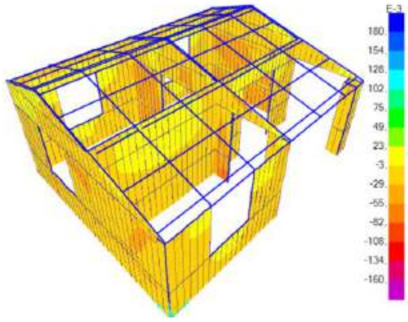
Load case: 10b
S11 [N/mm^2]
Visible face



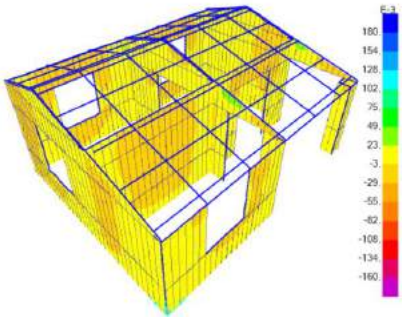
Load case: 10b
S22 [N/mm^2]
Visible face



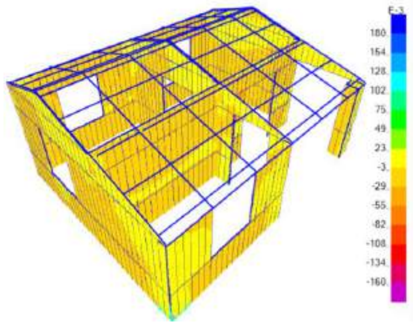
Load case: 10b
SMax [N/mm²]
Visible face



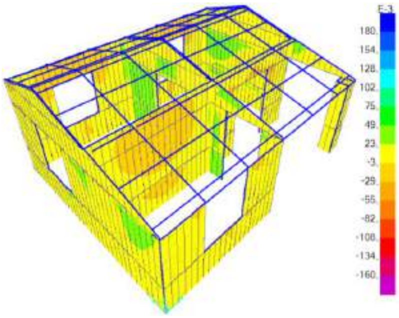
Load case: 10b
SMin [N/mm²]
Visible face



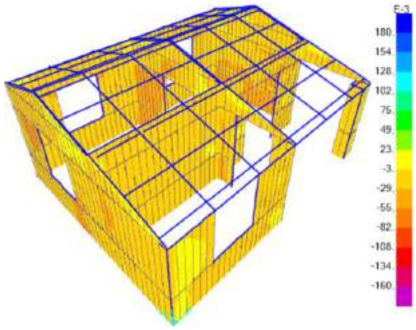
Load case: 11a
S11 [N/mm²]
Visible face



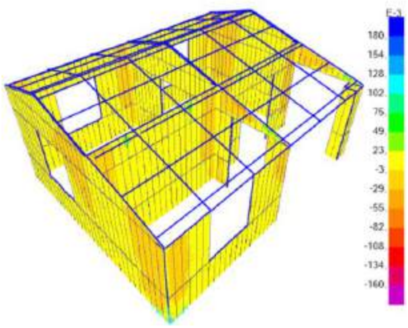
Load case: 11a
S22 [N/mm²]
Visible face



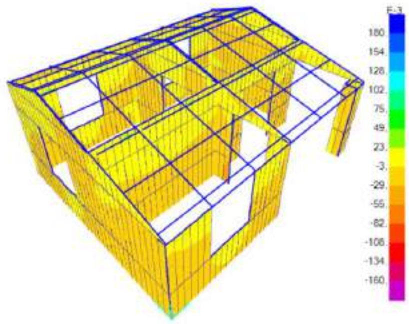
Load case: 11a
SMax [N/mm²]
Visible face



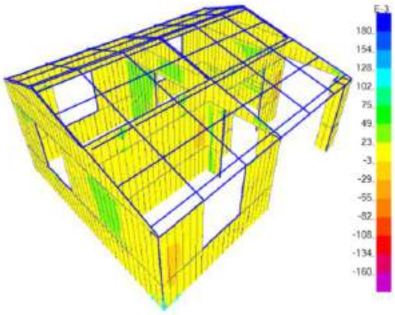
Load case: 11a
SMin [N/mm²]
Visible face



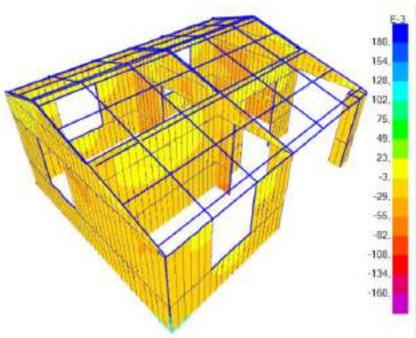
Load case: 11b
S11 [N/mm^2]
Visible face



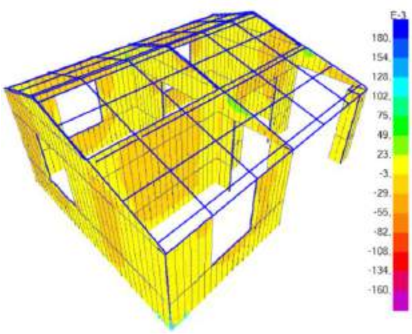
Load case: 11b
S22 [N/mm^2]
Visible face



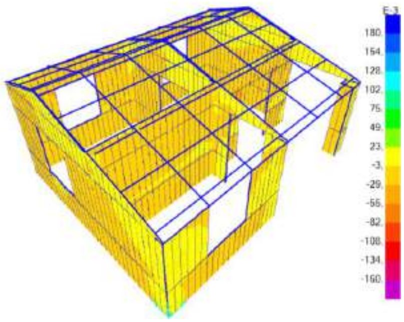
Load case: 11b
SMax [N/mm^2]
Visible face



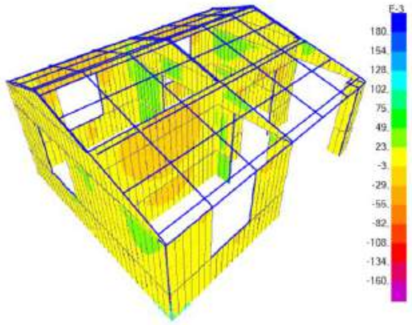
Load case: 11b
SMin [N/mm^2]
Visible face



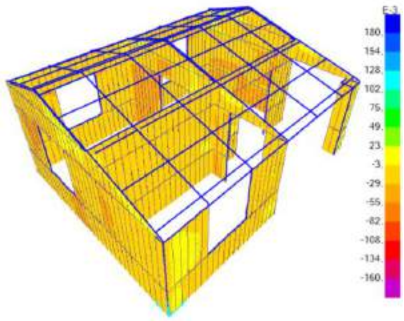
Load case: 12a
S11 [N/mm^2]
Visible face



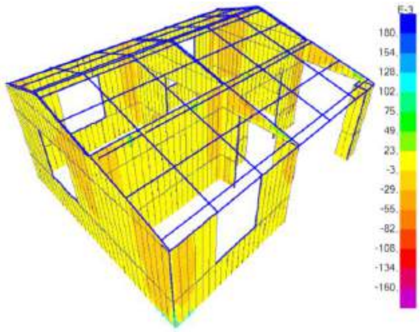
Load case: 12a
S22 [N/mm^2]
Visible face



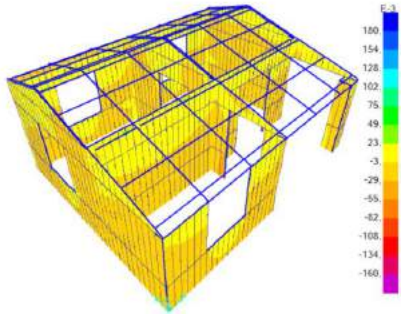
Load case: 12a
SMax [N/mm²]
Visible face



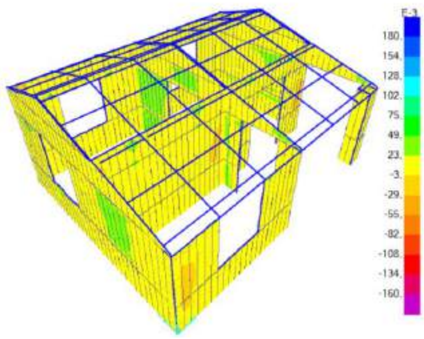
Load case: 12a
SMin [N/mm²]
Visible face



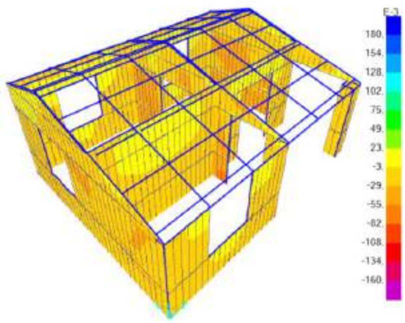
Load case: 12b
S11 [N/mm²]
Visible face



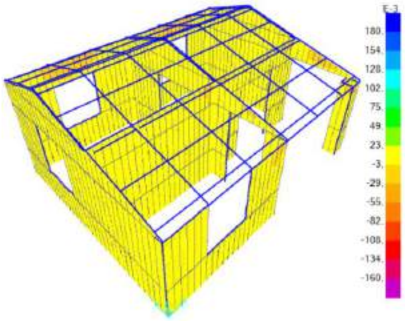
Load case: 12b
S22 [N/mm²]
Visible face



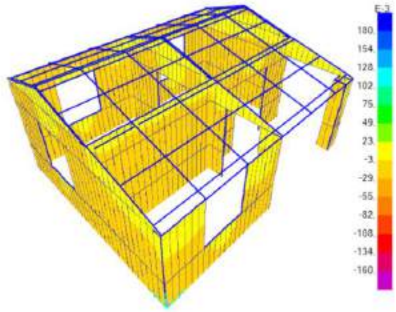
Load case: 12b
SMax [N/mm²]
Visible face



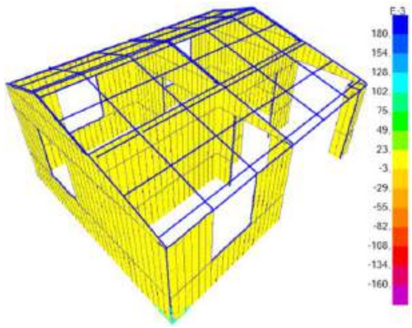
Load case: 12b
SMin [N/mm²]
Visible face



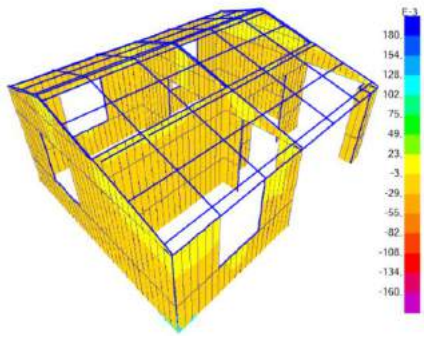
Load case: 13
S11 [N/mm²]
Visible face



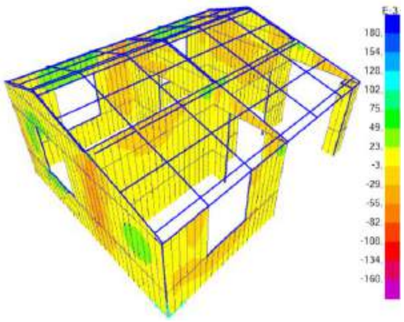
Load case: 13
S22 [N/mm²]
Visible face



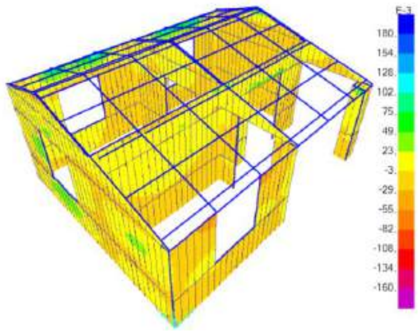
Load case: 13
SMax [N/mm²]
Visible face



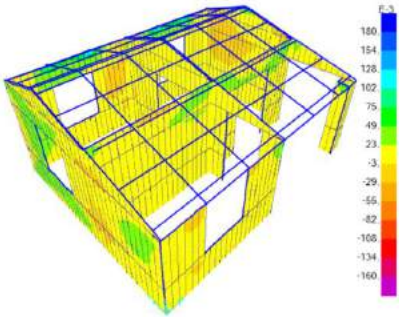
Load case: 13
SMin [N/mm²]
Visible face



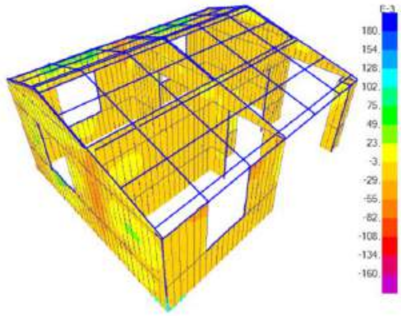
Load case: 14
S11 [N/mm²]
Visible face



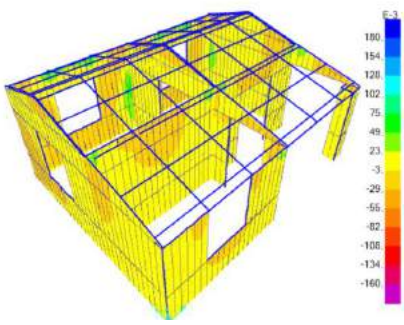
Load case: 14
S22 [N/mm²]
Visible face



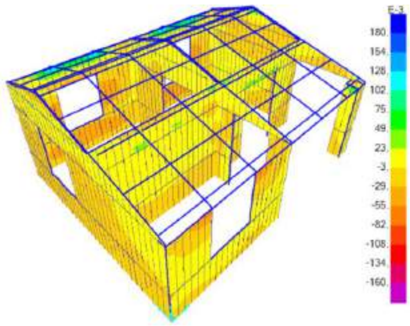
Load case: 14
SMax [N/mm²]
Visible face



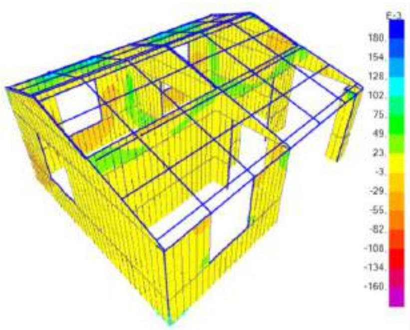
Load case: 14
SMin [N/mm²]
Visible face



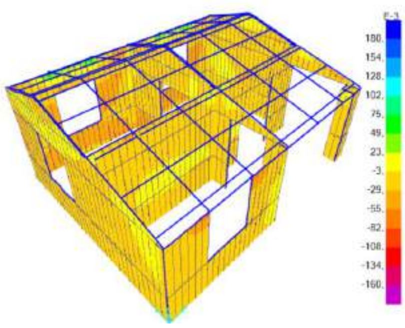
Load case: 15
S11 [N/mm²]
Visible face



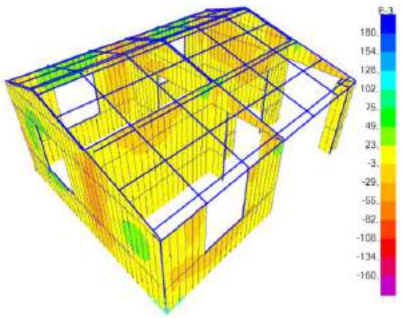
Load case: 15
S22 [N/mm²]
Visible face



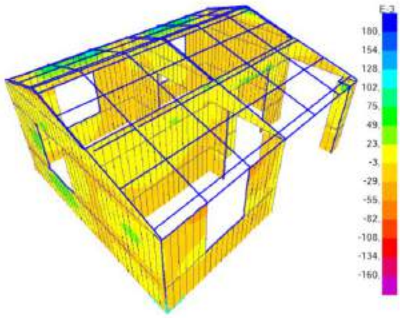
Load case: 15
SMax [N/mm²]
Visible face



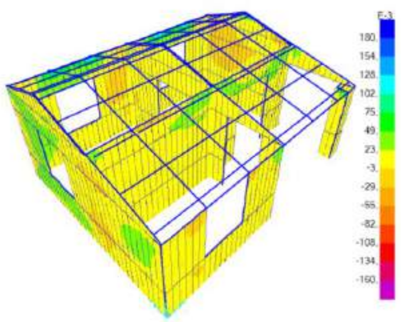
Load case: 15
SMin [N/mm²]
Visible face



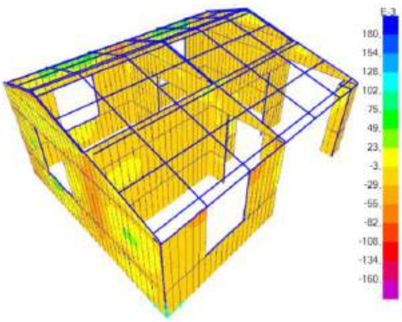
Load case: 16
S11 [N/mm^2]
Visible face



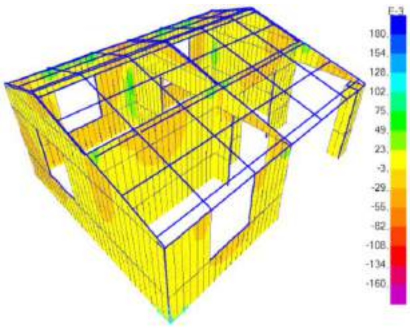
Load case: 16
S22 [N/mm^2]
Visible face



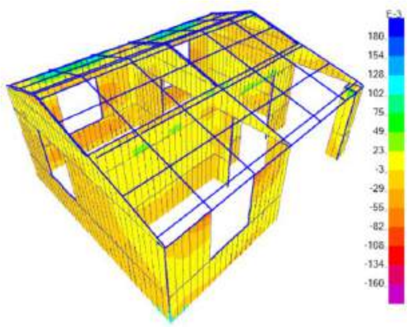
Load case: 16
SMax [N/mm^2]
Visible face



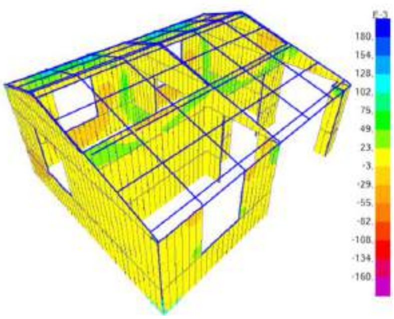
Load case: 16
SMin [N/mm^2]
Visible face



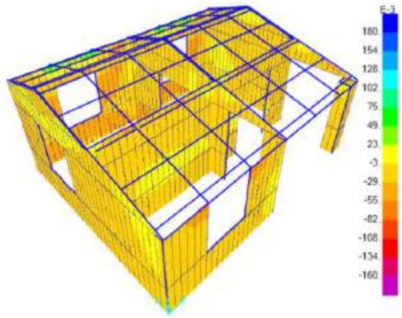
Load case: 17
S11 [N/mm^2]
Visible face



Load case: 17
S22 [N/mm^2]
Visible face



Load case: 17
SMax [N/mm²]
Visible face

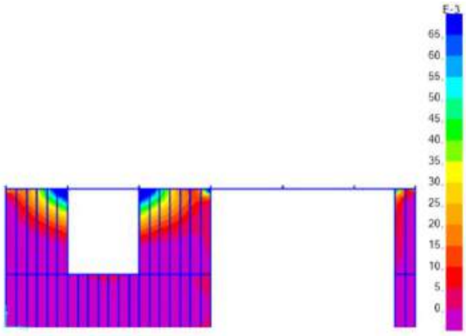


Load case: 17
SMin [N/mm²]
Visible face

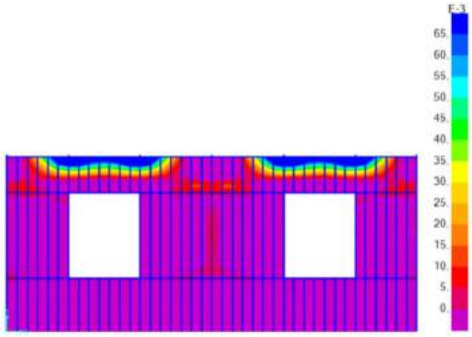
L

Tensile stresses

Ideal model – Tensile stress distribution

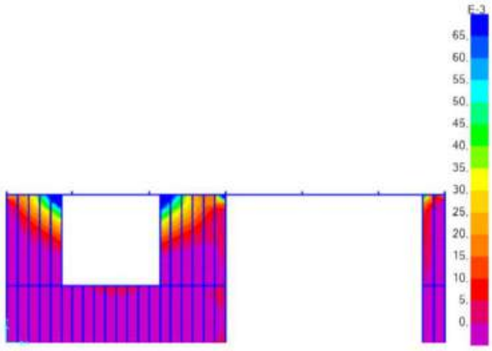


Wall 1 S22 LC 17 (bottom face)

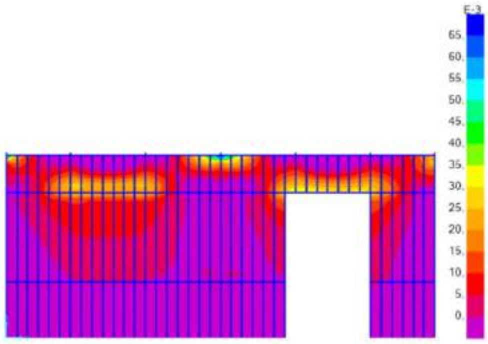


Wall 3 S22 LC 17 (top face)

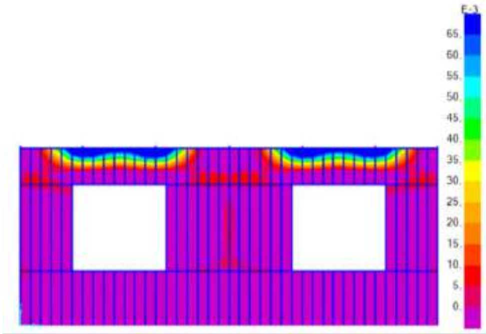
O1–Window openings made too big – Tensile stress distribution



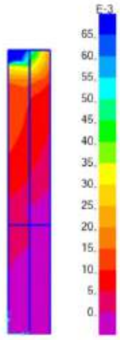
Wall 1 S22 LC 17 (bottom face)



Wall 2 S22 LC 17 (top face)

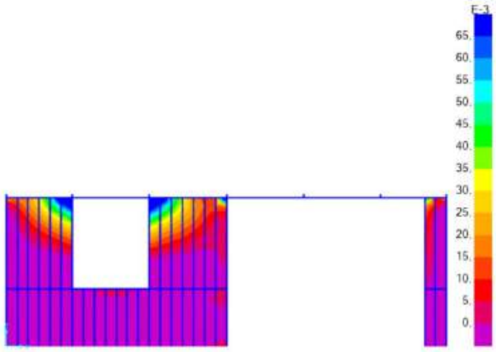


Wall 3 S22 LC 17 (top face)

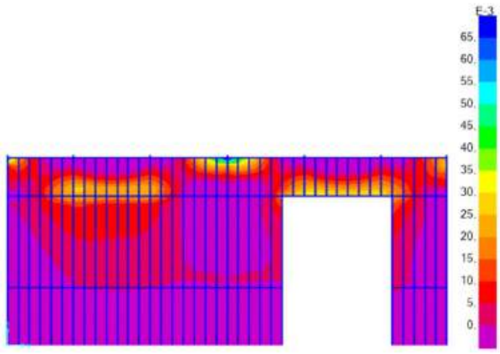


Column 1 S22 LC 17 (bottom face)

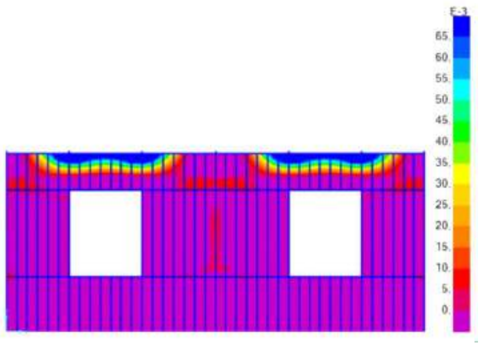
O1–Door openings made too big – Tensile stress distribution



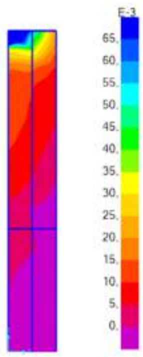
Wall 1 S22 LC 17 (bottom face)



Wall 2 S22 LC 17 (top face)

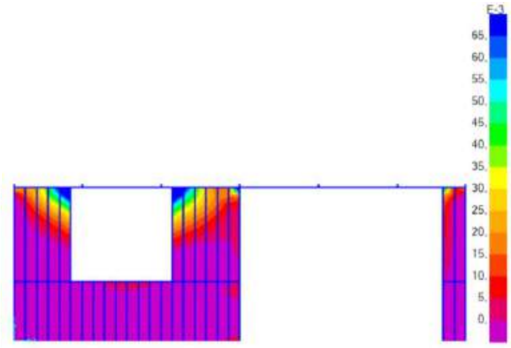


Wall 3 S22 LC 17 (top face)

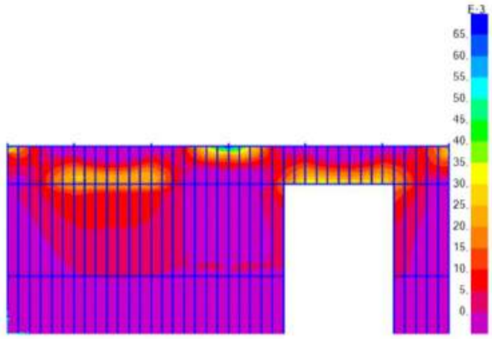


Column 1 S22 LC 17 (bottom face)

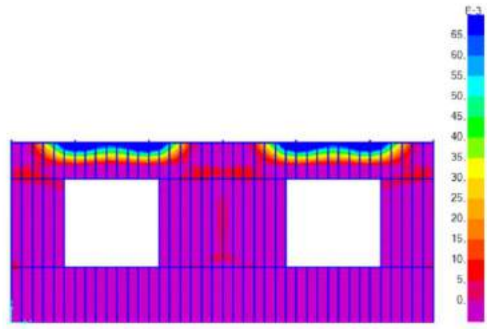
O1–All openings made too big – Tensile stress distribution



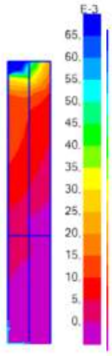
Wall 1 S22 LC 17 (bottom face)



Wall 2 S22 LC 17 (top face)

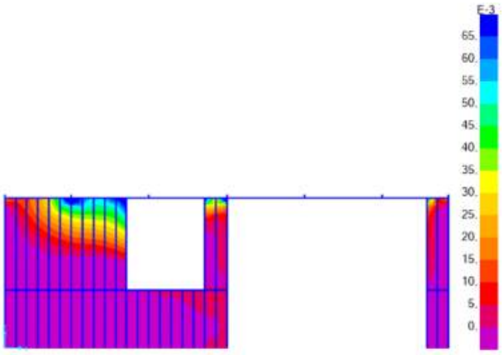


Wall 3 S22 LC 17 (top face)

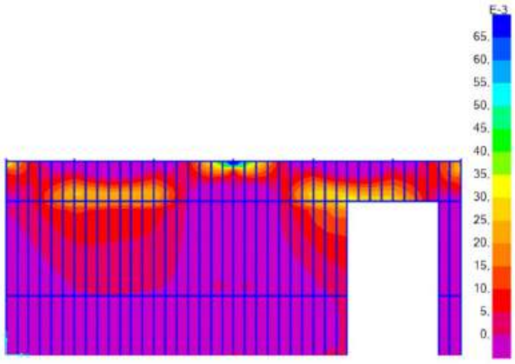


Column 1 S22 LC 17 (bottom face)

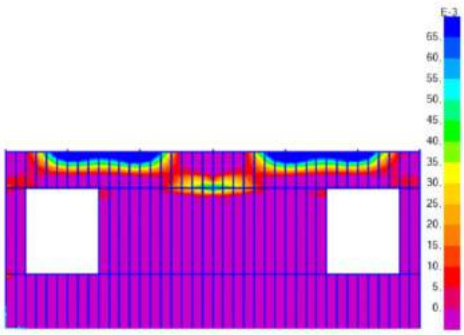
O2 – Tensile stress distribution



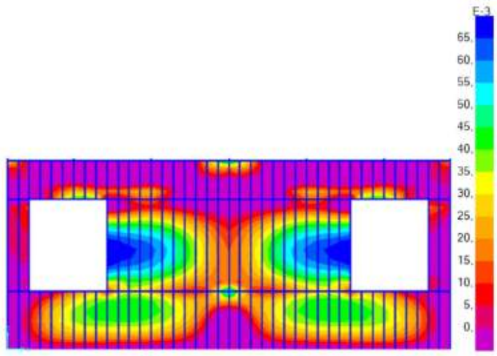
Wall 1 S22 LC 17 (bottom face)



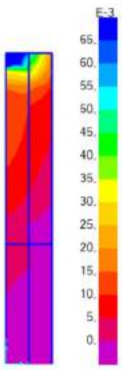
Wall 2 S22 LC 17 (top face)



Wall 3 S22 LC 17 (top face)

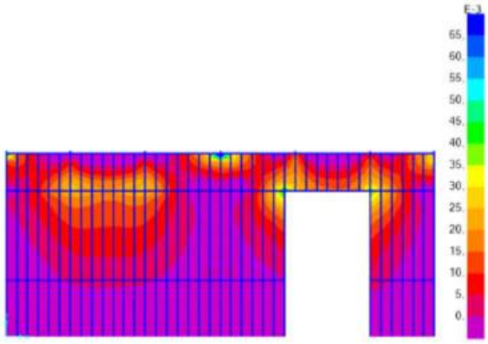


Wall 3 S22 LC 17 (bottom face)

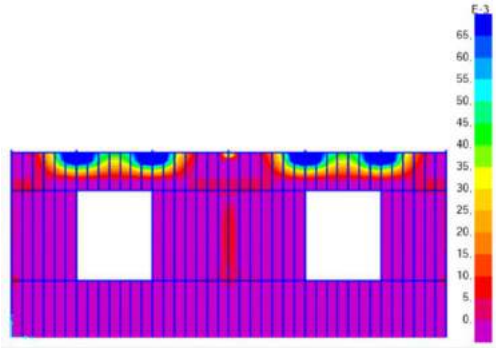


Column 1 S22 LC 17 (bottom face)

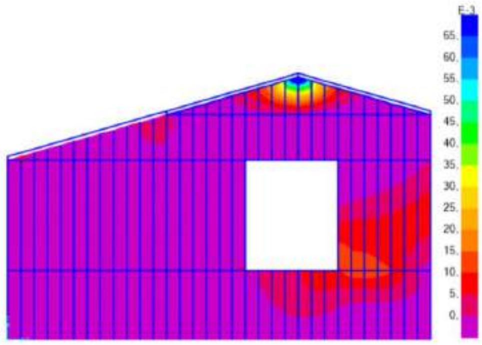
C2 – Tensile stress distribution



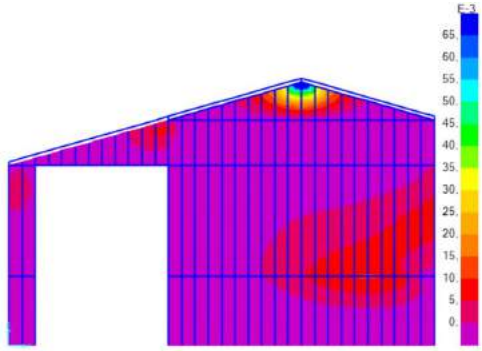
Wall 2 S22 LC 17 (top face)



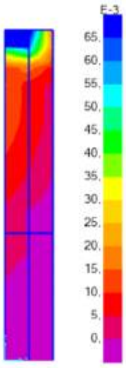
Wall 3 S22 LC 17 (top face)



Wall 4 S22 LC 17 (top face)

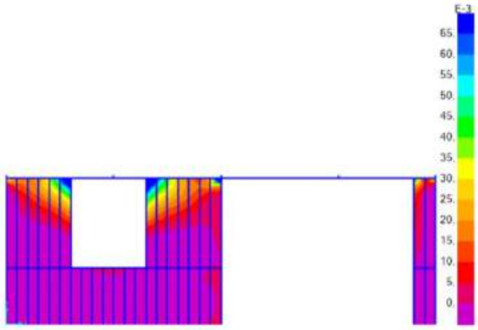


Wall 6 S22 LC 17 (bottom face)

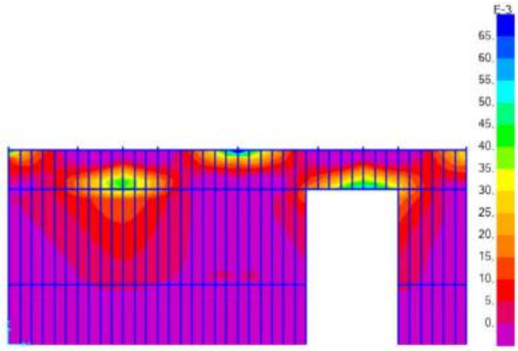


Column 1 S22 LC 17 (bottom face)

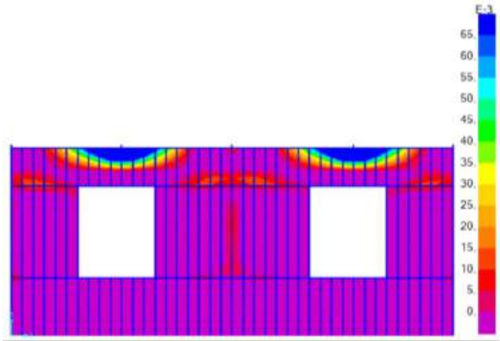
RO4 – Tensile stress distribution



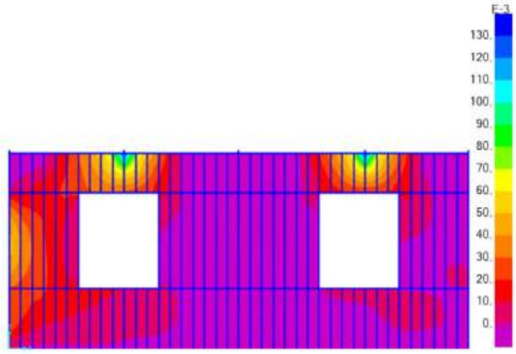
Wall 1 S22 LC 17 (bottom face)



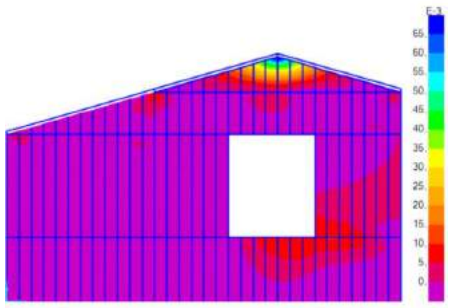
Wall 2 S22 LC 17 (top face)



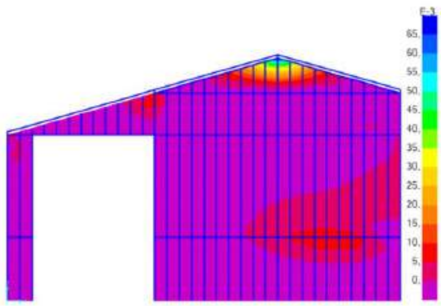
Wall 3 S22 LC 16 (top face)



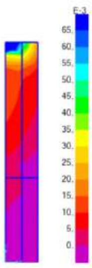
Wall 3 S11 LC 16 (top face)



Wall 4 S22 LC 17 (top face)

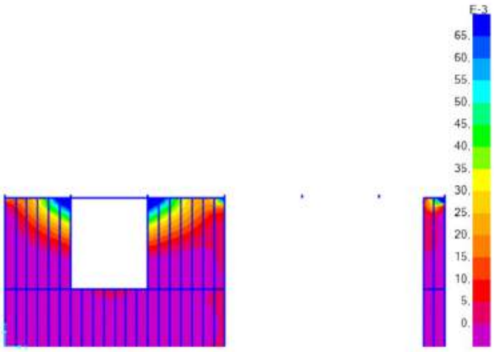


Wall 6 S22 LC 17 (bottom face)

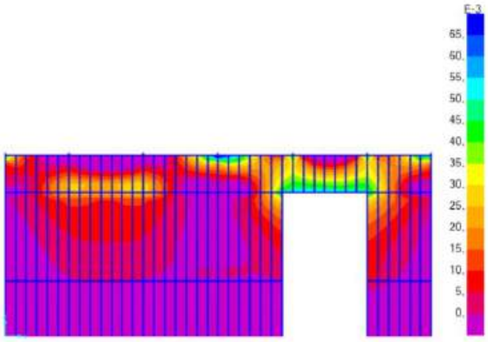


Column 1 S22 LC 17 (bottom face)

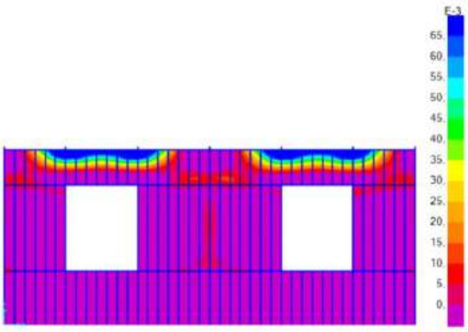
X1 – Tensile stress distribution



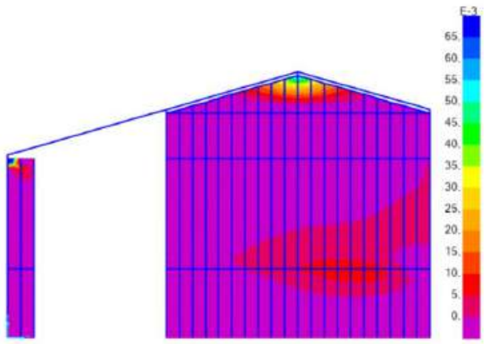
Wall 1 S22 LC 17 (bottom face)



Wall 2 S22 LC 17 (top face)

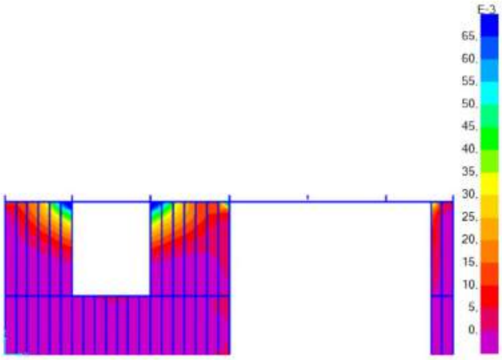


Wall 3 S22 LC 17 (top face)

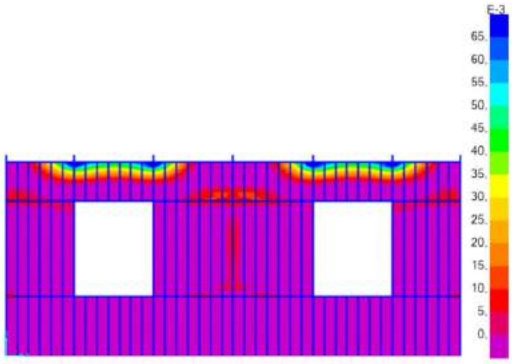


Wall 6 S22 LC 17 (bottom face)

X2 – Tensile stress distribution

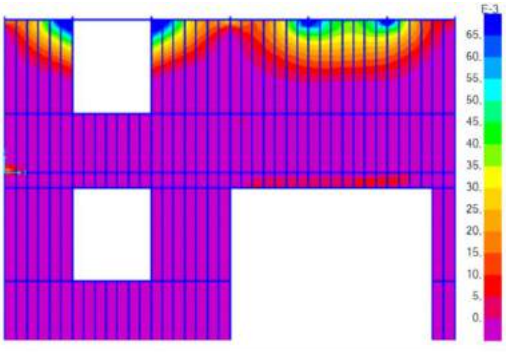


Wall 1 S22 LC 17 (bottom face)

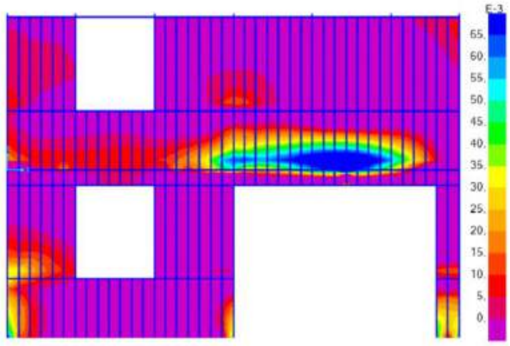


Wall 3 S22 LC 17 (top face)

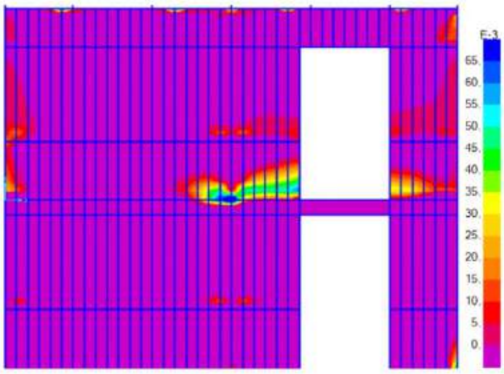
X3 – Tensile stress distribution



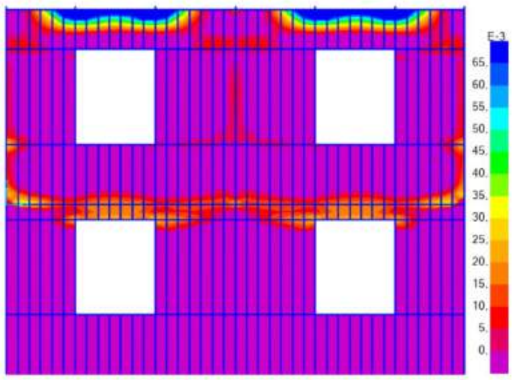
Wall 1 S22 LC 17 (bottom face)



Wall 1 S22 LC 11a (top face)

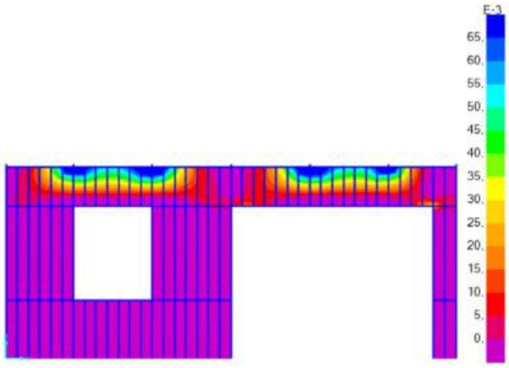


Wall 2 S22 LC 11a (top face)

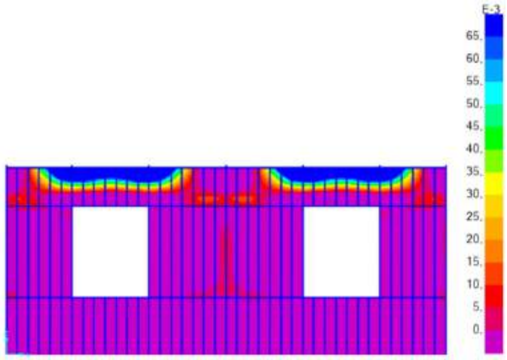


Wall 3 S22 LC 17 (top face)

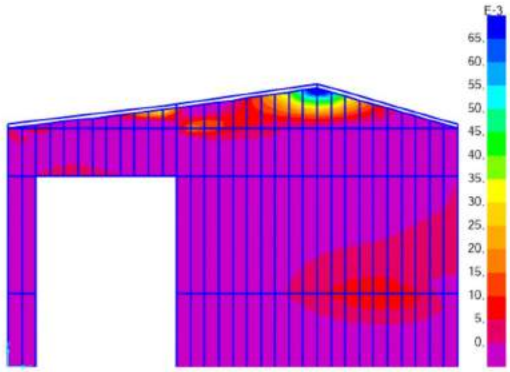
X5 – Tensile stress distribution



Wall 1 S22 LC 17 (bottom face)

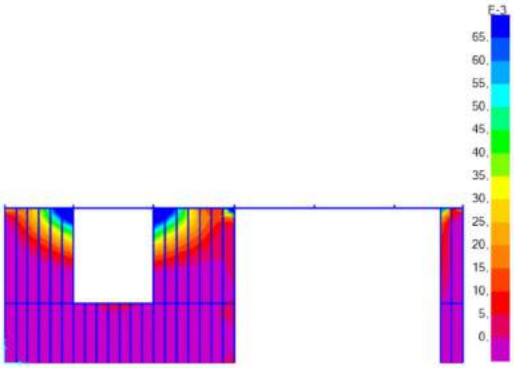


Wall 3 S22 LC 17 (top face)

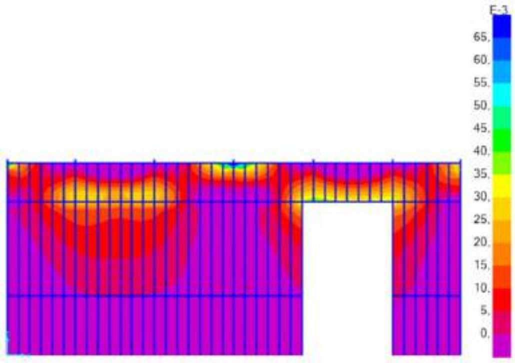


Wall 6 S22 LC 17 (bottom face)

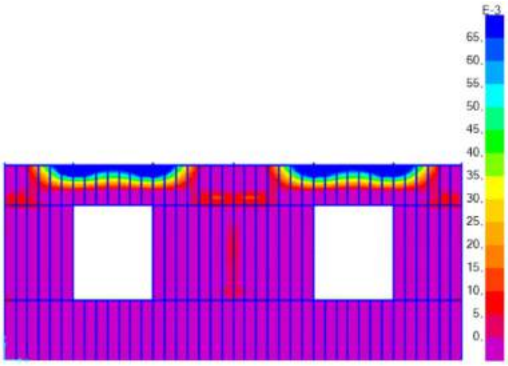
Q2-10MPa – Tensile stress distribution



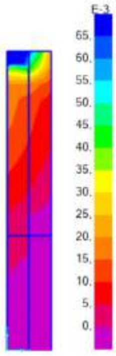
Wall 1 S22 LC 17 (bottom face)



Wall 2 S22 LC 17 (top face)

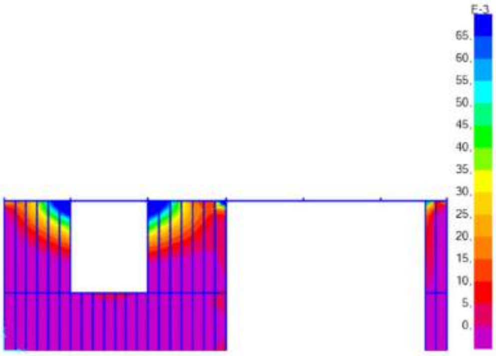


Wall 3 S22 LC 17 (top face)

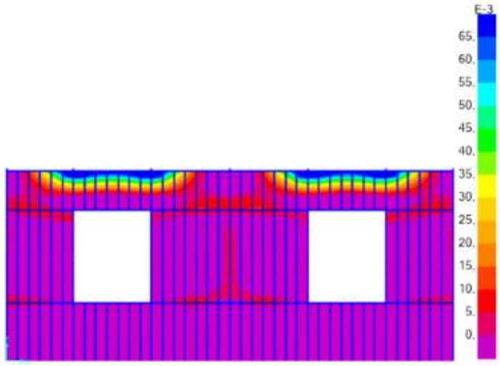


Column 1 S22 LC 17 (bottom face)

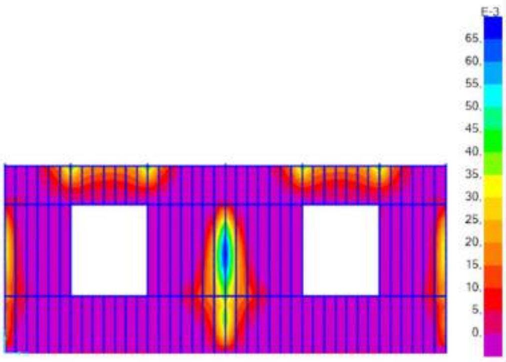
Q2-2MPa – Tensile stress distribution



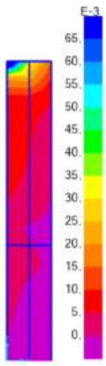
Wall 1 S22 LC 17 (bottom face)



Wall 3 S22 LC 17 (top face)



Wall 3 S11 LC 15 (top face)



Column 1 S22 LC 17 (bottom face)