Structural Analysis of Vernacular Architecture in Indian Himalayas

A DISSERTATION

Submitted in partial fulfillment of the requirements for the award of the degree of

MASTER OF ARCHITECTURE

By

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ABSTRACT

Himalayan region in India is home to 44 million people (source: census 2011) and a variety of flora and fauna. Due to its tough terrain and climatological conditions a lot of indigenous construction practices are prevalent in different parts. At the same time it is under the threat of various natural calamities like earthquakes, landslides and cloudbursts. The whole Himalayan region is under zone-5 in earthquake hazard zoning. Despite being susceptible to such calamity no proper structural analysis of the various construction techniques prevalent has been ever done. This analysis will contribute in risk assessment of the area, in providing model solutions and will help in incorporating the design patterns extracted from traditional architecture to contemporary.

The research is purely an applied one and seeks the following Objectives:

- To understand the concept of structural analysis and its various methods through literature study.
- Identify the various building typologies prevalent in the region.
- Study of the Force-Displacement characteristics of the identified typologies.
- Identification of structural drawbacks with reference to present day scenario.
- Providing model solutions by developing prototype and proposing retrofitting measures for each region.

INTRODUCTION

Significance

Aim

Goals and Objectives

Scope of Research

Methodology

Process of Research

1 Introduction

1.1 Significance

Himalayan region in India is home to 44 million people (source: census 2011) and a variety of flora and fauna. Due to its tough terrain and climatological conditions a lot of indigenous construction practices are prevalent in different parts. At the same time it is under the threat of various natural calamities like earthquakes, landslides and cloudbursts. The whole Himalayan region is under zone-5 in earthquake hazard zoning. Despite being susceptible to such calamity no proper structural analysis of the various construction techniques prevalent has been ever done. This analysis will contribute in risk assessment of the area, in providing model solutions and will help in incorporating the design patterns extracted from traditional architecture to contemporary.

1.2 Aim

To structurally analyze the vernacular architectural practices of Indian Himalayas.

1.3 Goals and Objectives

The research is purely an applied one and seeks the following objectives:

- To understand the concept of structural analysis and its various methods through literature study.
- Identify the various building typologies prevalent in the region.
- Study of the Force-Displacement characteristics of the identified typologies.
- Identification of structural drawbacks with reference to present day scenario.
- Providing model solutions by developing prototype and proposing retrofitting measures for each region.

1.4 Scope of Research

- This study will go into detail of only two (Thathra and Taq Dewari) architectural styles present in the study region.
- Structural analysis of each typology and testing of model solution will be done on virtual model.
- This study will be analyzed with samples of each typology.

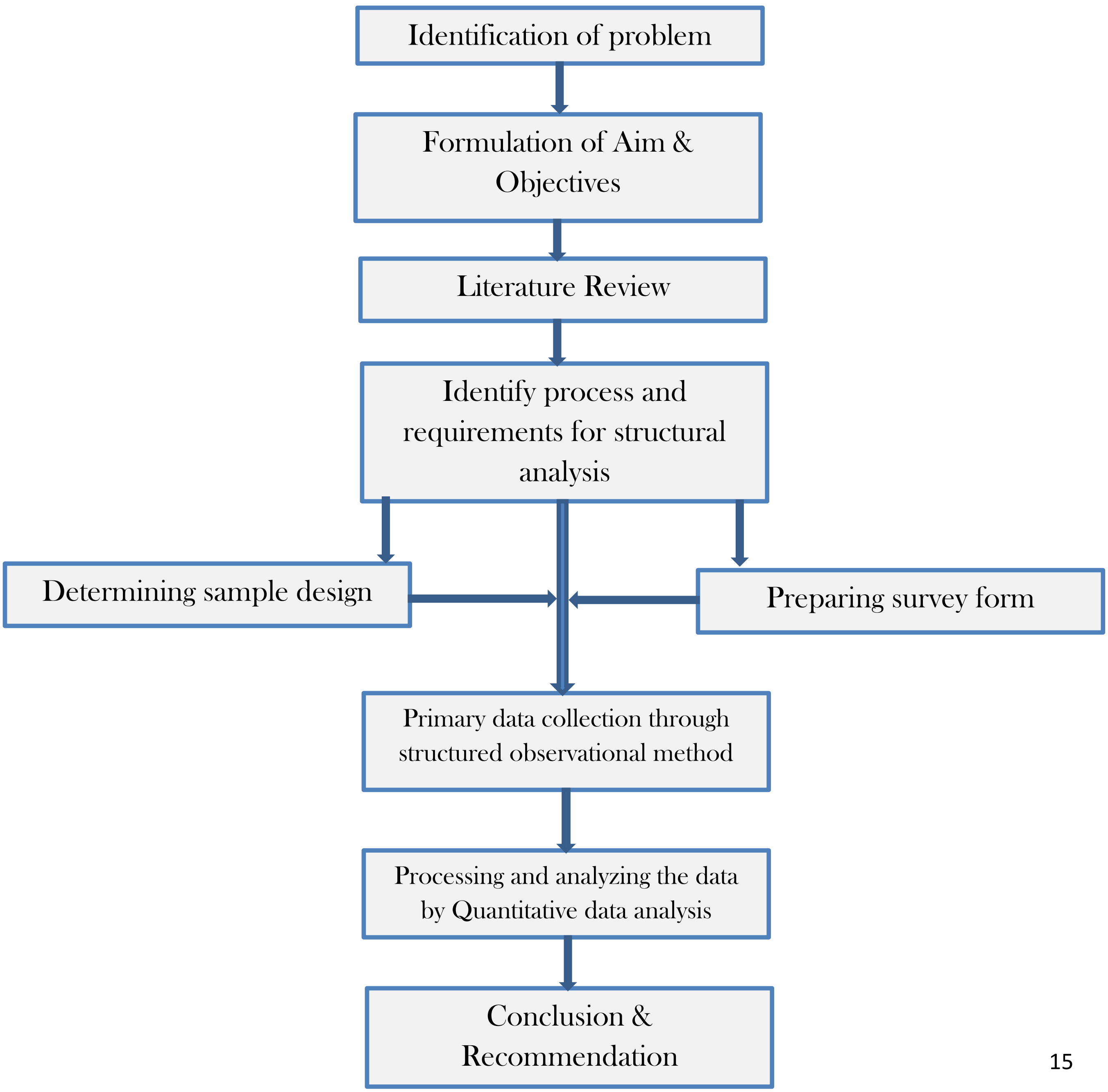
- No structural testing on life size scale will be done and the results of analysis of virtual model will be used.

1.5 Methodology

As this research can be categorized as descriptive applied research I have adopted the structured approach to inquiry.

1.6 Process of Research

Following is the research process being applied:



LITERATURE REVIEW

2 Literature review

2.1 Introduction:

This research can be broadly classified into following 3 parts:

- Descriptive identification of the building typology
- Structural analysis of the said typology
- Retrofitting measures for better performance

The main aim of this literature study is to understand, what structural analysis is and how to perform it.

2.2 Loads:

The loads to consider during structural analysis can be divided into two categories:

- Gravity Load
- Lateral Load

Gravity Load:

The loads that are governed by gravity are called gravity loads. It can be further classified into following two categories:

- Dead Load:
- Live Load:

Lateral Load:

The loads that act in lateral (Horizontal) direction on a structure come under this category.

- Earthquake Load
- Wind Load

According to IS 875 part 3 the wind load, F, acting in a direction normal to the individual structural element or cladding unit is:

F = (Cpe – Cpi) X A X pd

where

Cpe = external pressure coefficient,

Cpi = internal pressure coefficient

A = surface area of structural element or cladding unit, and

pd = design wind pressure

2.3 Structures:

Structures are classified in two parts with regards to the number of support reactions.

- Isostatic Structures
- Hyperstatic Structures

2.3.1 Isostatic Structures:

A structure with a number of external supports (or internal restrictions) equal to the minimum necessary to be in equilibrium for any set of acting loads (e.g. 2D structure with 3 supports). The distribution of forces and moments can be obtained through conditions of static equilibrium only.

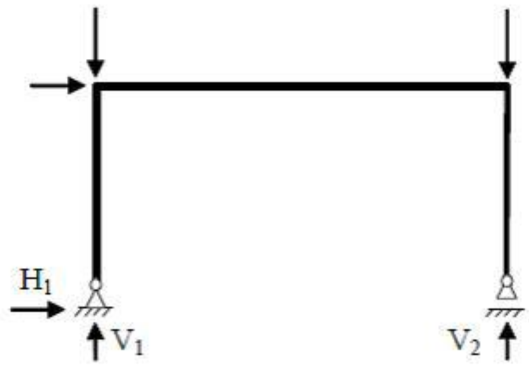


Figure 2: Statically determinated frame

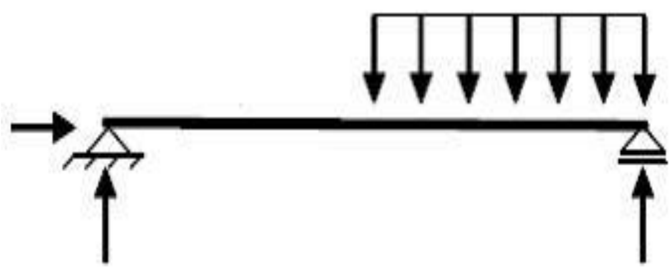


Figure 1: Statically determinated beam

2.3.2 Hyperstatic Structures:

A structure with more than the minimum number of external supports (or internal restrictions) necessary to be in equilibrium for any set of acting loads. In these structures, beyond the static conditions, conditions of compatibility of deformations must be used.

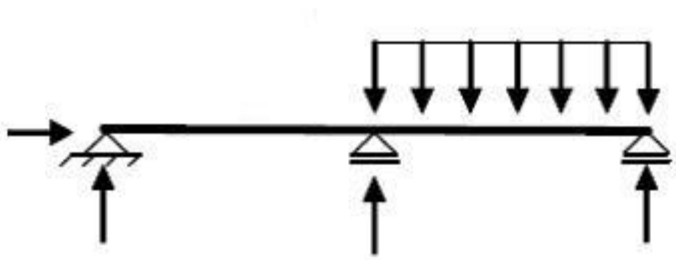


Figure 3: 2D indeterminate frame DOF=3

Figure 4: Indeterminate beam DOF=1

2.4 Structural Analysis: a brief history

There are many definitions of structural analysis:

"The structural analysis is a mathematical algorithm process by which the response of a structure to specified loads and actions is determined. This response is measured by determining the internal forces or stress resultants and displacements or deformations throughout the structure." (Dr. Mohan Kalani)

Structural analysis is the determination of the effects of loads on physical structures and their components.

Analytical process by which the response of the structures (in terms stresses, internal forces and deformations) to the acting loads is determined.

To summarize, structural analysis is a process which provides a set of rules to accurately model a structure and to proportion its parts so the loads can be carried out safely.

Various methods for structural analysis were evolved gradually over a period of time. To understand better structural analysis we have to go in its history.

Starting from the Renaissance, K.E. Kurrer has divided the history of structural analysis into five periods:

- The Preparation Time from 1575 until 1825,
- The Discipline Creation Period from 1825 until 1900,
- The Consolidation Period from 1900 until 1950,
- The Integration Period from 1950 until 1975 and
- The Diffusion Period from 1975 until now.

2.4.1 The Preparation Time (1575 – 1825)

In his Discorsi (1638) Galileo presented the idea of quality of materials.

Through probes wooden examples he decided without a doubt the quality in strain and thusly explored the quality of a cantilever shaft inserted in a brick work divider under weight toward oneself, watching its conduct with an expanding burden connected at its free end. In spite of the fact that the figured bowing quality was off base (because of the suspicion of a direct stretch circulation in the area at the fixed end of the shaft) Galileo arrived at the right conclusion that the quality of a rectangular bar is corresponding to the width and to the square of the stature of its segment. Through his work Galileo made a significant commitment to the bowing issue.

Numerous specialists researched the inquiry of the quality of the material, beginning from the theory of Galileo.

While Galileo was chiefly concerned with the quality of the material, Robert Hook (1629-1695) in England created the hypothesis of flexibility.

In the same period Simon Stevin (1548-1620) managed the issue of the disintegration of the power.

In these years the power of geometry and the different advancement of statics, quality of the material and flexibility hypotheses did not take into consideration a fitting examination of structural components. In the eighteenth century infinitesimal analytics discovered requisition in cosmology, hypothetical mechanics, geodesy and civil designing. Mathematicians like Leibniz (1646- 1716), Daniel and Jacob Bernoulli and Leonard Euler (1707-1783) made further advancement in the hypothesis of pillars and their versatile line. In 1791 Jacob Bernoulli uncovered that the bend of a shaft in unadulterated curving is relative to the estimation of the bowing minute.

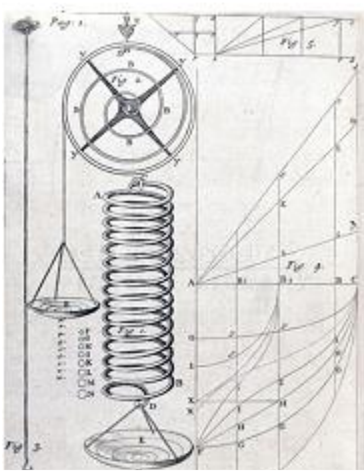


Figure 6 Hookes spring

Figure 5 Galileos cantilever beam embedded in a masonry wall.

Daniel Bernoulli, 50 years after the fact, rearranged extensively the science of his sibling's hypothesis, providing for it a "designing" methodology. In the first 50% of the eighteenth century the first building schools focused around the requisition of the microscopic math to specialized items emerged in France. In 1729 Bernard Forest de B`elidor distributed the first standard Code of Practice for Civil Engineering. In this book the discoveries of science are made into standards of outline where math discovered a down to earth requisition.

Fifty years after the fact, Charles Augustin de Coloumbs (1736-1806) in his work M'emoire broke down the earth push, the curve and the bar through tiny analytics in a moderately clear manner which made his work more decipherable and more fruitful than different past productions. In this book structural dissection shows surprisingly the aspects of a logical subject.

2.4.2 The Discipline Creation Period (1825-1900)

In the Discipline Creation Period the isolated discoveries of the Preparation Time were integrated with the elasticity theories developed in France in the first half of the 19th century.

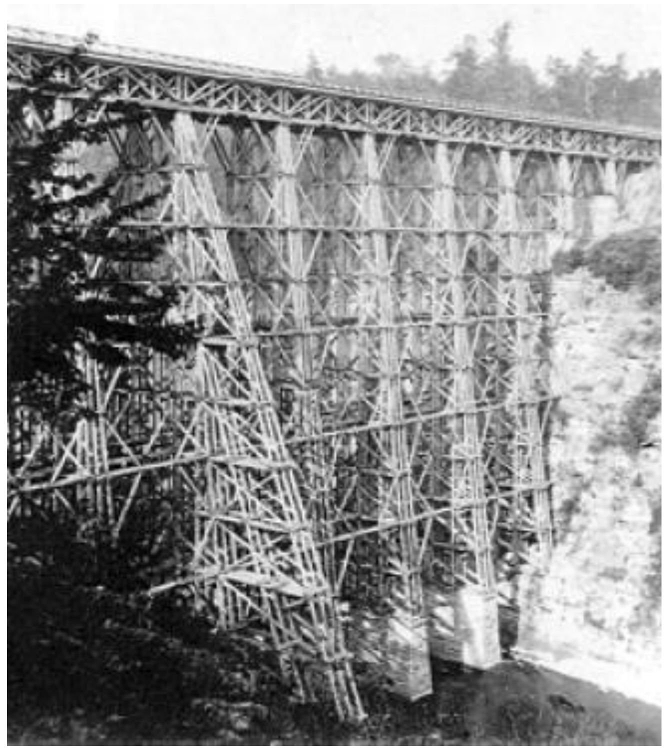


Figure 7: Truss Bridge

Henry Navier (1785-1836), educator at the Ecole des Ponts et Chauss'e, was additionally concerned with the quality of materials, examining various wood and steel developments. His thought was that a definitive enthusiasm of a specialist is to guarantee the security of a structure under particular burdens, not to examine the conduct of a structure close fall. To this end, an architect must figure the hassles in a stacked structure, to guarantee that they are beneath the flexible furthest reaches of the materials. While balance comparisons are sufficient to focus inside and outside powers in basic structures, the statically determinate or isostatic ones, they can't be utilized alone to break down hyperstatic structures. This issue, which soon turned into a critical issue in the field of the hypothesis of structures, was mulled over by Navier, who created a hypothesis to handle the second sort of structure. All the experimental mechanical assembly was accessible for this examination: Jacob Bernoulli had expressed that the versatile shape of the bar was corresponding to the estimation of the curving minute at each one point, and Daniel Bernoulli had demonstrated to set up the differential mathematical statement of flexible twisting to focus the redirected state of a pillar. Navier utilized the versatile mathematical statement joined together with the harmony comparison to examine a cantilever bar backed by an inflexible prop at its end, which is a hyperstatic structure and after that, summing up this basic case, he created the straight flexible hypothesis, treated in his book Resum'e des Lecons (1826). With this work Navier demonstrated a logical approach to model and numerically dissect a structure utilizing straightforward count apparatuses and material properties. Emulating the techniques managed in Navier's book, a specialist could, on a fundamental level, enhance its model and fabricate savvy structures equipped for securely convey the connected burdens.

Navier was also concerned with hyperstatic trusses, which were of energy at the nineteenth century on account of the advancement of steel augmentations. Notwithstanding the way that his system for grasping trusses was correct, the resulting figurings were prohibitively befuddled due to the gigantic number of correlations to be unwound.

In the second 50% of the nineteenth century, strategies were concocted to diminish the extent of the issue, among them some focused around design. The technique's most critical example was the Swiss educator Karl Culmann (1821-1881), who created the truss hypothesis and refined graphical dissection strategies. This hypothesis was fundamental for the development of steel truss connects in the second a large portion of the nineteenth century.

Culmann's system was valuable for isostatic trusses yet less fitting for hyperstatic ones. In the late nineteenth century Maxwell, Castigliano, Otto Mohr and M¨uller-Breslau further created the straight flexible hypothesis of pillar structures with the point of improving the complex numerical count required for hyperstatic structures. From Maxwell's work comes the proportional hypothesis, which evaded a portion of the complex scientific counts in the result of hyperstatic trusses. Castigliano formed the hypothesis which bears his name. Despite the fact that the dissection could be improved utilizing Castigliano's system, the amount of comparisons to be illuminated stayed extensive. The energy system created by M¨uller-Breslau (1851-1925) focused around the standard of virtual strengths permitted one to decrease the amount of mathematical statements to be tackled for hyperstatic structure.

2.4.3 Consolidation Period (1900-1950)

Figure 8: Heinz Isler: Shell model of office building roof, Burgdorf, Switzerland, 1965.

This period was portrayed by the spread of strengthened solid. Its innovation in Germany in the second 50% of the eighteenth century spoke to an insurgency in the field of development and in the meantime had an effect on structural examination, prompting the advancement of new hypotheses. Since 1915 the hypothesis of casing structures and by most accounts 10 years after the fact, the hypothesis of two dimensional structures like plates, shells and collapsed structures were produced. The misshapening strategy for hyperstatic structures supplanted, in a few cases, the power technique. In the 1920s with the expanded number of elevated structures, the misshapening system was upgraded by the cycle strategy for Hardy Cross (1930), more suitable for high-review statically uncertain structures.

By the 1940s, notwithstanding the utilization of cement and hot-moved steel areas, icy shaped steel parts started to be broadly utilized within building development, especially in top decks, carpet decks and divider boards. Steel top decks were effectively utilized as a part of collapsed plate and hyperbolic-illustrative top development. Contrasted with different materials, for example, solid, icy framed steel parts had numerous qualities: daintiness, high quality and firmness, simple erection, transportation and taking care of, the consolidation of which frequently brought about expense sparing. In any case, the slim nature of such structures causes nearby and worldwide clasping phenomena, which are of real sympathy toward outline. Albeit moved steel parts have been utilized as a part of development since the second 50% of the nineteenth century, just from 1940 were locking issues researched firstly in the USA and later in different nations.

In the same period a definitive quality of structures, specifically hyperstatic ones, turned into the destination of investigation and this prompted the improvement of the versatility hypothesis. The principal papers on pliancy hypothesis go once more to a congress in Berlin in 1936, which was held in perspective of the developing utilization of steel for modern and substantial business and household structures. So far they had been outlined on the premise of the flexible hypothesis. This is focused around the suspicion of an immaculate structure where little slips in assembling and development, temperature variety, settlements of backings, and so on are frequently enough to discredit the flexible computation. The conclusion was that the flexible hassles are not generally pertinent to the expectation of the quality of a structure. Jon Backer, a master part of the Steel Structures Research Committee set up by the British steel industry in 1929 was the first to have the "thought" of plastic outline. He worked primarily on nonstop shafts and entrance outlines. Benefactor's methodology to plastic examination was of a statical nature: the safety minute acting where plastic pivots have formed is brought into the statics comparisons. Along these lines the breakdown instrument of decently basic structures could be estimate exactly.

In 1949 Prager and his partners created the basic hypotheses of versatility hypothesis. The purported lower-bound hypothesis, in any case, had as of now been expressed and demonstrated in 1936 by the Russian researcher Gvozdev. As per his hypothesis, if a set of strengths inside the structure are in balance with the outer loads and don't defile the yield conditions, then the relating estimation of the heap following up on the structure is a lower-bound, i.e. safe evaluation of the breakdown load. The hypothesis clarifies why the flexible hypothesis brings about a safe, however frequently uneconomical configuration. All the more vitally, pliancy hypothesis made specialists consider the genuine conduct of a structure under breakdown loads. The trial work of this century has indicated that obscure and unusual "defects" – a little settlement in an establishment, a slip in an association, a differential climb in temperature - can deliver extensive changes in the real stretch state so that the estimations made to discover this state are potentially inconsistent. It has been an imperative commitment of pliancy hypothesis to demonstrate that blemishes have, truth be told, no impact on a definitive quality of bendable structures. The hypothesis was created to a great extent with reference to steel development yet can likewise be connected to cement structures strengthened with pliable steel.

2.4.4 Integration Period (1950-1975)

Until the mid 20st century, in spite of the utilization of streamlined figuring routines like the energy system, the uprooting strategy and the Hardy-Cross technique, it took quite a while to investigate structures even of medium intricacy, basically because of the trouble of comprehending direct comparison frameworks. In the late 1950s the approach of machines and the improvement of the Finite Element Method (FEM) totally reformed structural examination. The FEM was produced in the field of flying designing, where counts of high excess frameworks were the standard. The makers of FEM characterized it as a "system for investigation for exceedingly excess structures which is especially suited to the utilization of highspeed computerized processing machines" [3]. The primary persons concerned with the beginning improvement of the FEM were Argyris (Stuttgart and London), Clough (Berkeley) and Zienkiewicz (Swansea). The principal FEM projects SAP, ADINA, ANSYS, NASTRAN, MARC, and so on., were utilized just by experts within enormous organizations and figuring focuses which could exertion exorbitant centralized computer machines. They were extensive, abate and with no graphical client interface. The FEM consolidated with the workstation has prompted present day computational mechanics, of which structural examination is a part. Through advanced registering, non-paltry computations concerning progress, breakdown instruments, materials and geometrical nonlinearities and extreme burdens could likewise be routinely performed.

2.4.5 Diffusion Period (1975 until now)

While the FEM hypothesis has been to a huge degree created in the 60s and early 70s, the right to gain entrance to suitable equipment stayed troublesome. Be that as it may, with the appearance of the PC in the 80s and the Internet in the 90s, the work identified with structural examination has been completely reformed. Today, on account of improvements in machine design, a substantial mixed bag of easy to use FEM workstation projects are effectively accessible, which in for all intents and purpose no time can any sort of estimation. A structural designer has now only to enter a suitable structural model into a FE program and detail the heaps to be conveyed. The workstation will then promptly give the interior structural powers and the comparing burdens. In the event that the structure does not fulfill the security criteria, the workstation can, in a few cases, itself roll out improvements to the outline until wellbeing criteria have been satisfied.

The cordiality of today's structural examination projects could prompt the wrong conviction that no particular faculty is obliged to work them. Actually, encounter demonstrates that the client of a FEM bundle must have the capacity to legitimately display a structure, to know the major hypotheses of FEM and have an inclination for structural conduct to appropriately evaluate the legitimacy of the machine results. Without these key necessities, unsafe slips can undoubtedly happen.

There are many approaches prevalent for structural analysis nowadays. All of them make some assumptions and deliver results with varying degree of accuracy, but each of them analysis the same fundamental relationships:

- Equilibrium
- Compatibility
- Constitutive

2.5 Finite Element Method

This is a numerical method of solving differential equations, which is used in all engineering mechanical calculations. The total region, which is supposed to be analyzed, is divided into small elements and each element is then solved with the method. When the differential equations of each element are solved they are put together in a so called finite element mesh, which shows the solution over the whole body. This means that even though the method is an approximation, the result can be very close to the reality. The smaller elements - the better approximation over the whole body.

The FEM works as follows:

1. The structure to be analyzed is first subdivided into finite elements of simple geo- metric shape, e.g. straight prismatic bars. The elements are joined to each other or supported at the nodes.

2. At each node displacement parameters are introduced which represent the primary unknowns to be numerically determined. In the case of plane frames two displace- ments in the horizontal

and vertical directions and the nodal rotation are considered, whereas in the case of plane trusses only the rotation is left out.

3. In order to determine these unknowns, nodal equilibrium conditions are formulated as linear equations. Nodal equilibrium is satisfied when the resultants of all the forces in the horizontal direction, of all forces in the vertical direction and, in the case of plane frames, of all moments acting on each node vanish.

4. The fundamental idea of the FEM is to determine the numerical coefficients needed to formulate nodal equilibrium conditions by considering each element separately. These coefficients are first "locally" determined for each single finite element taking into account its geometry, material properties and loads.

5. From these local element coefficients a "global" equation system is then assembled whose equations represent nodal equilibrium conditions for the unknown nodal dis- placement parameters.

6. This global system of linear equations is solved numerically so as to determine all nodal displacements. In two cases eigenvalue problems, which arise from the formulation of the same kind of nodal equilibrium conditions, are to be solved.

7. Once the nodal displacements are known, secondary quantities, like section forces or support reactions, are determined.

2.6 Summary:

After analyzing the whole literature following conclusions have been made.

The structural analysis should take into account following effects:

- deformability and stiffness of the structure and supports;
- strength and stiffness of joints;
- stability of the structure (global, members and local);

- behavior of cross-sections (classification of sections);
- imperfections (global and member imperfections).

Structural analysis should give answers to following questions:

- What are the elastic deformations of the structure caused by the loads?
- What are the section forces associated with these deformations?
- What are the stresses caused by these section forces?
- Can the bars of the structure with their assigned sections withstand these section forces?
- What are the influences of 2nd order effects on structural behaviour and how great can compressive normal forces become before buckling occurs?
- In which cases can time-dependent loads lead to dangerous dynamic effects?
- How far can external loads be increased without the structure's collapsing and how would the structure collapse if the loads reach this limit?

To answer to these questions mechanical models, which approximate physical reality, are needed. The FEM is ideally suited both to build such models and to analyze them numerically.

SITE STUDY

Introduction

Identified Building Typologies

3 Site Study

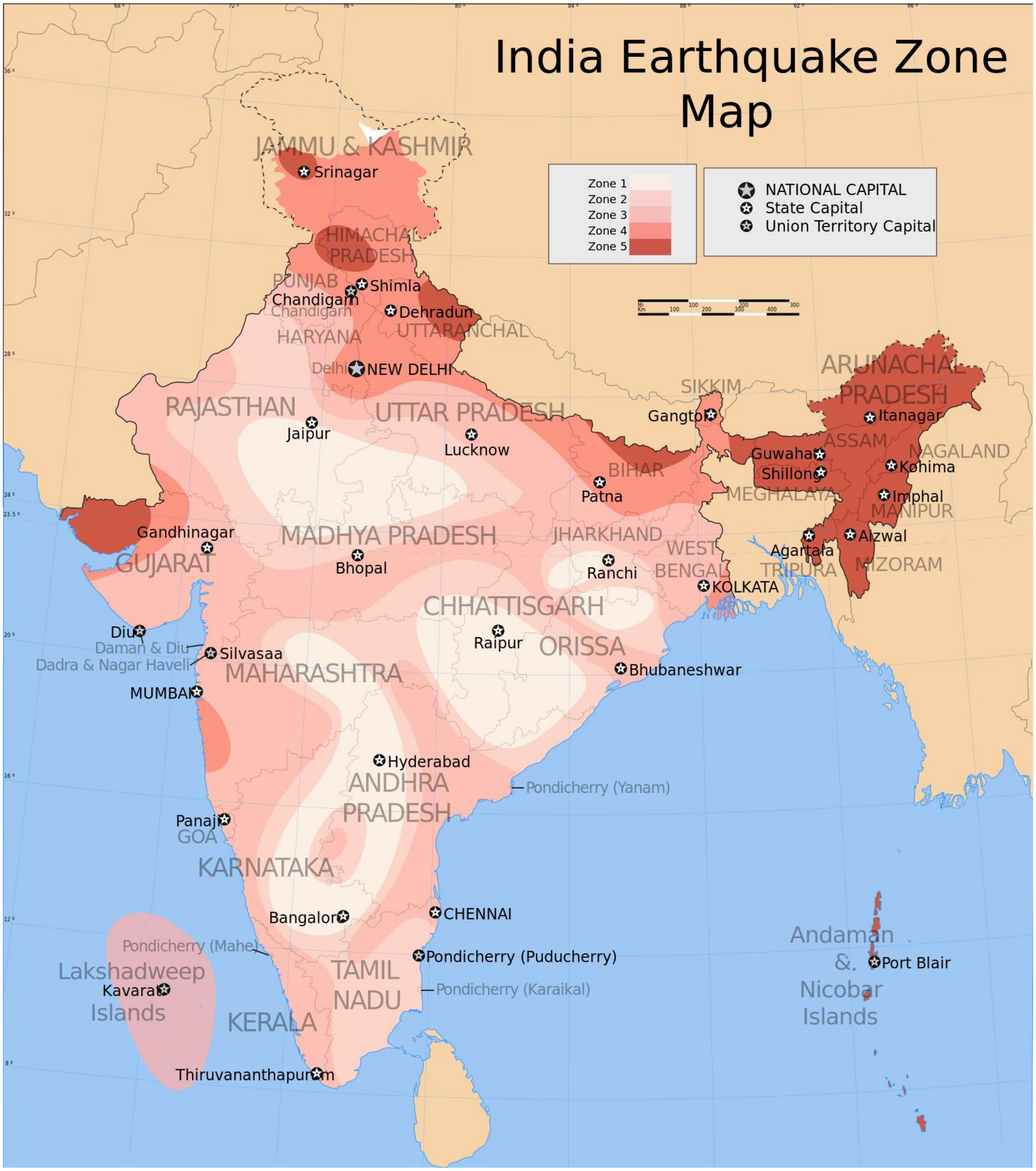


Figure 9: India Earthquake zone map

3.1 Introduction

Himalayan region in India is home to 44 million people (source: census 2011) and a variety of flora and fauna. Due to its tough terrain and climatological conditions a lot of indigenous construction practices are prevalent in different parts. These indigenous architecture is evolved by the people of the region over a period of time through their own knowledge and experiences without any professional help or training.

3.2 Identified Building Typologies

Following are some of the construction styles found in the region:

3.2.1 Thathara Style:

Figure 10: Typical Thathara house

This building style has been identified in Himachal Pradesh, a northern state in India. Nowadays, this type of construction practice can be seen for houses and temples, however, earlier photographs suggest that the same style was adopted to build palaces, bridges as well as various other structures. The construction style is named “Thathara” as this term is locally used for wooden planks that make the vertical load-carrying members (columns) locally known as thola(s). Tholas (a peculiar combination of timber and stone) and wood are primarily used for the vertical and horizontal frame elements, respectively. The region where this building typology is found is characterized by cold climate and witnesses heavy rainfall during the rainy season (from June to

July) as well as snowfalls in winter (from October to March). These effects have been considered well in the construction style, like e.g. small openings, a verandah to take sun but prevent from rain and snow, wooden and mud interiors which are good insulators and keep the interiors warm, sloping roofs with adequate projections as well as other features. Being located in the Himalayan region, the area has experienced numerous strong earthquakes and this construction technique has evolved eventually to withstand seismic action.

The wall system is framed structure in which the columns (tholas) are in 'thathara' style. Beams are of deodar or kail wood, sometimes of the tree trunk itself. The partition walls are a variety of construction types either of stone, wood or both. In some cases it was observed that the walls of the lower storey are entirely made of stone and hence are load bearing, while the upper storey(s) have tholas with partitions of wooden planks. In some constructions tholas can be seen in the upper storey directly above the posts of the verandah which is considered to be a poor construction practice. Thathara houses usually have gable roofs with a slope of about 17 degrees. Over the verandah, the roof slope becomes a bit gradual in order to have adequate headroom.

3.2.2 Mud wall construction:

The style too has been identified in Himachal Pradesh. It is concentrated in the upper reaches of the state in the Lahaul and Spiti districts, which are located in a cold-desert area with very hot days and chilling nights. Precipitation usually only occurs in the form of snowfall with almost no to very little rainfall. This dryness of the local climate is reflected in the

Figure 11: Typical Mud construction

architecture of this construction typology which consists of thick mud walls with small openings in order insulate the interior form the harsh outside climate. This style of construction which is predominantly used for residential houses and temples is still being practiced today.

The houses in Spitian architecture have load bearing wall system. The 300 or 500mm thick mud walls take up the entire load. Walls up to 600 to 900 mm height are made of local available field stones. Above this base, the walls are entirely constructed of rammed earth. As the area has very little or no rainfall, the roofs are flat and constructed by laying horizontal wooden beams which are covered by plaster of mud over reed leaves. To make the parapets, a 300 mm thick and 300 mm high course of mud is laid at the periphery of the terrace. On top of it, dried bushes are packed densely. These bushes are projecting to the outside and hence also act as a sunshade.

The construction of these buildings is the only efficient option in the valley as the main building material mud is easily available and also suits to the harsh weather conditions.

3.2.3 Dry stone construction:

Figure 12: Typical Dry Stone construction

The building type has been identified in Himachal Pradesh and Uttarakhand. Nowadays, this type of construction practice can be seen prevalent in the areas where people have been forced to leave their traditional construction practices due to scarcity of wood. Thus, this construction style is nothing but the traditional housing style while omitting the wooden elements. Be it Kath Kunni style or Thathara style, the reason why people did not opt mud which is another easily available material, is this region's heavy precipitation both in terms of rainfall (June to July) as well as snowfall (October to March).

The stone walls are load bearing with a thickness of 500 mm and a height of 2.5 m. Both external and internal walls are of the same thickness. These are made of locally available undressed stones of varying sizes, packed together without any mortar. Sometimes the size of a single stone is large enough to cover the entire opening as a lintel beam. Kail or deodar wood is used for frames and panels of doors and windows. As the area receives rain in monsoon and heavy snow in winters, the houses have sloping roofs, i.e. gable or hipped roofs. These are covered with slate stones which are locally available while deodar or kail wood is used for beams, rafters and purlins.

Response of this construction technique during earthquake is excellent and construction economics also favours it thus making it a viable option in present day scenario.

3.2.4 Kaath Kuni

Figure 13: Typical Kath Kuni house

Houses made of wood and stone in a peculiar way are a common sight in upper Shimla and Kinnaur region of Himachal Pradesh but this architecture style is also found in Uttarakhand and Kashmir. Houses of this type are still being built today and are being constructed for past hundreds of years. During field visit, houses around 300- 400 years old are also found. The site chosen for house construction is with gentle slope so that it can be leveled easily while laying plinth stones and hence no cut/ fill is required. In rare cases, to make more room on ground, site is cut but again there is no filling. Locally available stone is used for foundation. Trench is dug about 900mm in depth and 900mm in width. This trench is then filled with large slabs (dry stones) with approximate width of 900mm. In some cases where the ground is hard enough, no trenches are dug for foundation. There is no plinth in these houses. The walls have framed-type construction and are of thickness 600mm.These are made up of stones and wood of deodar ,rai and perman trees without any mortar. Plaster is a mixture of mud, cow-dung and kail wood powder or wheat husk.

For construction of these walls frame-work of wooden beams (150x200mm) is made which is then packed with stones.To make the frame, two wooden beams (150x200mm) are laid side-by-side with a gap in between, defining the wall width i.e. 600mm. This is done for two opposite walls. The same is done for the upper course but for other two walls. The arrangement is repeated until it reaches the wall height.

3.2.5 Taq Dewari:

Taq timber development a blending of wood and unreinforced workmanship laid on frail mortar gives the building the obliged adaptability and utilization conventional construction modeling and material. Taq development is a heading divider stone work development with level timber binding implanted in the workmanship; it is generally arranged with a secluded design of brick work wharfs and window bayous entwined with stepping stool like development of even timber installed in the workmanship at each one story level.

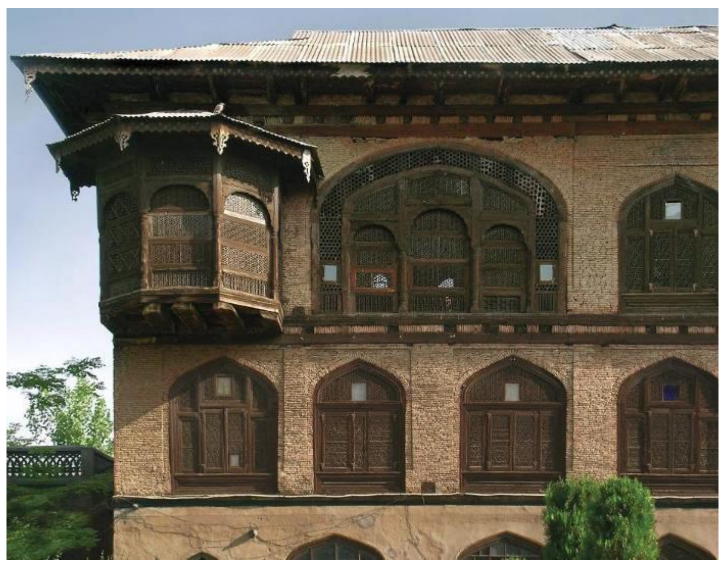


Figure 14: Typical Taq Dewari construction

These even "step groups" are found at the base of the structure to the particular design of the docks and window bayous, i.e. a five-taq house is five inlets wide. The brick work over the establishment (das or dassa), and at each one story level and at the window lintel level Taq alludes wharfs (tshun) are practically constantly 1½-2 feet (45-60 cm) square, and the narrows are roughly 3-4 feet (90-120cm) in width. Since this measured wharf and straight outline and the timber-bound burden bearing stone work dock and divider framework go together, the name now distinguish the structural framework.

Taqsystem does not comprise of complete edges rather has bigger timber runners resting along the heap bearing workmanship dividers with floor bars and runners from the weight dividers lapping over them.

The development practices utilized for these structures, which remained rather than today's codes and acknowledged practices, incorporate (1) the utilization of mortar of unimportant quality, (2) the absence of any holding between the infill dividers and the docks, (3) the shortcoming of the security between the normally acknowledged wythes of the brick work in the dividers, and (4) the regular utilization of substantial turfs.

3.2.6 Dhajji Dewari:

Dhajji dewari construction is profoundly found in Jammu and Kashmir, Himachal Pradesh and Uttarakhand. This construction technique is basically a patchwork of wood and stone. Walls are made of timber frame which are filled with criss crossed wooden panels. These are further filled with stones. These structures are earthquake resistant as each small panel is an identity on its own thus dividing the earthquake load evenly over the whole structure. The friction between all the small elements breaks the energy.

Figure 15: Typical Dhajji dewari framing

CASE STUDY – THATHRA ARCHITECTURAL STYLE

4 Case Study – Thathra architectural style

4.1 Background


Figure 16: Thathra house in Chamba

The Thathara Houses are the traditional houses dating back to 400 years found in upper reaches of Chamba district of Himachal Pradesh. The construction style is named "Thathara" as this term is locally used for wooden planks that make the vertical load-carrying members (columns) locally known as thola(s). Tholas (a peculiar combination of timber and stone) and wood are primarily used for the vertical and horizontal frame elements, respectively. The region where this building typology is found is characterized by cold climate and witnesses heavy rainfall during the rainy season (from June to July) as well as snowfalls in winter (from October to March. Plan of this architecture style is usually rectangular in shape with the sides being 3 to 10 meters and 5 to 8 meters. The structure is usually 2 to 3 storey(s) with each storey being 2.5 meters.

4.2 Foundation

Foundation is entirely made up of stones. Trench of 1to1.5m depth, depending upon the type of soil, and 500mm in width is dug and courses of stones are laid without any mortar, which rises up to 500mm above ground level.

Figure 17: Stone foundation rising above ground level, in a typical thathara house - village Rohta, Distt. Chamba (H.P.)

4.3 Wall Typology

4.3.1 *WALL SYSTEM:*

The wall system is framed structure in which the columns are in *"thathara"* style and beams are of deodar or kail wood, sometimes the tree trunk itself. Infill walls are either of stone, wood or both. In some cases it is seen that walls of lower storey are entirely of stone and hence load bearing but upper storeys have thatharas.

Thathara

A thathara is the vertical load carrying member in the Gaddi houses. It is made up of stone and wood and can be constructed in any of the following two ways:

Method I:

Unfinished wooden planks generally of size 500x350x100mm are placed on edge on two sides at a distance of 400mm. In the alternate course, planks are placed across. Same arrangement is repeated till about 2.5m (height of storey) thus forming a hollow box-like structure. This hollow structure is then hand-packed with stones without any mortar. The thatharas thus formed have unfinished appearance.

Figure 18: Hollow box-like frame for thathara

Method II:

Another way of constructing thathara is laying wooden planks and stones at the same time over a single course. There is no mortar but stones in courses are tightened at their place with stone chips. The

Figure 19: Thathara with stone and planks laid together

wooden planks are also hewn in such a case. Hence the overall thathara has very neat finish.

Connection between planks in a thathara:

The wooden planks used for thathara have small holes which have wooden pins inserted into them so that planks do not move and retain their position. Another way is, having mortise and tennon joint between two planks placed in alternate course.

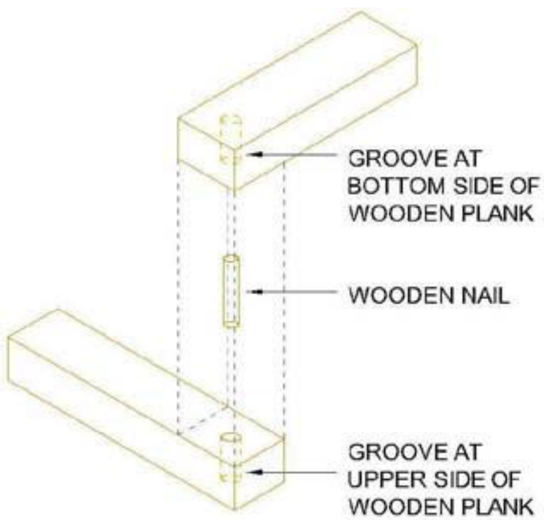


Figure 21: Joint detail of two planks in a thathara using wooden pins

Figure 20: Mortise and tennon joint in planks of thathara

4.3.2 Construction Material:

The main materials used in the construction of walls are deodar or kail wood, locally available stone and mud.

4.3.3 Construction methodology of walls:

Step I: Above the raised plinth of stones "Thatharas" (columns) are constructed at the corners. In case of larger spans intermediate "Thatharas" are also provided where distance between each is about 3.5m centre to centre.

Step II: Infill walls of following types are then erected between these Thatharas.

(1) **Stone:** Undressed stone laid in courses with mud mortar is used for infill walls in certain

Figure 22: Stone with mud mortar as infill

structures. These stone infill walls are of 500mm thickness which is also the width of the thathara.

(2) **Wood:** Wooden planks of 20 mm thickness are also used as infill walls. Usually these wooden planks infill is used in top most storey of the structure so as to make the upper storey lighter and hence reducing the load on storeys below.

Figure 24: Wooden planks as infill in the upper storey - village Bharmour, Distt. Chamba (H.P.)

(3) **Stone and Wood:** In some houses infill wall between two thatharas is made from both wood and stone. Sometimes there is

Figure 25: Diagonal bracing of wood in infill wall

Figure 23: A mud plastered thathara

diagonal bracing of wood and where small stones with mud mortar are packed in the remaining space (Dhajji Diwari). Usual practice is having unhewn wooden battens placed every second or third stone course or more.

Step III: The interiors and exteriors are plastered with mixture of mud and cow-dung. Afterwards, the walls are treated with mud and cow-dung slurry to give the final finish. Wooden walls are also finished with the above slurry in the inside as well as outside. In few cases, houses with exposed exteriors are also seen.

Walls over verandah:

Verandah at ground floor has only wooden posts and no walls in between. Walls over the verandah i.e. balcony above, are always made of wooden planks so as to minimize the load on verandah posts.

4.4 Floor System

4.4.1 Ground floor:

Wooden planks of thickness 20mm are laid over rammed earth. These planks are then plastered with mud and finally finished with mud and cow-dung slurry. The floor s coated with mud and cow-dung slurry every third day.

Figure 27: Main beam supported over posts (khambe) in the verandah and secondary beams also laid over main beam

Figure 26: Main beam supported over wooden post (thamb) in the middle of the room

4.4.2 Other floors:

Main beam of cross-section 270x230mm rests over the extreme thatharas spanning over whole length of room i.e. 6m without any joint in between. Distance between two main beams is

Figure 29: Stones filled in the gap between secondary beams

Figure 28: Planks laid over secondary beams

approximately 3m centre to centre. In the verandah, main beams are supported by wooden posts (khambe) of size 150x150mm at a distance of 1.5m.

In the middle of the room, main beam is supported over main wooden post (thamb) of size 270x270mm. Secondary beams of length 3m and cross-section 100x160mm are just laid over these main beams at a distance of 400mm centre to centre. Gap between two thatharas is filled with stones. Wooden planks of size 20x 300mm and length 2.5m are placed across over these secondary beams without any gap in between. These planks are covered with 25mm mud plaster and finished with slurry of mud and cow-dung mixture. These floors are coated with mud and cow-dung slurry every third day or so, to keep these clean and repair any small crack and patch that has developed.

4.5 Roof Typology

4.5.1 *ROOF SYSTEM:*

Thathara houses have usually gable roofs, with slope of about 17°. The roof changes angle over verandah and becomes a bit gradual so as to have adequate head room. Some houses have hipped roof too.

Construction methodology of roofs:

(1) **Gable-end roofs:** The most common practice is raising the two opposite thatharas which are at the middle, up to ridge level. Ridge beam (nhas) is directly placed (without any connection) over these thatharas with an intermediate support of wooden post (thamb). Ridge beam is always a single member without any joint in between. Sometimes this ridge

Figure 30: Gable end roof where ridge beam is supported over thathara

beam is a wooden log itself. Rafters at distance of around 1m centre to centre are laid above, which are connected at one end to the ridge beam and the wall plate (jail-dal) on the other. Rafters are secured at their place with iron nail connections. These wall plates (wooden) are directly placed over the wall without any connection. Rafters have purlins (batte) above nailed to them, to support the roof covering material - slate. Slates are also connected to the purlins with iron nails.

(2) **Hipped roofs:** A-type frame is constructed and rested over wall plates at a distance of about 1.5m. Over these purlins are nailed at a distance depending upon size of slates. Slates are nailed to these purlins. The slates are generally of the following sizes: 6"x12", 7"x14", 8"x16", 9"x18", 10"x20".

Figure 32: A-type frames

(3) **Flat roofs:** In a few cases where there is an additional single storey unit attached to the house, mainly for cattle or storage, then these units have flat roof made up of mud and wood. Main wooden beams are supported over thatharas at the corner. Secondary beams are placed over these nearly touching each other. 20mm thick wooden planks are laid across over these beams. This is covered with 150mm thick layer of mud.

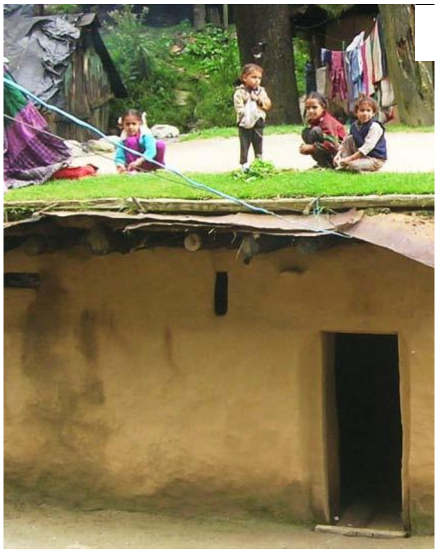


Figure 31: A flat roof unit

4.6 Structural Details

4.6.1 Gravity Load-Resisting System

The main load-bearing system of this building typology consists of 'Tholas' and wooden beams. Tholas are provided at corners and/or ridges of the building and support the horizontal beams which in turn support the inclined rafters and purlins. A positive connection between Tholas and beams has generally not been observed and the beams are simply kept over the Tholas.

4.6.2 Lateral Load-Resisting System

The resistance to lateral loads is provided by wooden framing or in-plane action of walls. Although these walls are generally made of poor quality material, such as adobe or random rubble, the large cross-sectional area with minimal openings provides adequate lateral resistance if built and maintained well. The lateral load-resisting feature of these buildings are horizontal members (ties) provided at several intermediate levels between the floors to support the walls in out-of-plane action. This type of construction known as Kath-Kunni has been traditionally used in the northern states Himachal Pradesh and Uttarakhand and has been presented in the WHE report #150 (Rautela et al. 2009). In some cases, Dhajji-Diwari construction (Figure 16, see WHE report #146; Hiçyılmaz et al. 2012) is also used for partition, in which diagonal braces are used in wooden frame. In the uppermost storey, generally wooden frames and planks are used as partition material to reduce the seismic weight of the building. The original construction practice involving use of wooden planks for roof covering was also motivated from the concept of reducing mass at the top. In verandahs, where larger openings are required, wooden frames are used in place of masonry walls. These wooden frames result in reduced seismic mass and better lateral load resistance.

4.7 Seismic Vulnerability Analysis

4.7.1 Structural and Architectural Features

Structural/ Architectural Feature	Statement	Most appropriate type		
		True	False	N/A
Lateral load path	The structure contains a complete load path for seismic force effects from any horizontal direction that	☑	☐	☐

	serves to transfer inertial forces from the building to the foundation.			
Building Configuration	The building is regular with regards to both the plan and the elevation.	☑	☐	☐
Roof construction	The roof diaphragm is considered to be rigid and it is expected that the roof structure will maintain its integrity, i.e. shape and form, during an earthquake of intensity expected in this area.	☐	☑	☐
Floor construction	The floor diaphragm(s) are considered to be rigid and it is expected that the floor structure(s) will maintain its integrity during an earthquake of intensity expected in this area.	☐	☑	☐
Foundation performance	There is no evidence of excessive foundation movement (e.g. settlement) that would affect the integrity or performance of the structure in an earthquake.	☐	☑	☐
Wall and frame structures - redundancy	The number of lines of walls or frames in each principal direction is greater than or equal to 2.	☑	☐	☐
Wall proportions	Height-to-thickness ratio of the shear walls at each floor level is: Less than 25 (concrete walls); Less than 30 (reinforced masonry walls); Less than 13 (unreinforced masonry walls);	☑	☐	☐
Foundation-wall connection	Vertical load-bearing elements (columns, walls) are attached to the foundations; concrete columns and walls are doweled into the foundation.	☐	☑	☐
Wall-roof connections	Exterior walls are anchored for out-of-plane seismic	☐	☑	☐

	effects at each diaphragm level with metal anchors or straps			
Wall openings	The total width of door and window openings in a wall is: For brick masonry construction in cement mortar : less than ½ of the distance between the adjacent cross walls; For adobe masonry, stone masonry and brick masonry in mud mortar: less than 1/3 of the distance between the adjacent cross walls; For precast concrete wall structures: less than 3/4 of the length of a perimeter wall.	☑	☐	☐
Quality of building materials	Quality of building materials is considered to be adequate per the requirements of national codes and standards (an estimate).	☐	☑	☐
Quality of workmanship	Quality of workmanship (based on visual inspection of few typical buildings) is considered to be good (per local construction standards).	☐	☑	☐
Maintenance	Buildings of this type are generally well maintained and there are no visible signs of deterioration of building elements (concrete, steel, timber)	☐	☑	☐

4.7.1.1 Seismic Features

Structural Element	**Seismic Deficiency**	**Earthquake Resilient Features**
Wall	Generally of poor quality material, without any reinforcement.	Horizontal wooden members (similar to ties, known as 'Kath-Kunni' in local language) to protect the walls in out-of-plane action and small window openings. In some cases wooden bracings (known as 'Dhajji-Diwari' in local

		language) is also used.
Frame (columns, beams)	No positive moment connection between columns (tholas) and beams.	Enlarged cross-section of wooden columns (Tholas) results in enhanced lateral resistance.
Roof and floors	No cross bracings provided in floors/roofs, no ties in sloping roof, no anchorage of roof/floor with walls, even in newer constructions.	Light-weight wooden plank covering and A-shaped bracing in old constructions.
Columns (Tholas)	Packed with dry stones without mortar.	Interconnected Thatharas (wooden planks constituting Tholas) provide enhanced lateral resistance.

FINITE ELEMENT ANALYSIS OF THATHRA ARCHITECTURE

5 Finite element analysis of *thathra* architecture

5.1 Background

Before analysis of individual substructures, a general discussion about the 3D finite-element model of the full structure is necessary. As the literature review suggests, analytical model of the whole structure had to be simplified for sake of calculation as computational solution of the detailed model may not be feasible. To retain each and every detail in the model is impossible because of the large number of motion associated with each node. So the building was assumed to be an assembly of substructures for computational purposes.

The following major substructures and components are analyzed:

- Walls
- Floor
- Intercomponent connections

Substructures are joined in the structural framework by means of intercomponent associations. The general reaction of the framework to a connected load is nonlinear, however a percentage of the substructures hold a linear reaction while others react nonlinearly. Intercomponent associations need separate investigations and examinations to know their reactions to loads.

The analysed 3D model was outlined as a composition of walls, frame, floor and roof joined by intercomponent associations. The roof, wall and floor were broke down as superelements. The floor is connected with a model of orthotropic plate as an optional way to analyze.

The idea of superelements and substructuring is utilized. The superelement, a piece of the structural model, acts linearly and is joined with the model through its nodes. Through condensation process the degrees of freedom of the superelement are removed and don't effect the result of the whole structure.

The idea of substructures on a fundamental level is, the substantial consistent area is partitioned into the subdomains and just the degrees of freedom on limits are used. The computational subdomain is additionally called a superelement.

The full structure analysis requires simplifications based on the literature review and computational resources. The following assumptions are made:

- The behavior of roof is linear in bending
- The behavior of diaphragm of the roof is linear.
- The behavior of floor is linear in bending and also acts as an orthotropic plate.
- Intercomponent connections are nonlinear.
- Self-weight of the whole model is analyzed as dead load.

5.2 Walls

Walls are vertical substructures designed to transfer forces to the foundation. For this analysis the wall model was considered to be 3 meters in height, 7 meters in length and 450 mm thick. Construction material used was fieldstone with following properties which matches most to the stones used for construction of thatra style.

Material	E (MPa)	G (MPa)	NI	LX (1/°C)	RO (kN/m3)	Re (MPa)
Fieldstone	77000.00	30000.00	0.25	0.00	25.50	27.58

The following forces were identified and were considered during structural analysis of the model:

- Horizontal shear force due to wind and earthquake.
- Bending and torsional moments resulting from the wind loadand forces acting on the wall boundary connected to other substructures.
- Axial forces due to the weight of the carried substructures (roof, floor, including dead loads, live loads and other loads resulting from the function of the structure).
- Body forces due to the self-weight.

These forces can act all the while and the proper load combinations are analyzed. The model comprises of a detailed 3D finite element model.

5.2.1 Analysis

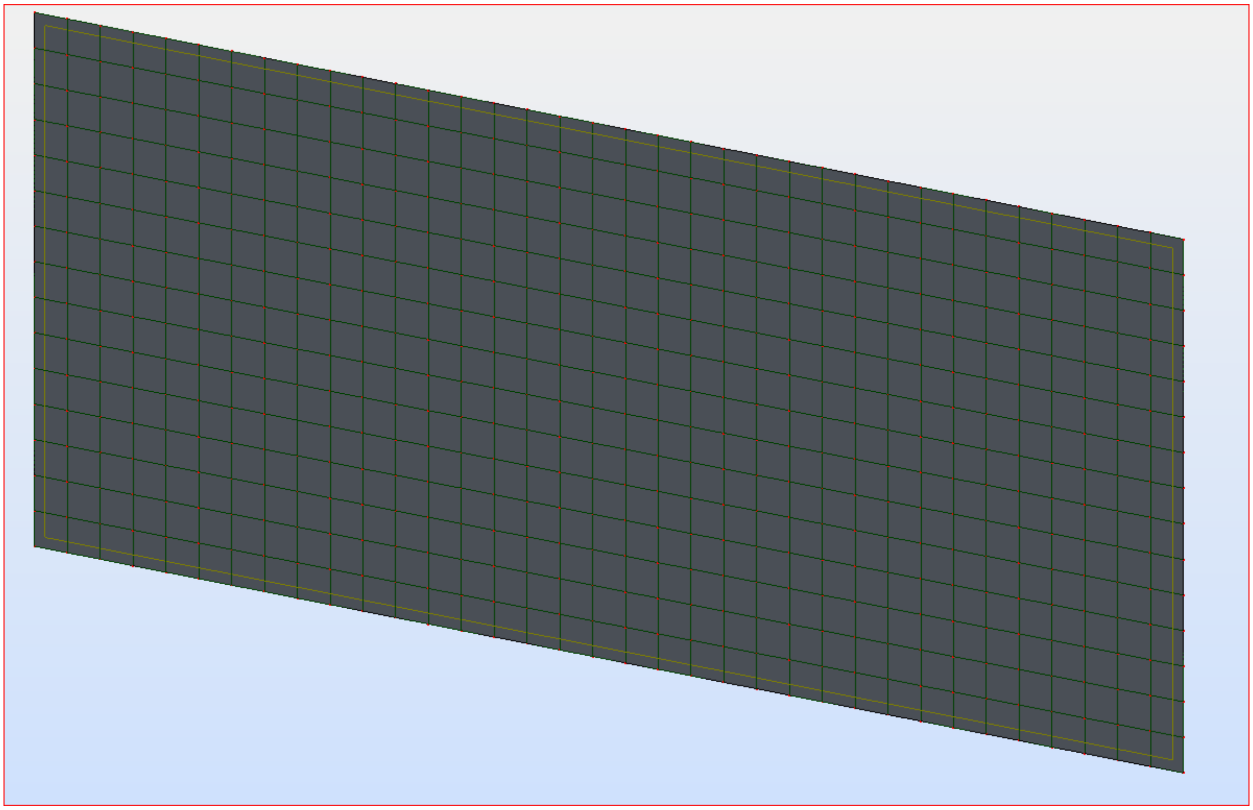


Figure 33: Finite element mesh

The analyzed portion of the wall of 7 X 3 X .5 m is further divided into 576 nodes and 525 planar elements during creation of finite element mesh of the model. This analysis in various small parts of the whole structure gives better accuracy in delivering results.

Following loads were applied on the structure during analysis:

S.No	Loads	Direction	Value (KN/m2)
1	Dead Load	-Z	Self-Weight
2	Live Load	-Z	2
3	Wind Load	XY	As per IS 875(part 3)
4	Seismic Load -- I	XX	As per IS 1893:2001
5	Seismic Load -- II	YY	As per IS 1893:2001

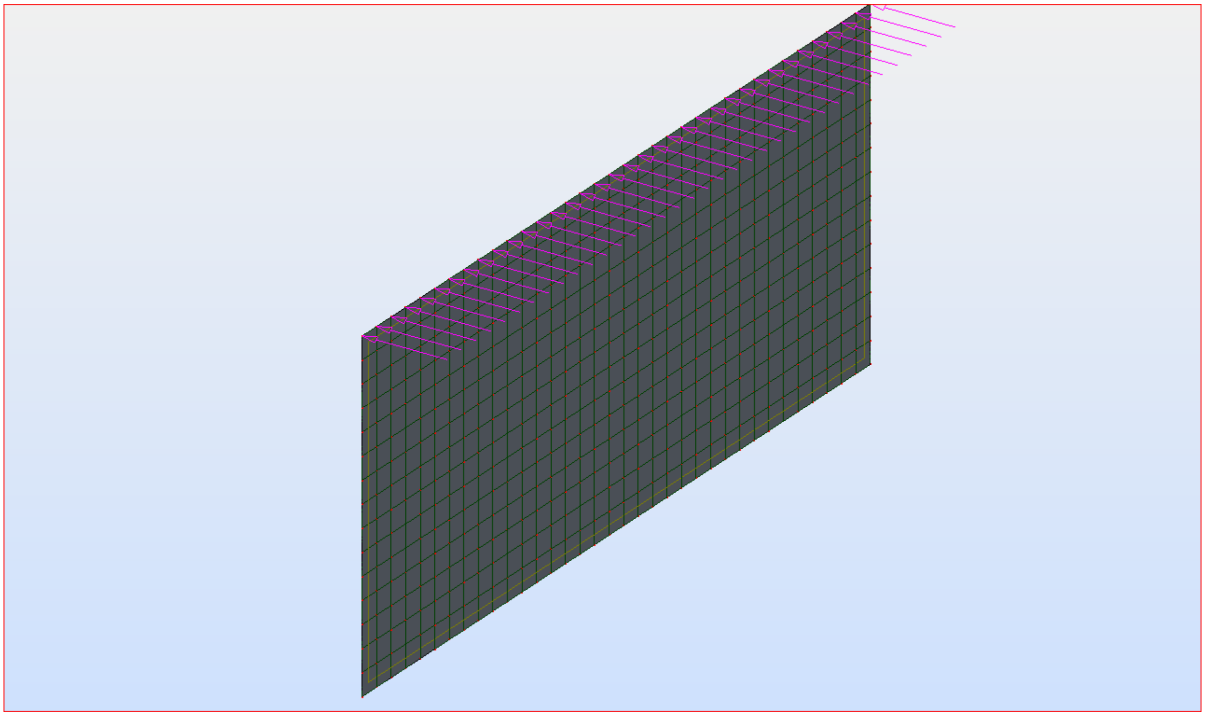


Figure 34: seismic load XX

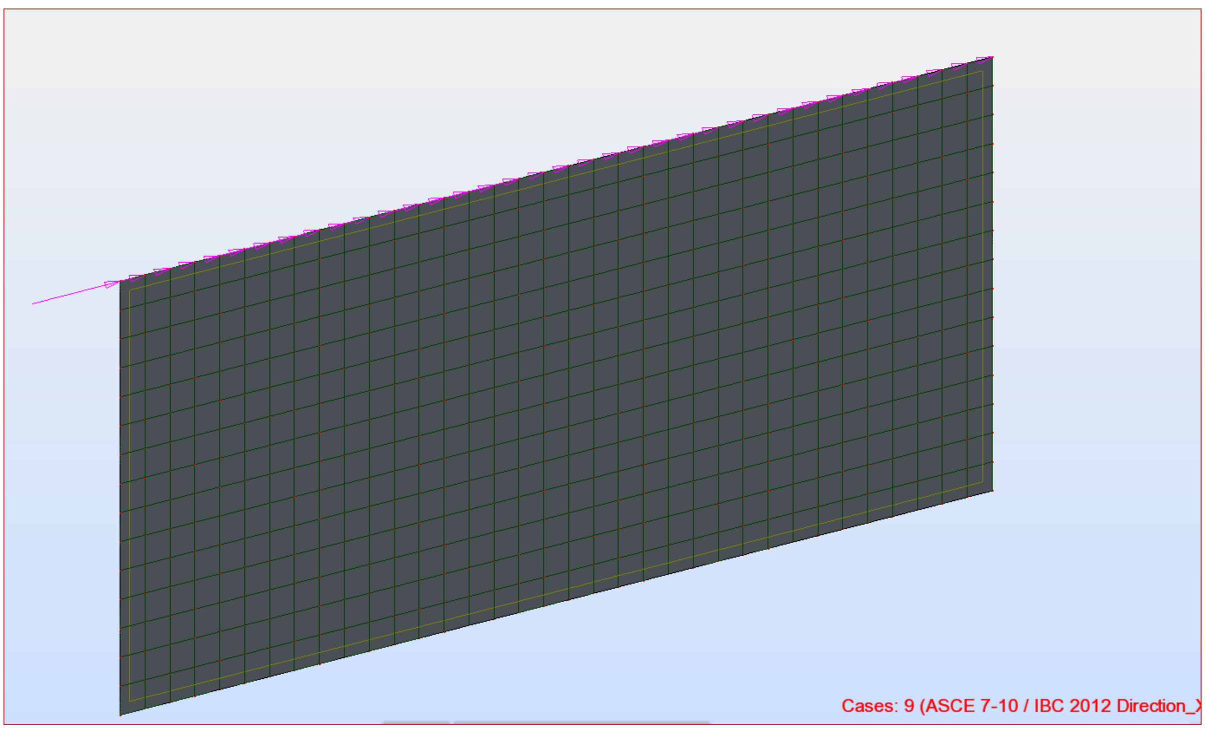


Figure 35: seismic load YY

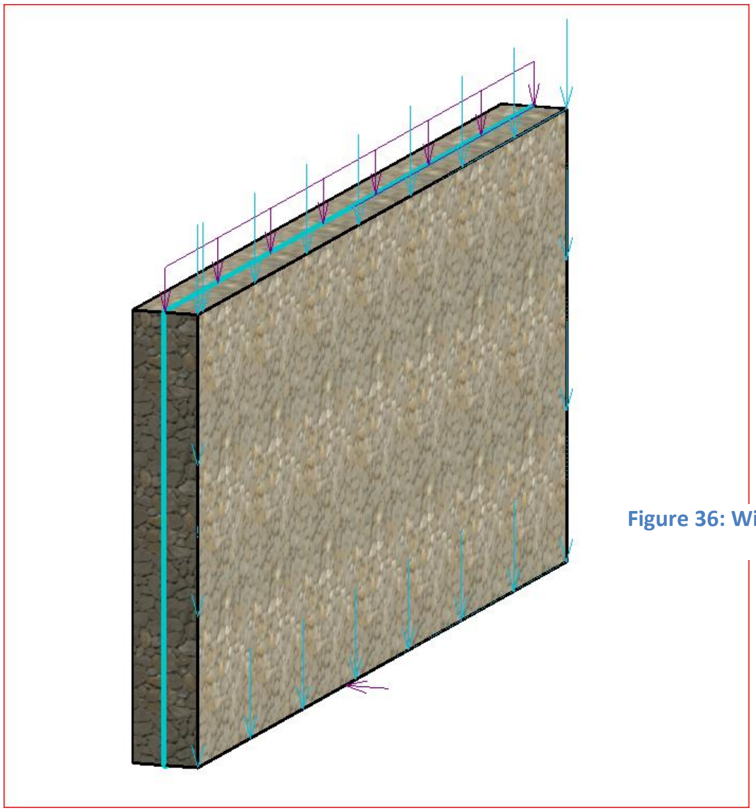


Figure 36: Wind, live and dead loads action on wall

Results

Characteristics of analysis example:

Structure type: Shell

Structure geometrical center coordinates:

X = 3.500 (m)

Y = -0.000 (m)

Z = 1.500 (m)

Structure gravity center coordinates:

X = 3.500 (m)

Y = 0.000 (m)

Z = 1.500 (m)

Central moments of inertia of a structure:

Ix = 18427.500 (kg*m2)

Iy = 118755.000 (kg*m2)

Iz = 100327.500 (kg*m2)

Mass = 24570.000 (kg)

Structure description

Number of nodes: 576

Planar finite elements: 525

Cases: 7

Calculation summary

Solution method - SPARSE

No of static degr. of freedom: 3456

Table of load cases / analysis types

Case 1 : DL1

Analysis type: Static - Linear

Potential energy : 8.53365e+010 (kN*m)

Precision : 4.08906e+001

Case 2 : ASCE 7-10 / IBC 2012 Direction_X

Analysis type: Static - Seismic

Excitation direction:

X = 1.000

Y = 0.000

Z = 0.000

Data:

Soil : A

S1 : 0.100

SS : 0.250

Spectrum parameters:

Fa =	0.800	Fv =	0.800
SMS =	0.200	SM1 =	0.080
SDS =	0.133	SD1 =	0.053
To =	0.080	TS =	0.400
TL =	2.000		
I =	1.000	R =	1.000

Fundamental period:

Approximated method T = 0.111 (s)

Other structures Ct = 0.02 (0.0488) x = 0.75

Structure range:

Effective height Hn = 3.00(m)

Base shear

Cs = 0.133

Cs max = 0.479

Cs min = 0.010

Effective seismic weight W = 24570.00(kG)

Shear force V = 32.13(kN)

Vertical distribution of seismic forces

Story	Height (m)	Weight (kG)	F(kN)	M(kN*m)

Level 1	3.00	24570.00	32.13	0.00

Case 3 : ASCE 7-10 / IBC 2012 Direction_Y

Analysis type: Static - Seismic

Excitation direction:

X = 0.000

Y = 1.000

Z = 0.000

Data:

Soil : A

S1 : 0.100

SS : 0.250

Spectrum parameters:

Fa = 0.800 Fv = 0.800

SMS = 0.200 SM1 = 0.080

SDS = 0.133 SD1 = 0.053

To = 0.080 TS = 0.400

TL = 2.000

I = 1.000 R = 1.000

Fundamental period:

Approximated method T = 0.111 (s)

Other structures Ct = 0.02 (0.0488) x = 0.75

Structure range:

Effective height Hn = 3.00(m)

Base shear

Cs = 0.133

Cs max = 0.479

Cs min = 0.010

Effective seismic weight W = 24570.00(kG)

Shear force V = 32.13(kN)

Vertical distribution of seismic forces

Story	Height (m)	Weight (kG)	F(kN)	M(kN*m)
Level 1	3.00	24570.00	32.13	0.00

5.2.2 Shear resistance

The shear firmness was demonstrated by a nonlinear behavior of the nodes and planes. The utilization of shaft components made it conceivable to incorporate extra nodes at the limit and these nodes were utilized to join the divider to whatever remains of the structure. Additionally, imperative mathematical statements were characterized to guarantee an unbending body turns of the studs in the plane of the wall. Stipulation comparisons express the level of flexibility at one essential node as a direct blend of degrees of freedom at some other nodes. The mathematical statement comparing to the essential level of flexibility is expelled from the framework. The upper bar experiences an inflexible body movement in the load direction.

The edge shafts have insignificant out-of-plane twisting stiffness and very large resistance in direction of plane. Utilizing this arrangement, the main commitment to the worldwide firmness grid is the interpretation in the z-course and extra bowing firmness in the out-of-plane bearing as managed by flat bars. The load direction curve from the shear examination of the model is the spring firmness for the equal model, or it could be the result from test or whatever viable suitable model.

The below image shows the shear stress developed during the simple load combination of all forces. It reflects that the mid-section of the wall is developing large stress which can lead to its failure whereas the stress level remains under limit in rest of the substructure.

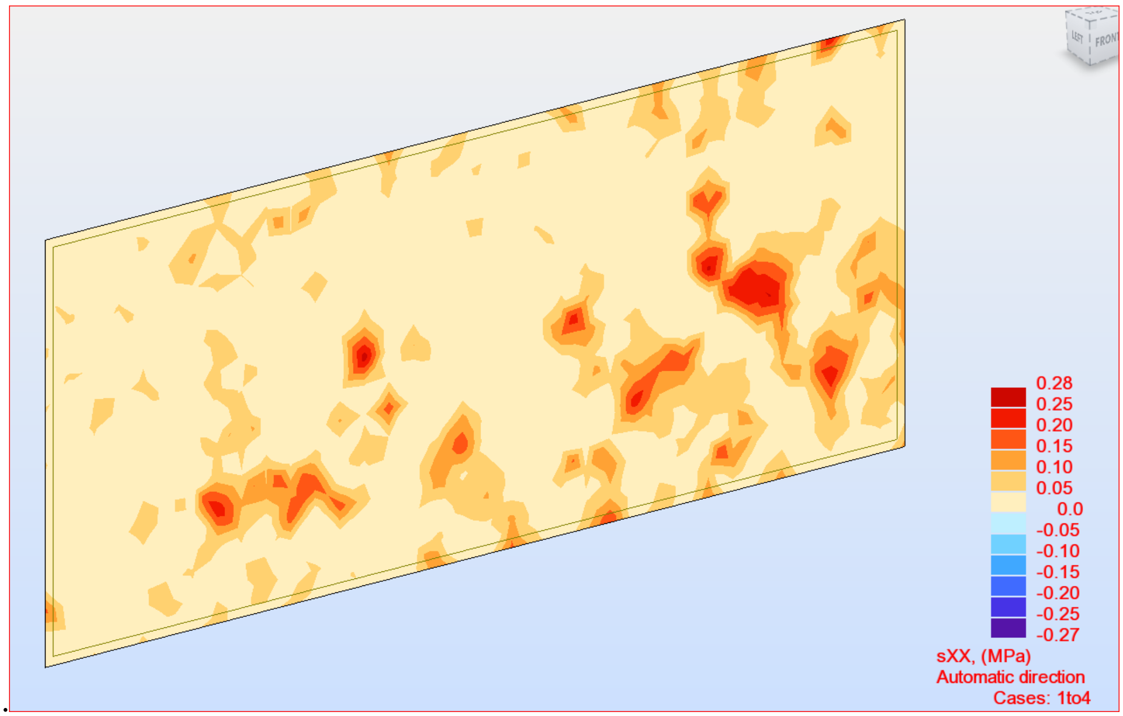


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Figure 37: Shear stress in XX

The diagram below represents the shear stress developed in YY direction when under the loads. It increases towards the top of the wall but that may be attributed to lack of anchorage in the wall allowing it for out of plane motion.

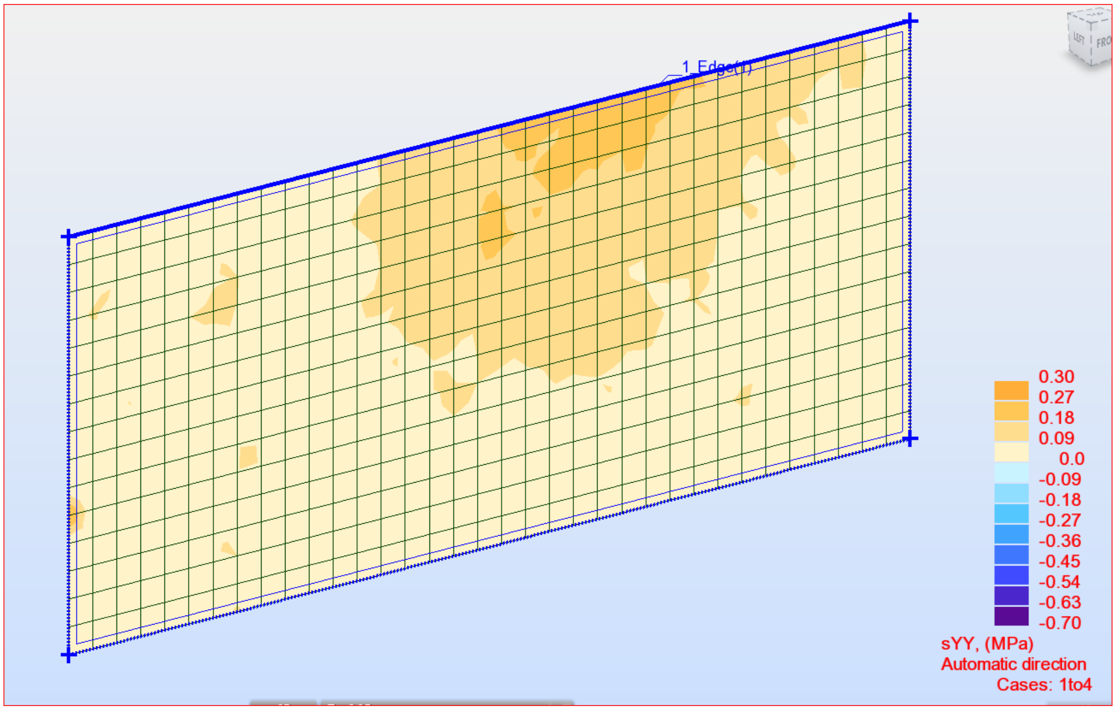

Figure 38: Shear stress in YY

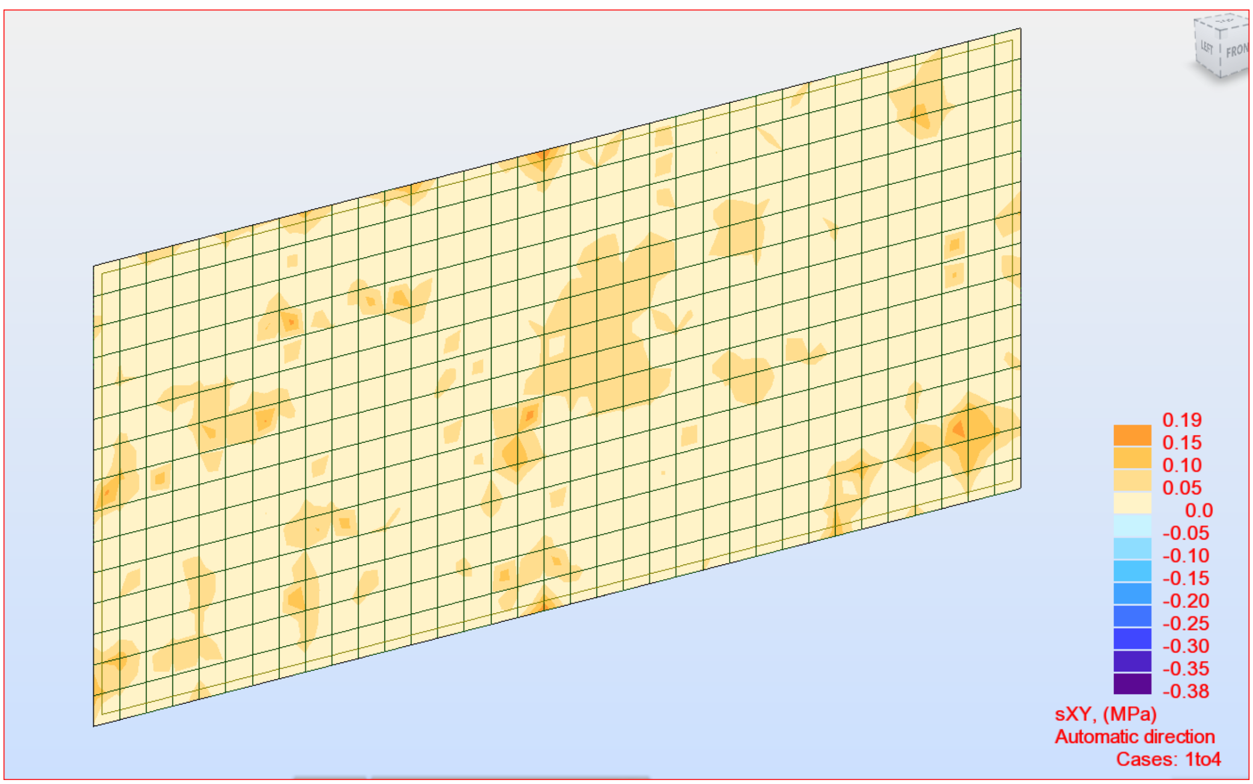

Figure 39: Shear stress in XY

From the scientific examination of the coupling in the middle of bending and shear loads, it takes after that shear firmness may be lessened when the wall is stacked by weight. To record for the impact of the wall twisting on the shear firmness, the heap distortion bend of the wall in shear could be gotten by analysing the 3D model for a consolidation of wind weight and shear rather than for the straightforward shear. This might be effortlessly performed since wind weight (suction) on the outside walls is known from the IS 875 (part 3). The weight and shear must be augmented at the same time to represent the concurrent impact of the wind load. Since the walls in the building for the most part stacked in shear are not at the same time subjected to high wind weight (shear walls are spotted in the heading parallel to the wind course normally bringing about suction impressively lower than wind weight on a windward side of the building), the dissection for concurrent impact won't prompt considerable change in the divider shear solidness.

5.2.3 Out-Of-Plane Stiffness

Wind weight and reaction forces from perpendicular walls and diaphragms leads to bending and torsional moments. These strengths are opposed by the wall which tends to behave as a bending member.

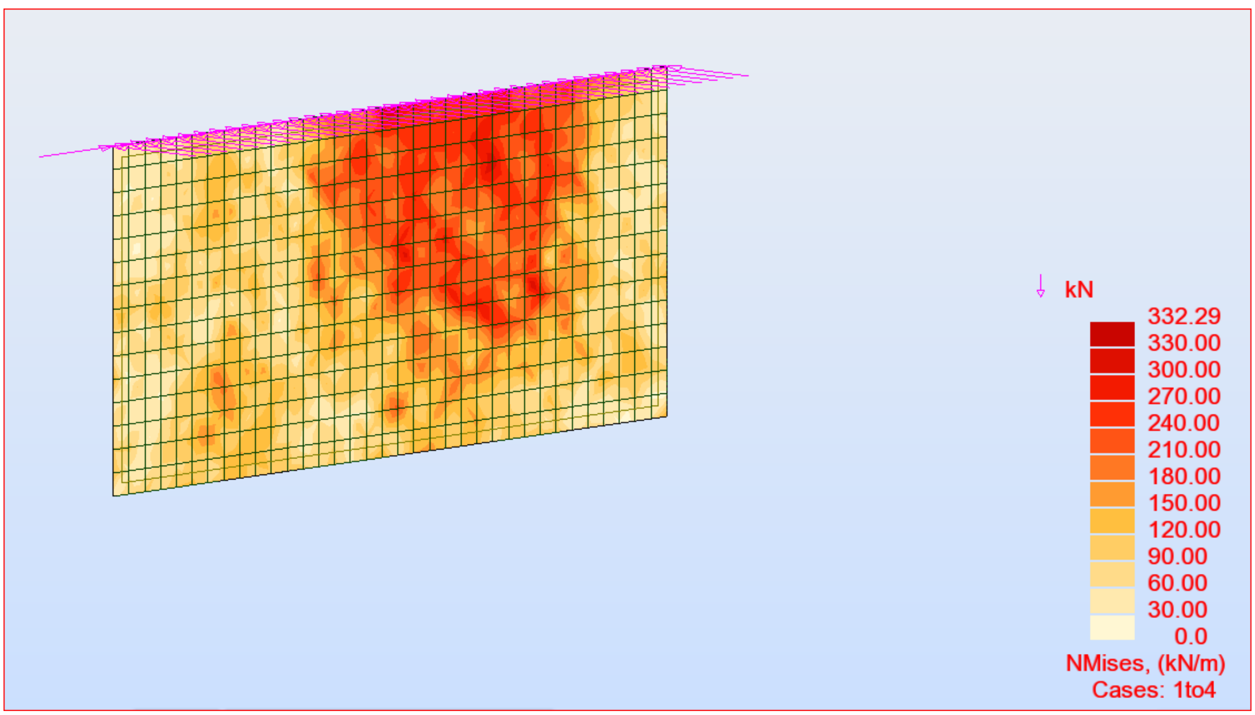

Figure 40: Membrane forces

The bending stiffness is displayed by superimposing a two dimensional plate component with the layer firmness uprooted. The plate is orthotropic to record for diverse stiffnesses of the divider in the tallness and width of the divider. An iterative answer for the orthotropic plate under distinctive limit and stacking conditions is utilized to make the plate firmness.

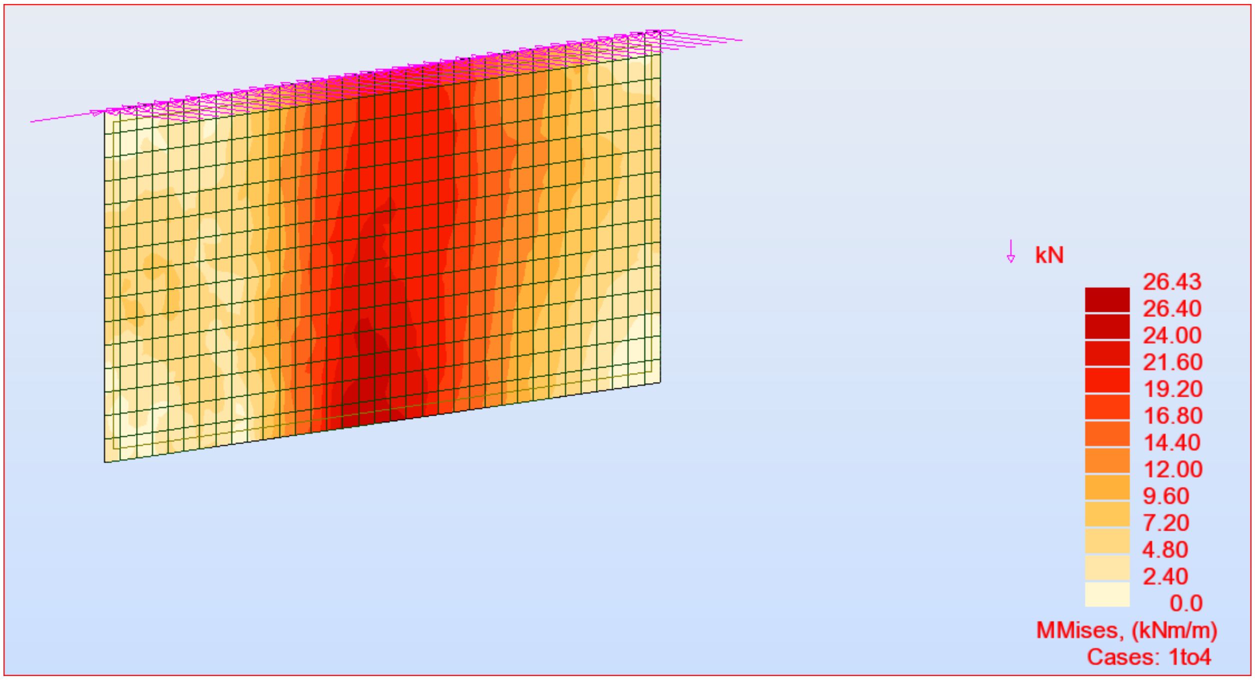

Figure 41: Moments

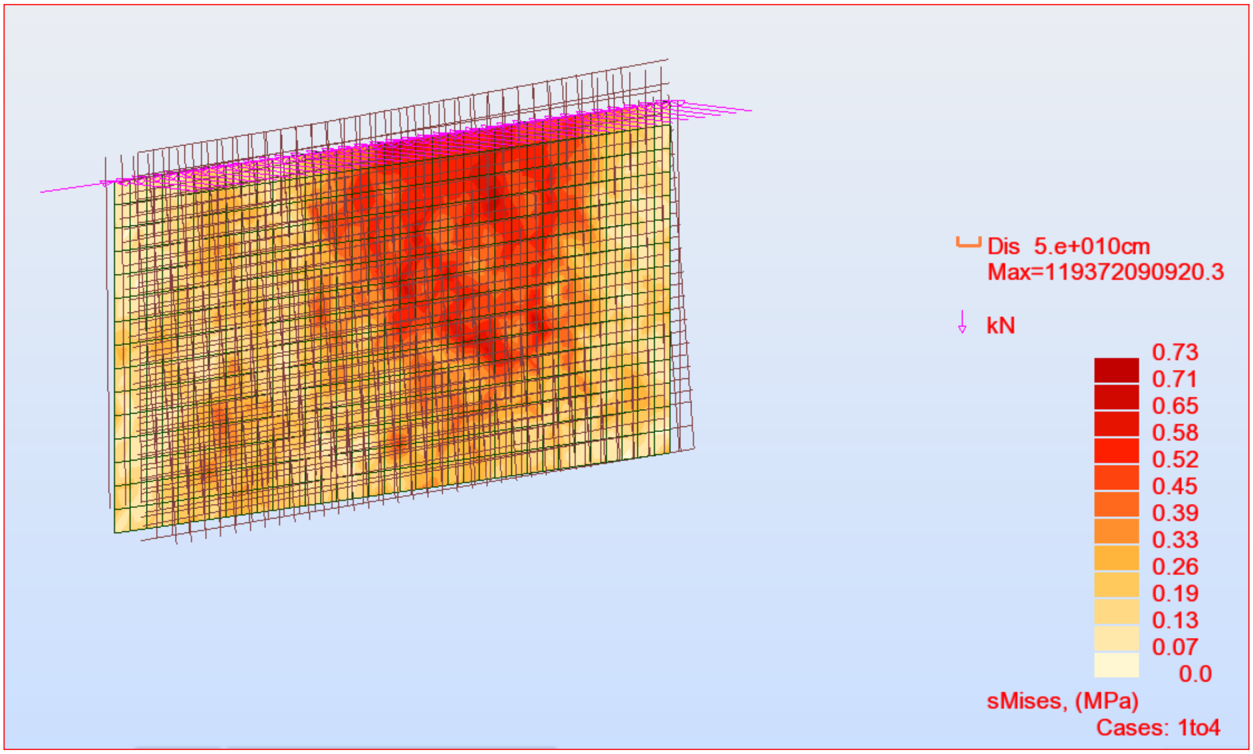

Figure 42: Active deformations

5.2.4 Summary

After analyzing the various maps and calculation results it has been found that behaves well in shear but fails in bending.

5.3 Floor

Floors are integral substructures which transfers the live loads to the frame. Floor slab for analysis in the said architecture style is opted to be dimension of 7X7 meter. Main beam of size 270X230 mm rests on the *thathra* on either side. As the span is larger another main beam run through the centre which is supported in the middle by a single timber member of dime

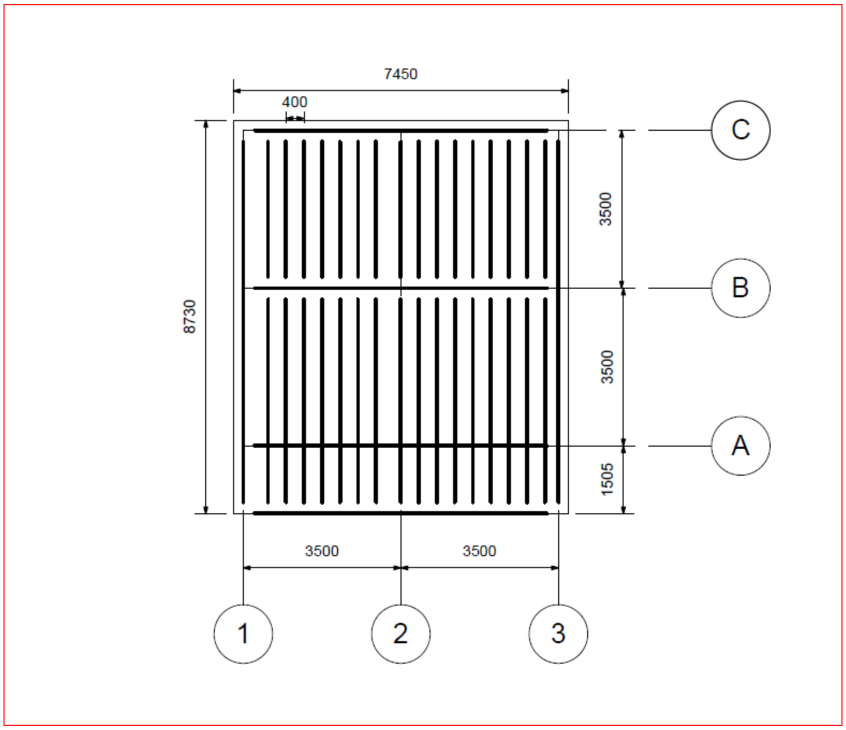

Figure 43: Floor plan

nsion 270X270 mm. Secondary beams of size 3000 X 160 X 100 mm are nailed to the main beams at a distance of 400mm centre to centre. A total of 3 main beams run in X axis and2 main beams and 17 secondary beams run in y direction. Thus this slab will be considered as one way slab as the secondary reinforcing beams run in

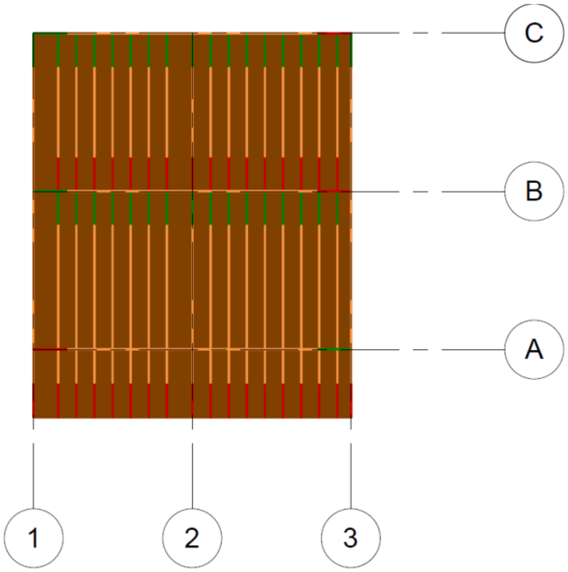

Figure 45: Analytical plan

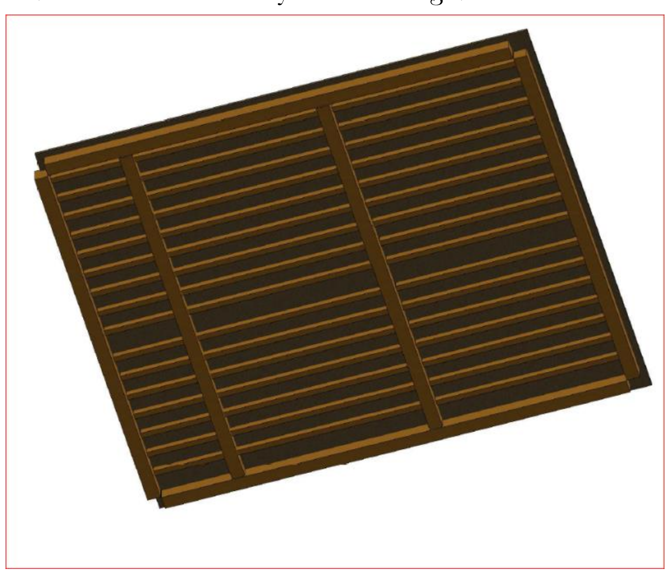

Figure 44: 3D model

single axis. This slab will behave as flexible diaphragm.

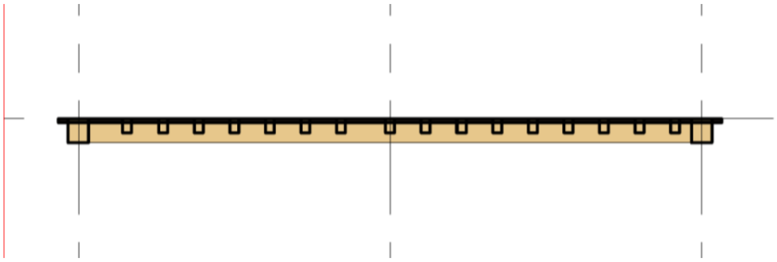

Figure 47: Section

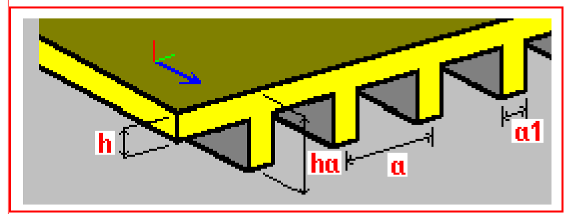

Figure 46: Section detail

H=45mm, ha=200mm, a=400mm, a1=100mm

The definite finite element model utilized two-dimensional orthotropic shell components for joists and sheathing. Joists furthermore sheathing had free node numbering and were joined by means of nonlinear burden redirection components. For effortlessness, gaps were excluded from the model and discontinuities in the sheathing were joined by diverse node numbers for every plywood board.

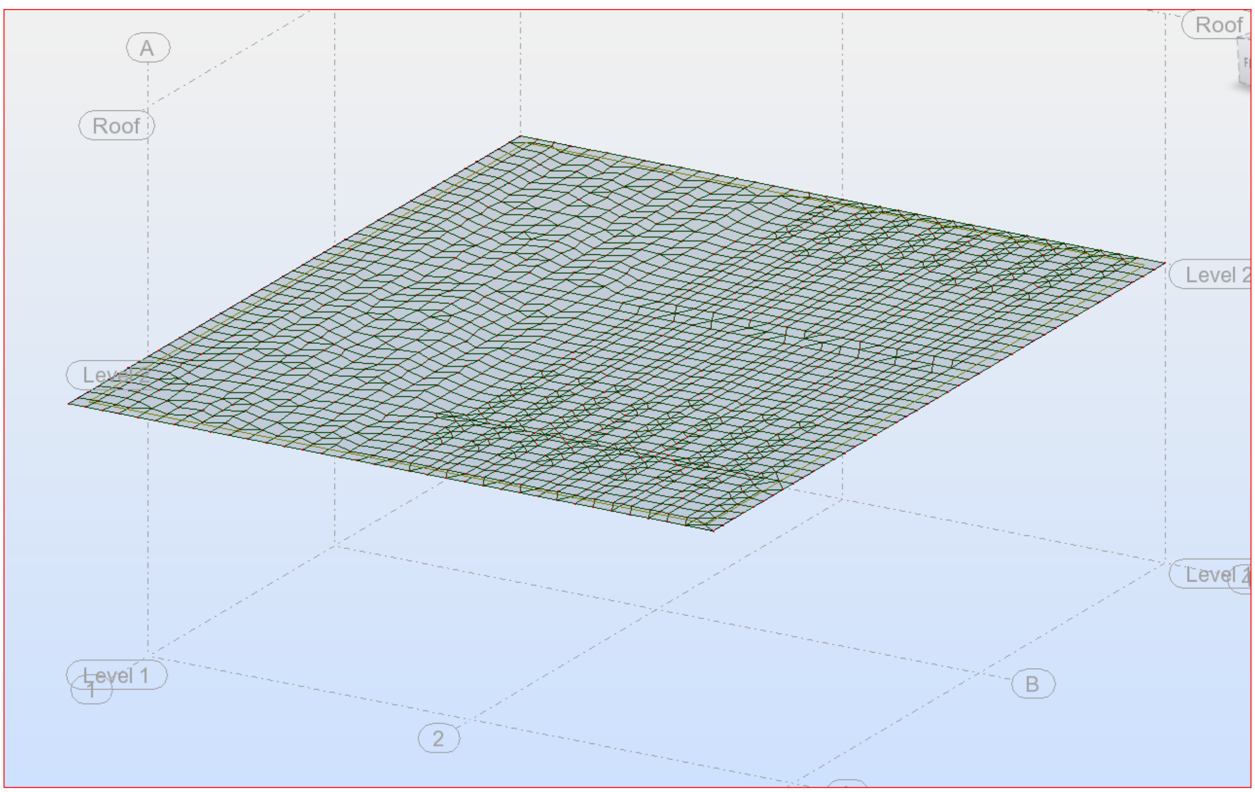

Figure 48: Finite element mesh

The following assumptions were made for carrying out the analysis:

- Loads will only be in transverse direction and will be uniformly distributed.
- Clasps have consistent spacing all through the length of the board.
- All joists have the same spacing.
- Floor is simply supported.

5.3.1 Analysis

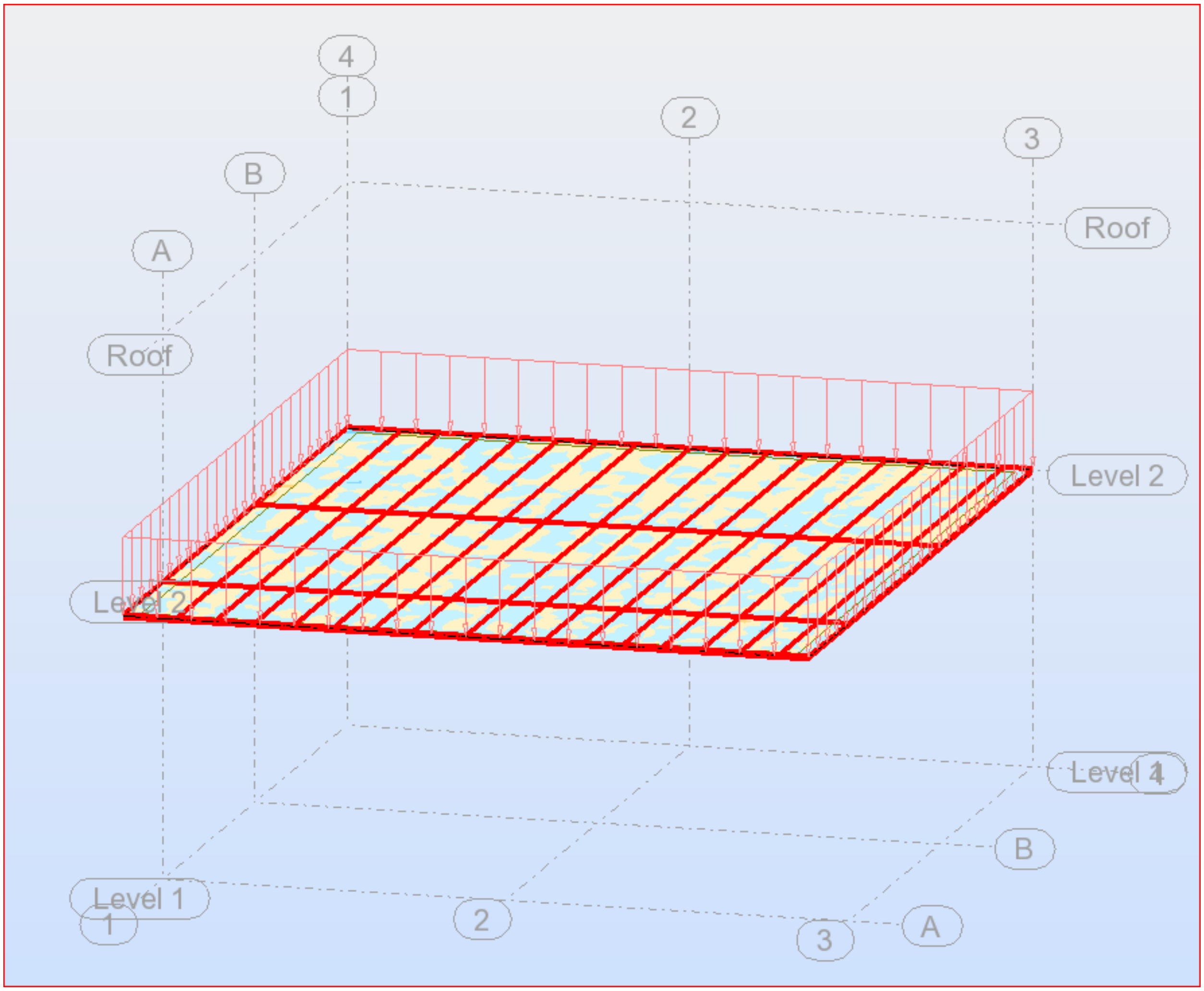

Figure 49: Loads acting on structure

Results

Structure type: Shell

Structure geometrical center coordinates:

X = 3.500 (m)

Y = 2.750 (m)

Z = 3.000 (m)

Structure gravity center coordinates:

X = 3.526 (m)

Y = 2.672 (m)

Z = 3.000 (m)

Central moments of inertia of a structure:

Ix = 44093.760 (kg*m2)

Iy = 30958.870 (kg*m2)

Iz = 75040.353 (kg*m2)

Mass = 6479.038 (kg)

Structure description

Number of nodes: 1944

Number of bars: 38

Bar finite elements: 952

Planar finite elements: 2187

Calculation summary

Solution method - SPARSE M

No of static degr. of freedom: 11664

Table of load cases / analysis types

Case 1 : DL1

Analysis type: Static - Linear

Potential energy: 7.36895e+011 (kN*m)

Precision: 2.83574e+001

Case 2 : LL1

Analysis type: Static - Linear

Potential energy: 2.48287e+012 (kN*m)

Precision: 3.07059e+001

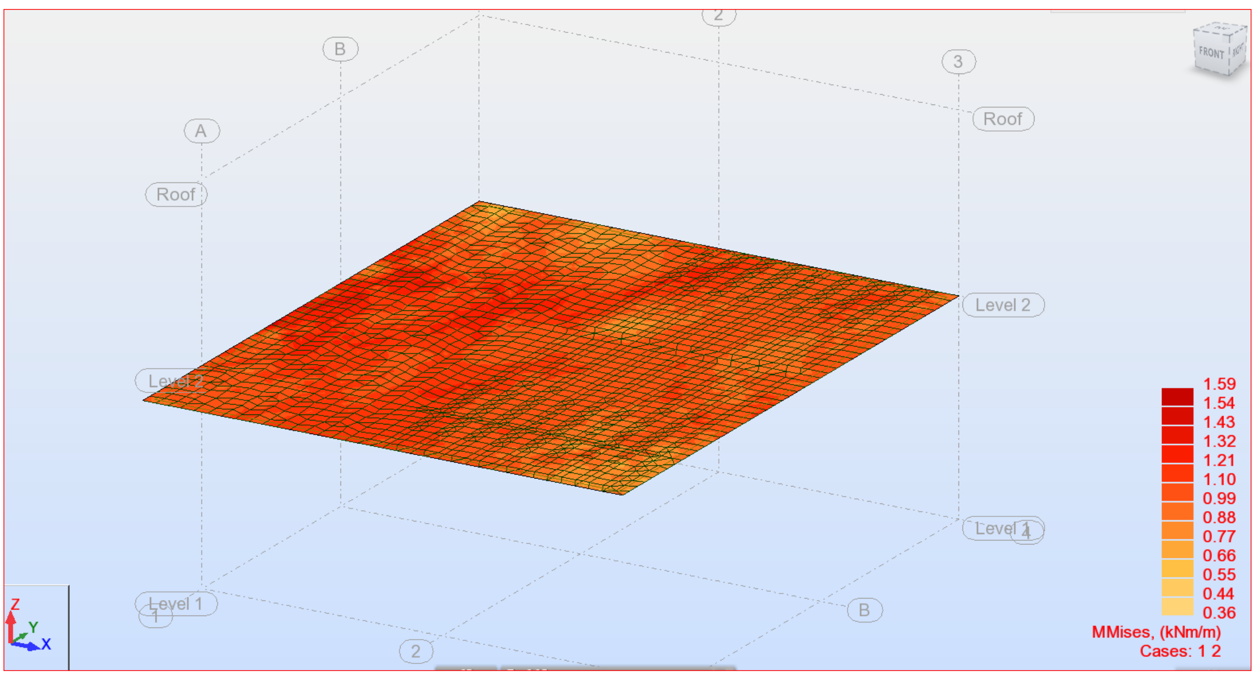

Figure 50: Moment in whole structure

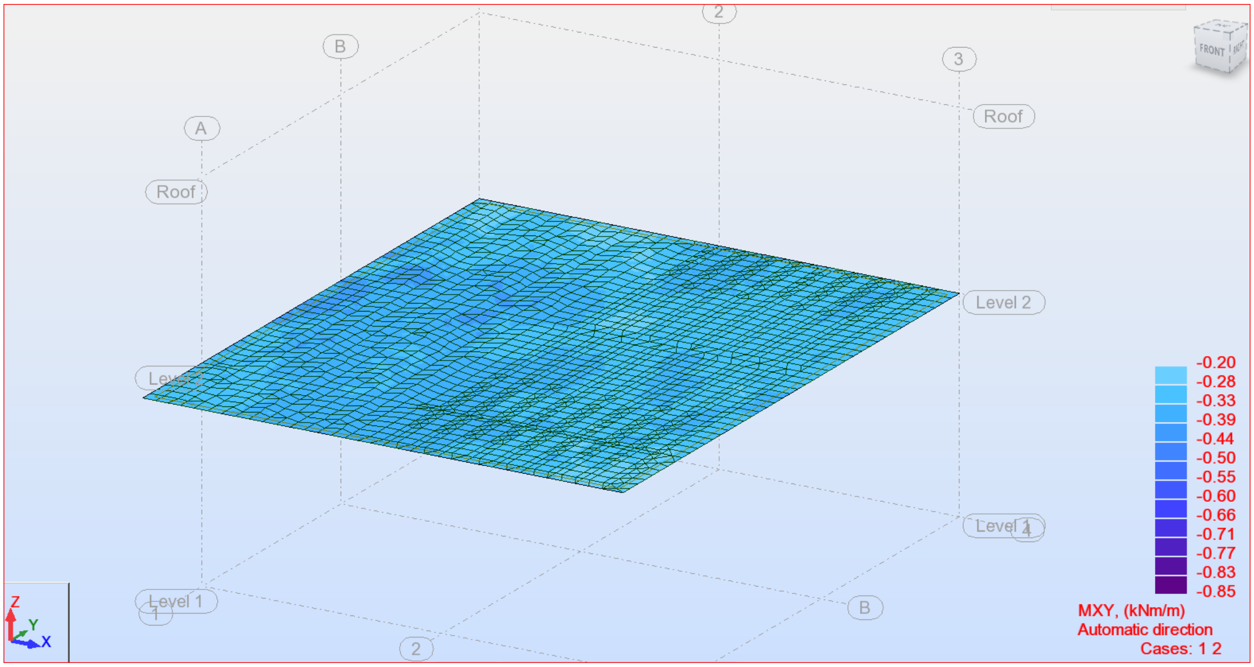

Figure 51: Moment in XY

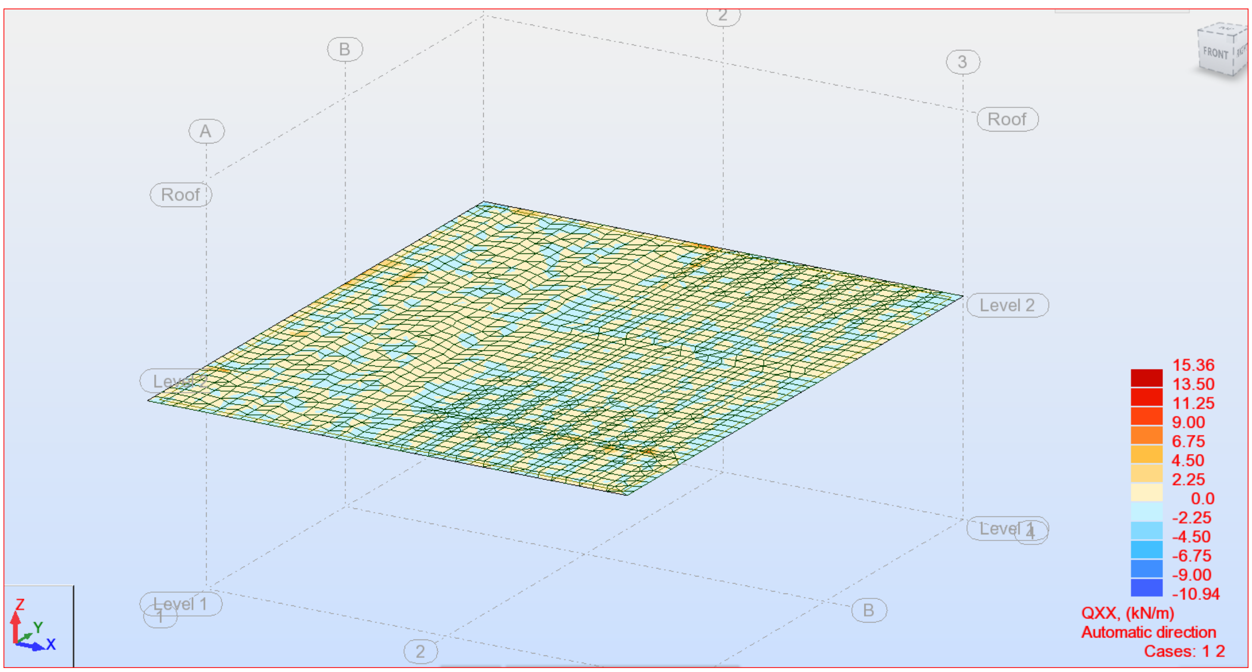

Figure 52: Shear forces in XX

The above maps of moment in the floor plate reflect the torsional force the nodes in the plane are going through. As it is reflected through the diagrams the moment acts most in the Y direction towards geometrical center of the floor. Thus the chances of deformation in the separate zones crop up.

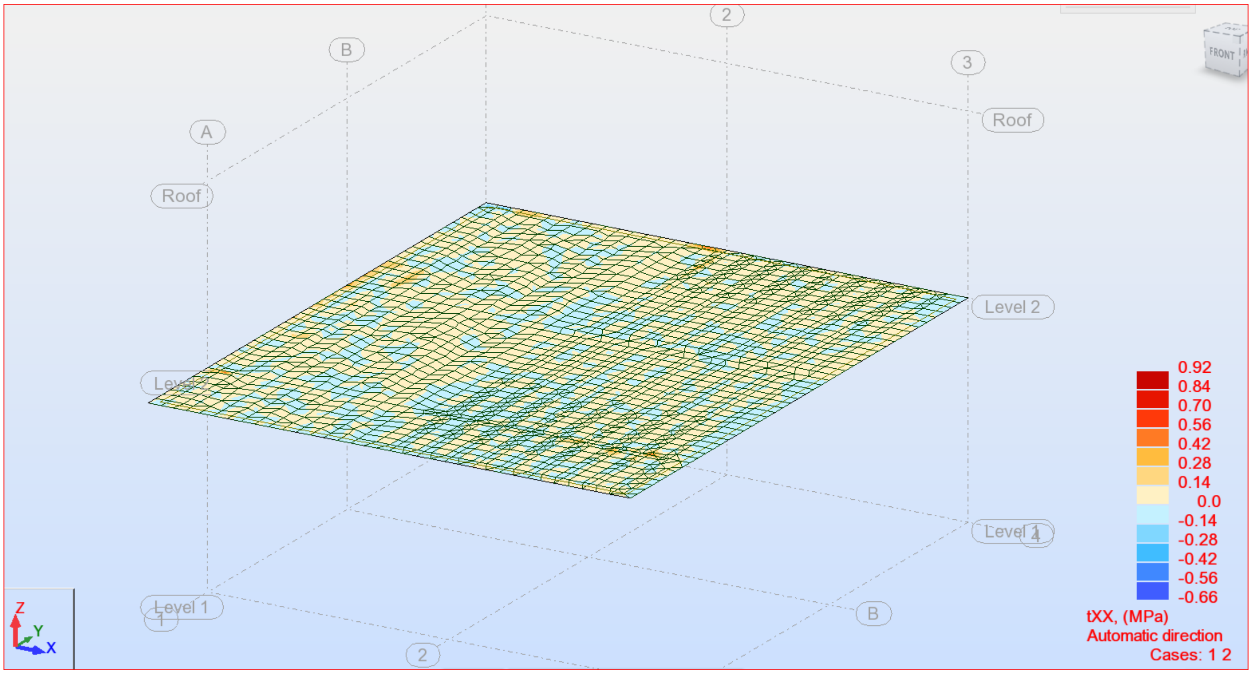

Figure 53: Shear stress in XX

The following diagram represents the shear stress developed in different directions of the substructure. The shear gradient prevalent is pretty uniform thus eradicating the chances of shear failure in the floor plate. Whereas a much more uniform distribution of shear load will be appreciated through the load transferring beams.

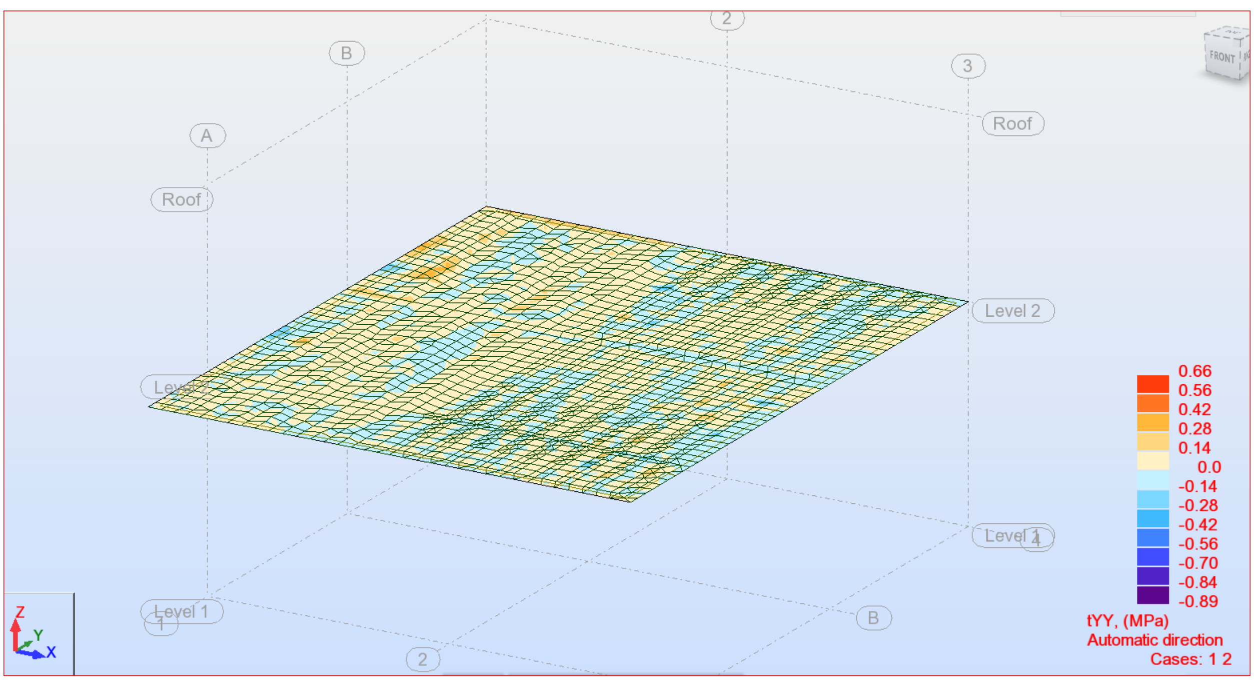

Figure 54: Shear stress in YY

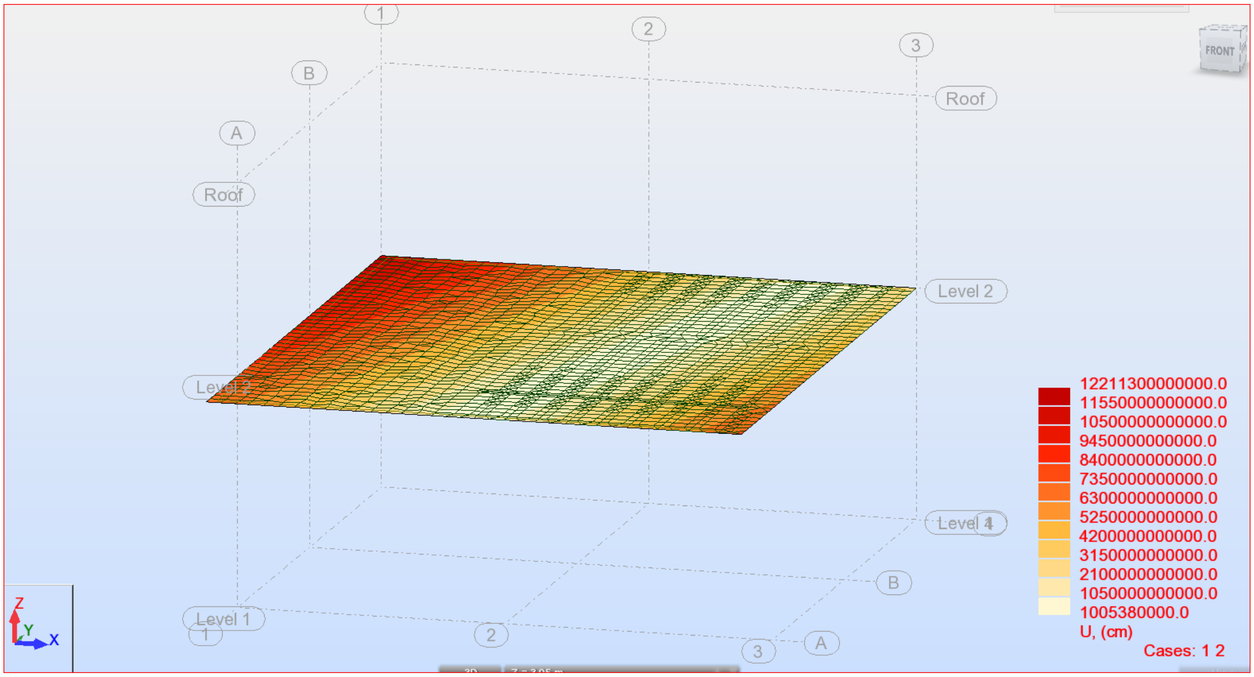

Figure 55: Total displacement

These maps represent the total displacement and deformation of the nodes. It shows that lack of proper connection at ends results in excessive moment at the corner leading to deformation in the zone.

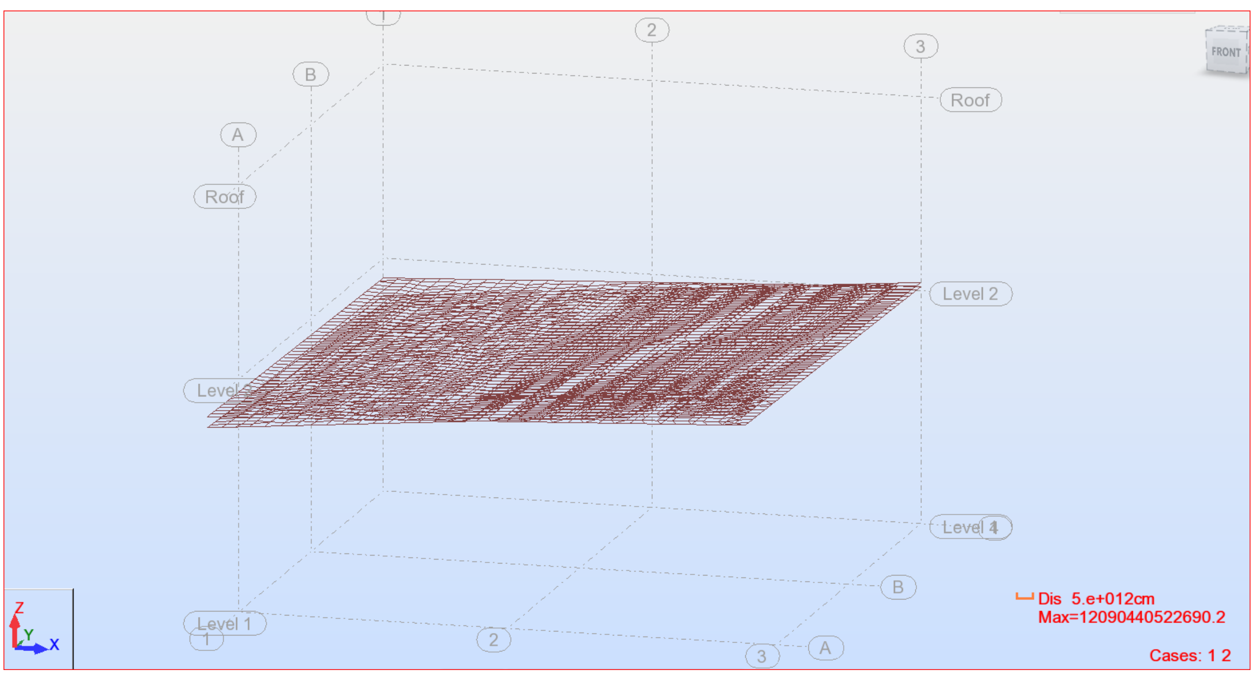

Figure 56: Deformation

5.3.2 Summary

After analyzing all maps it has been found that the floor behaving as an orthotropic plate fails in shear at the corners where it transfers the load to wall through its nodes. The floor plates also show deformation when transverse loads acts on it.

5.4 Intercomponent Connections

There are various diverse intercomponent associations thathra architecture style building. Associations range from basic nailing of two continous parts to extraordinarily composed metal-plate connectors, for example, hooks or T-straps. The associations are an indispensable piece of the building, and their capacity is to hold the single person substructures together and to exchange forces between substructures.

The reaction of the intercomponent association with the load is by and large nonlinear and is a function of numerous parameters, for example, wood type, wood thickness, and dampness substance of the wood. It is likewise extraordinarily affected by variability of the materials. Hence, test results will be diverse actually for the same association sort and same wood species.

To incorporate every individual association in the whole model will bring about a tremendous number of degrees of freedom because of the need of refining the finite element network at the region of association points.

Additionally, this would oblige that the majority of the substructures in the model exist in their three-dimensional structure, which will prompt a very large issue.

A vast gathering of the intercomponent associations in this style is made out of the nail associations. Nails are utilized as associating components and properties of the individual nail association focus the conduct of the entire subassembly. Likewise, the associations rehash at certain endorsed frequencies. In this manner, the intercomponent association might be segregated from the structure also investigated as a different substructure. This substructure might be subjected to load and limit conditions reproducing the real physical test, which would be utilized to focus the association solidness in interpretation and pivot. In the end load-deformation curves for the subassembly are built and utilized as the trademark properties of an one-dimensional nonlinear burden avoidance component consequently used to associate the limits of semi superelements and superelements in the full-structure model.

This procedure is like the semi superelement examination. For instance, the corner association between two outer walls is investigated as an average portion of unit width. Load-deformation aspects in shear and torsion are acquired as force-deflection or moment-rotation connections and these serve as an immediate information for the model.

The following intercomponent connections were analyzed:

- Connections at corner between external walls.
- Floor and external wall connections.
- External walls and roof connections.

For each intercomponent connection degree of freedom was identified by analyzing the displacement during application of loads.

Connections at corner between external walls:

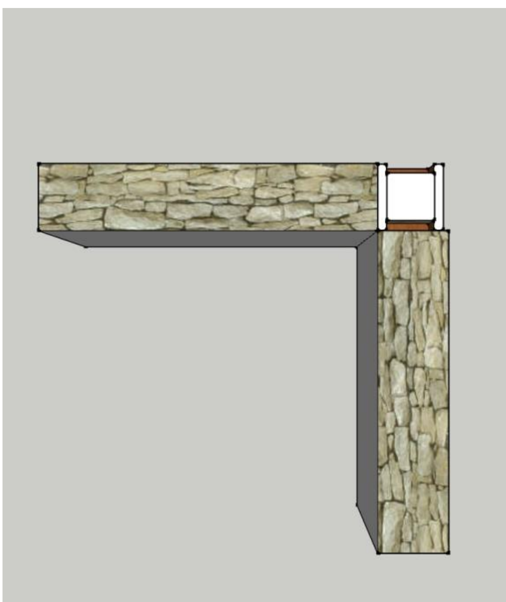

Figure 58: Plan of connection at corner between external walls

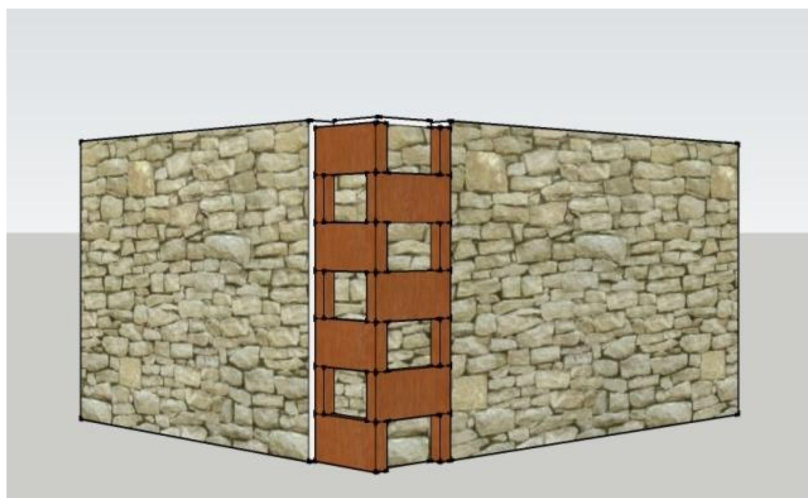

Figure 57: Connection at corner between external walls

In almost all the structures surveyed a thathra existed at the corner. The walls didn't had any connection with the thatharas and behaved as free standing walls. Thus the movements will be 1 in translational and 2 in rotational giving us the total degree of freedom of 3.

Floor and external wall connections:

Figure 60: Connection between floor slab and wall at village Rakh

Figure 59: Connection between floor slab and internal walls at village Khyah

As we can identify from the images above the floor beams are nailed to the wall allowing for translational movement in only single direction. Thus this connection can be considered as a pinned support giving the master degree of freedom as 1.

External walls and roof connections:

Figure 62: Connection between roof and wall in structure at village Bharmour

Figure 61: Connection between a flat roof and wall

Wooden logs rest on the wall with gravity but its movement is restricted in translational direction so this connection is considered as roller support giving the master degree of freedom as 2.

5.5 Full Structure base model

The scientific model is obliged to yield reaction forces and distortions for every shear walls when the structure is stacked by horizontal forces, for example, wind force. Only static loading is considered in this study.

The model of the full building is a composition of superelements speaking of both, the roof and floor, and semi superelements speaking of the walls and intercomponent associations. The semi superelements for the walls and intercomponent associations carry on nonlinearly, although superelements are direct. Besides, the conduct of the semi superelements might be nonconservative, which makes the structural reaction to the cyclic stacking potential study region.

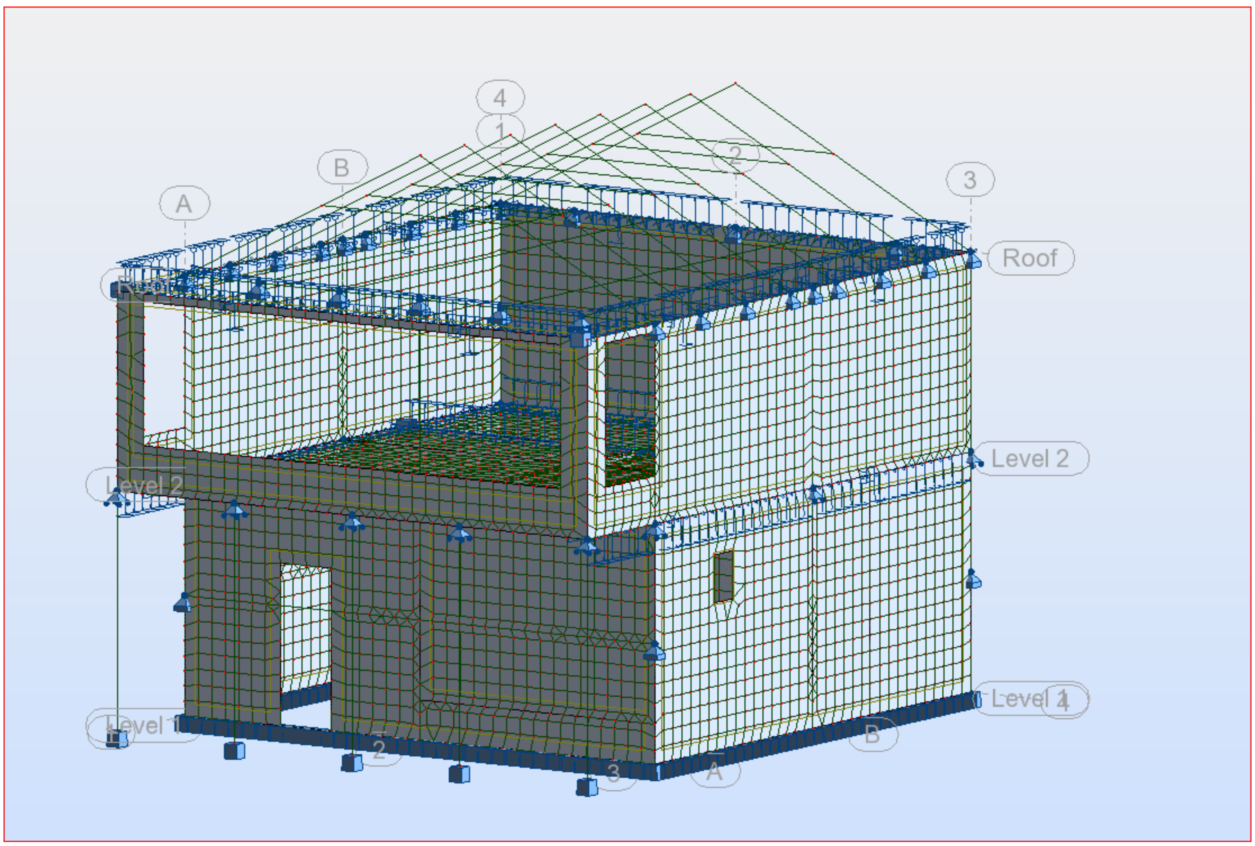

Figure 63: Finite Element mesh of the model

For analysis of the model following boundary conditions were applied:

- Thatras and wooden columns were connected to ground through fixed support.
- Thatra was connected to the beam through pinned support.
- Floors were connected to beam through roller support.

- Floors were connected to external walls through pinned line support.
- Each node in A type truss was connected to each other through pinned support.
- Trusses were connected to beam through pinned support.
- Edges of external walls were connected through pinned line support.
- Beams were connected through pinned support.

5.5.1 Loads and Load Combination

Following loads were applied on the model during analytical analysis:

- Dead Load of the self-weight was applied in –ve Z direction.
- Dead Load of another 1kn/m2 in –Z direction was applied considering of furnitures.
- Live Load of 2kN/m2 was applied in –ve Z direction.
- Wind Load of 1.2kN/m2 was applied according to the regulation: IS:875 (Part5).
- Seismic Load was applied according to IS 1893(part 1) : 2002

5.5.2 Analysis:

Structure type: Shell

Structure geometrical center coordinates:

X = 3.500 (m)

Y = 2.750 (m)

Z = 3.650 (m)

Structure gravity center coordinates:

X = 3.387 (m)

Y = 3.918 (m)

Z = 1.818 (m)

Central moments of inertia of a structure:

Ix = 1058307.947 (kg*m2)

Iy = 1238310.051 (kg*m2)

Iz = 1838016.767 (kg*m2)

Mass = 132094.059 (kg)

Structure description

Number of nodes: 6954

Number of bars: 87

Bar finite elements: 1324

Planar finite elements: 7271

Supports: 468

Cases: 5

Calculation summary

Solution method - SPARSE M

No of static degr. of freedom: 39307

Stiffness matrix diagonal elements

Min/Max after decomposition: 2.910383e-011 1.332455e+011

Precision: -6

Table of load cases / analysis types

Case 1 : DL1

Analysis type: Static - Linear

Potential energy : 3.98593e+011 (kN*m)

Precision : 1.32995e-001

Case 2 : LL1

Analysis type: Static - Linear

Potential energy : 1.18755e-002 (kN*m)

Precision : 9.52601e-001

Case 3 : WIND1

Analysis type: Static - Linear

Potential energy : 7.56150e-004 (kN*m)

Precision : 2.34790e+000

Case 4 : ASCE 7-10 / IBC 2012 Direction_X

Analysis type: Static - Seismic

Excitation direction:

X = 1.000

Y = 0.000

Z = 0.000

Data:

Soil : A

S1 : 0.100

SS : 0.250

Spectrum parameters:

Fa = 0.800 Fv = 0.800

SMS = 0.200 SM1 = 0.080

SDS = 0.133 SD1 = 0.053

To = 0.080 TS = 0.400

TL = 2.000

I = 1.000 R = 1.000

Fundamental period:

Approximated method T = 0.245 (s)

Other structures Ct = 0.02 (0.0488) x = 0.75

Structure range:

Top story Level 6

Bottom story Level 1

Effective height Hn = 8.60(m)

Base shear

Cs = 0.133

Cs max = 0.218

Cs min = 0.010

Effective seismic weight W = 132094.06(kG)

Shear force V = 172.72(kN)

Vertical distribution of seismic forces

Story	Height (m)	Weight (kG)	F(kN)	M(kN*m)
Level 1	0.60	0.00	0.00	0.00
Level 2	3.00	119436.73	0.00	0.00
Roof	2.50	10262.45	0.00	0.00

Case 5 : ASCE 7-10 / IBC 2012 Direction_Y

Analysis type: Static - Seismic

Excitation direction:

X = 0.000

Y = 1.000

Z = 0.000

Data:

Soil : A

S1 : 0.100

SS : 0.250

Spectrum parameters:

Fa = 0.800 Fv = 0.800

SMS = 0.200 SM1 = 0.080

SDS = 0.133 SD1 = 0.053

To = 0.080 TS = 0.400

TL = 2.000

I = 1.000 R = 1.000

Fundamental period:

Approximated method T = 0.245 (s)

Other structures Ct = 0.02 (0.0488) x = 0.75

Structure range:

Top story Level 6

Bottom story Level 1

Effective height Hn = 8.60(m)

Base shear

Cs = 0.133

Cs max = 0.218

Cs min = 0.010

Effective seismic weight W = 132094.06(kG)

Shear force V = 172.72(kN)

Vertical distribution of seismic forces

Story	Height (m)	Weight (kG)	F(kN)	M(kN*m)
Level 1	0.60	0.00	0.00	0.00
Level 2	3.00	119436.73	0.00	0.00
Roof	2.50	10262.45	0.00	0.00

The following map represents the stress developed in whole structure when under dead load. This diagram shows that at the nodal joints of the external walls the stress developed is way beyond limit. It also reflects the failure of wall in shear at midpoints.

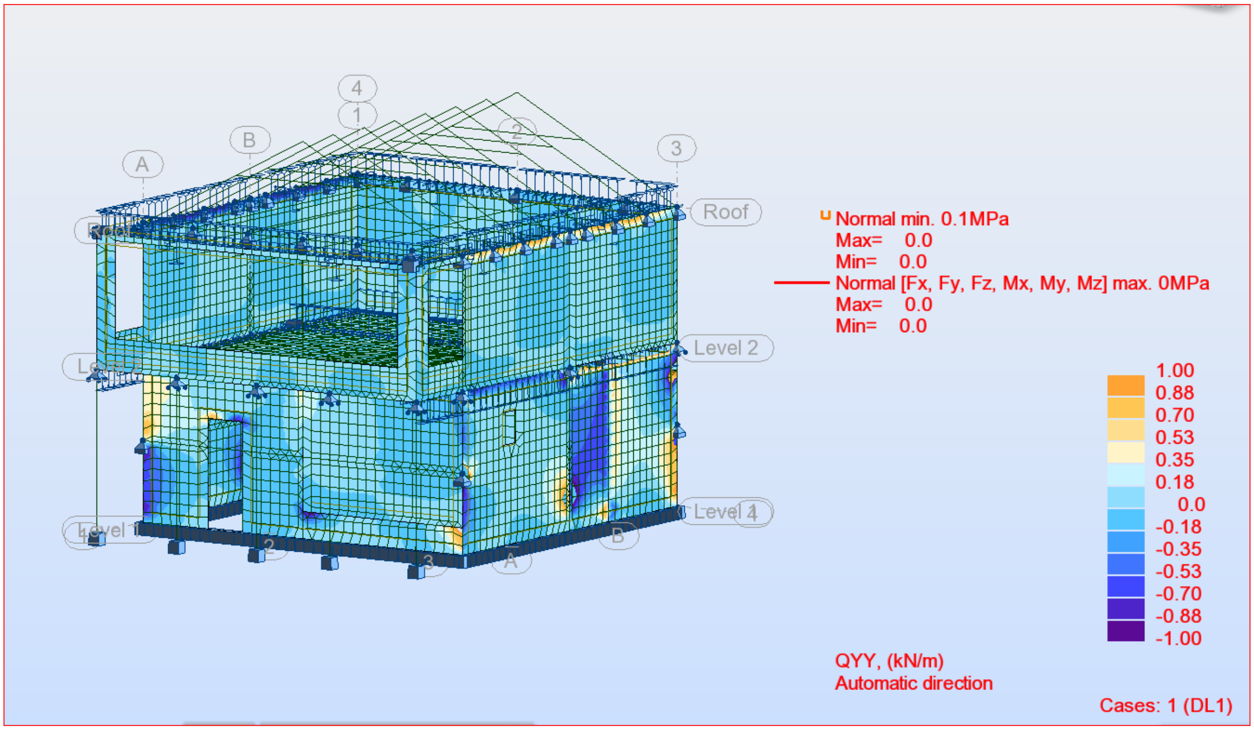

Figure 64: Stress developed in whole structure

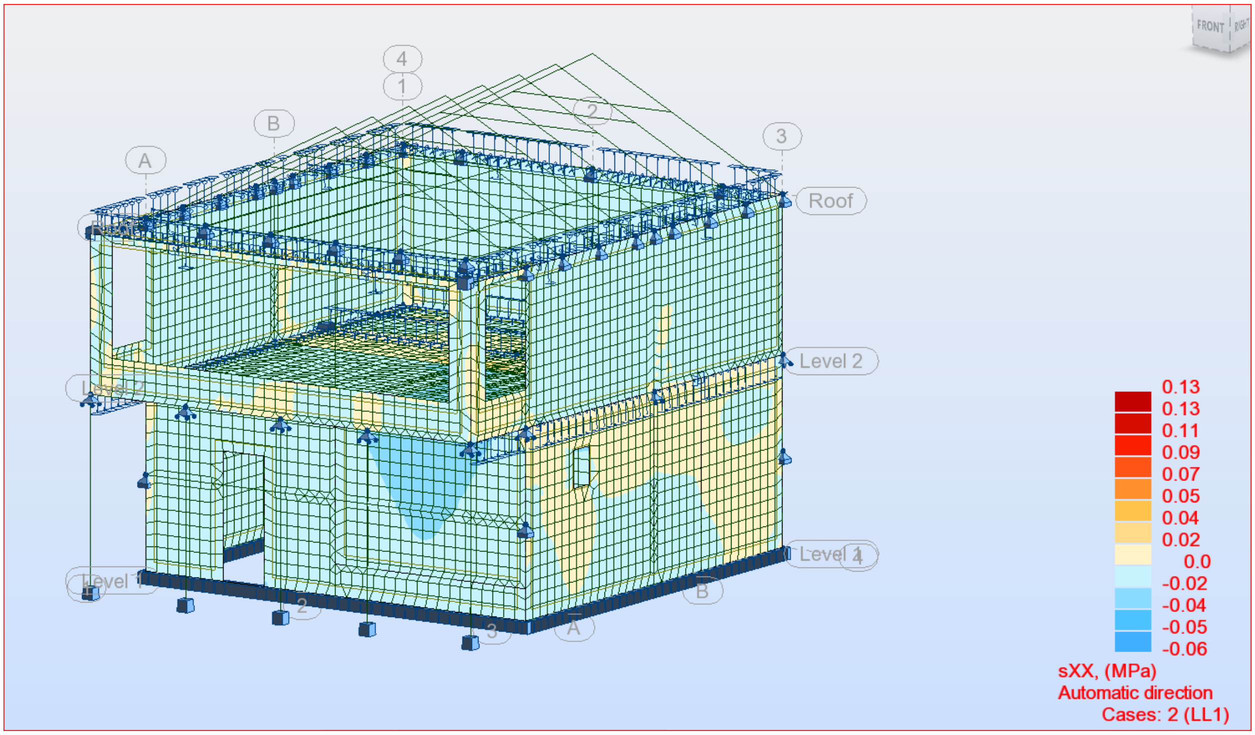

Figure 65: Stress in XX for live load

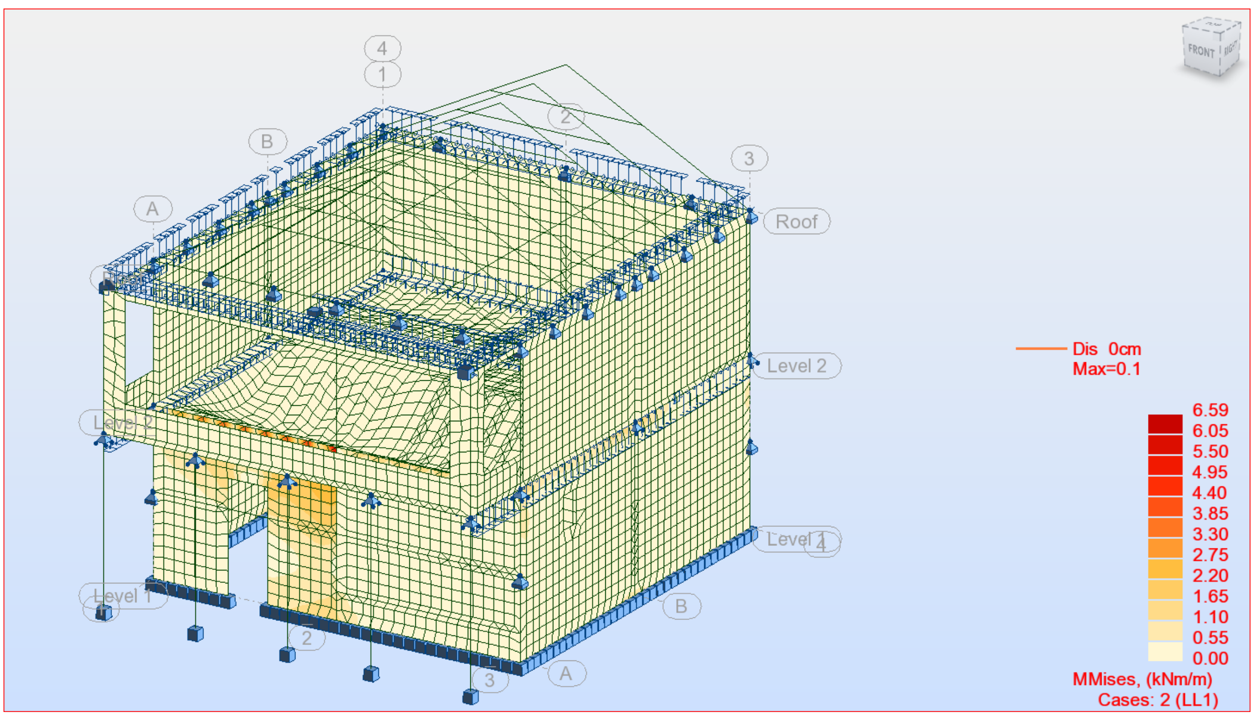

Figure 66: Deformation during Live Load

Failure in torsion and shear as deformation is denoted through this map. It shows that the floor which is considered as an orthogonal plate fails due to the moment created by the live load. This tendency of failure reflects the unidirectional reinforcement by the beams provided.

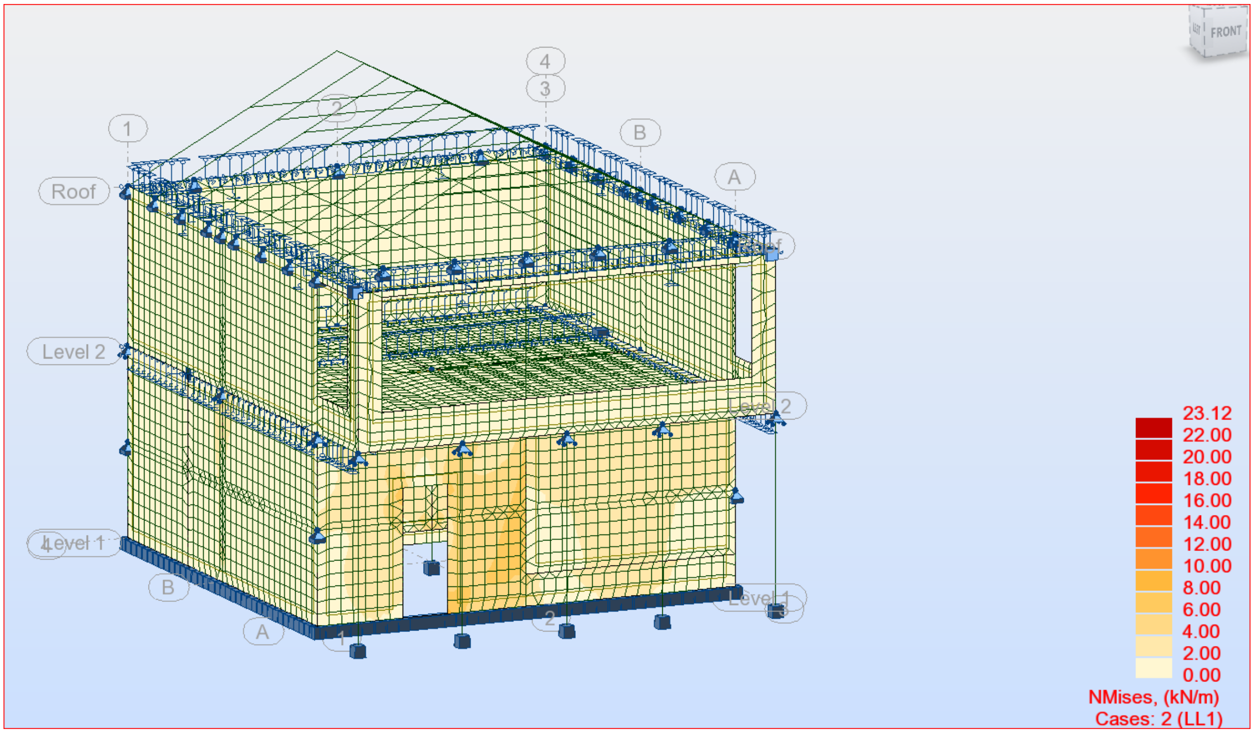

Figure 68: Membrane forces during live load

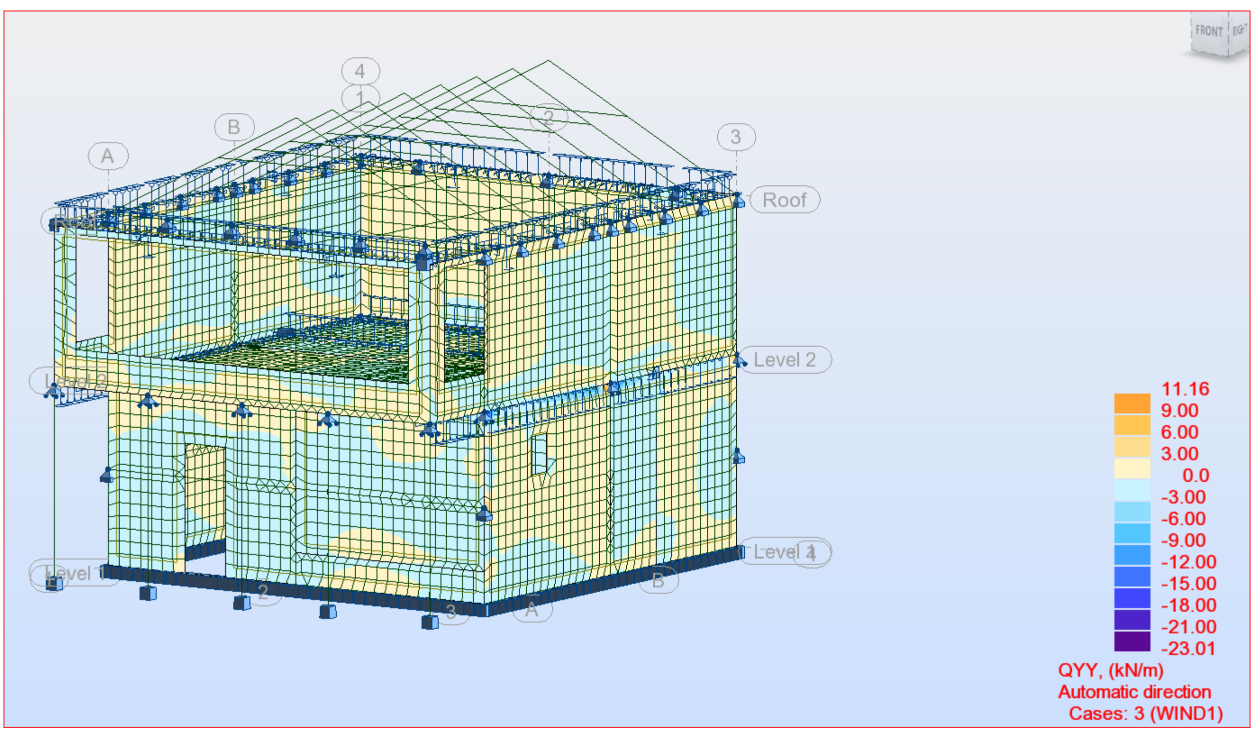

Figure 67: Shear forces in YY for wind load

	Normal [Fx, My, Mz] min.	Normal [Fx, My, Mz] max.	Normal min.	Normal max.	Tau xy max.	Tau xz max.	von Mises max.
	MPa	MPa	MPa	MPa	MPa	MPa	MPa
Case	6 : ULS (1)						
Extreme value	-0.85	0.83	-0.85	0.83	0.15	0.17	0.85
Section position	0.70	0.70	0.70	0.70	0.00	0.85	0.70
Bar	34	34	34	34	77	77	34
Case	7 : ULS+ (1)						
Extreme value	-0.85	0.83	-0.85	0.83	0.15	0.17	0.85
Section position	0.70	0.70	0.70	0.70	0.00	0.85	0.70
Bar	34	34	34	34	77	77	34
Case	8 : ULS- (1)						
Extreme value	-0.85	0.83	-0.85	0.83	0.15	0.17	0.85
Section position	0.70	0.70	0.70	0.70	0.00	0.85	0.70
Bar	34	34	34	34	77	77	34
Case	9 : SLS (1)						
Extreme value	0.00	0.00	0.00	0.00	0.00	0.00	0.00
Section position	0.00	0.00	0.00	0.00	0.00	0.00	0.00
Bar	1	1	1	1	1	1	1
Case	10 : ACC (1)						
Extreme value	-0.85	0.83	-0.85	0.83	0.15	0.17	0.85
Section position	0.70	0.70	0.70	0.70	0.00	0.85	0.70
Bar	34	34	34	34	77	77	34
Case	11 : ACC+ (1)						
Extreme value	-0.85	0.83	-0.85	0.83	0.15	0.17	0.85
Section position	0.70	0.70	0.70	0.70	0.00	0.85	0.70
Bar	34	34	34	34	77	77	34
Case	12 : ACC- (1)						
Extreme value	-0.85	0.83	-0.85	0.83	0.15	0.17	0.85
Section position	0.70	0.70	0.70	0.70	0.00	0.85	0.70
Bar	34	34	34	34	77	77	34

Figure 69: Global extremes of stress analysis

After the stress analysis of the whole structure global extremes are generated for each loading combination. This table gives us the maximum and minimum forces and moments exerted on the structure under different load combination scenario. Through this table it can be said that the structure behavior don't change much over the whole course and it behaves correctly during different load combinations.

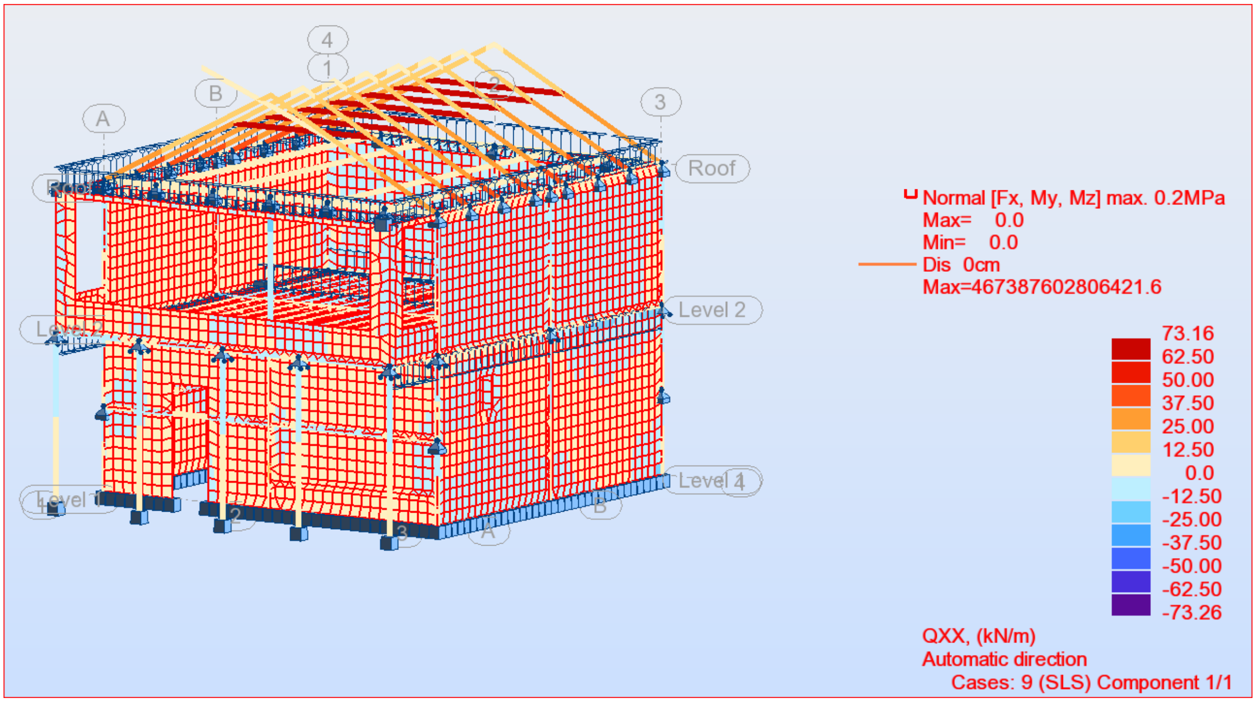

Figure 71: Shear forces during SLS

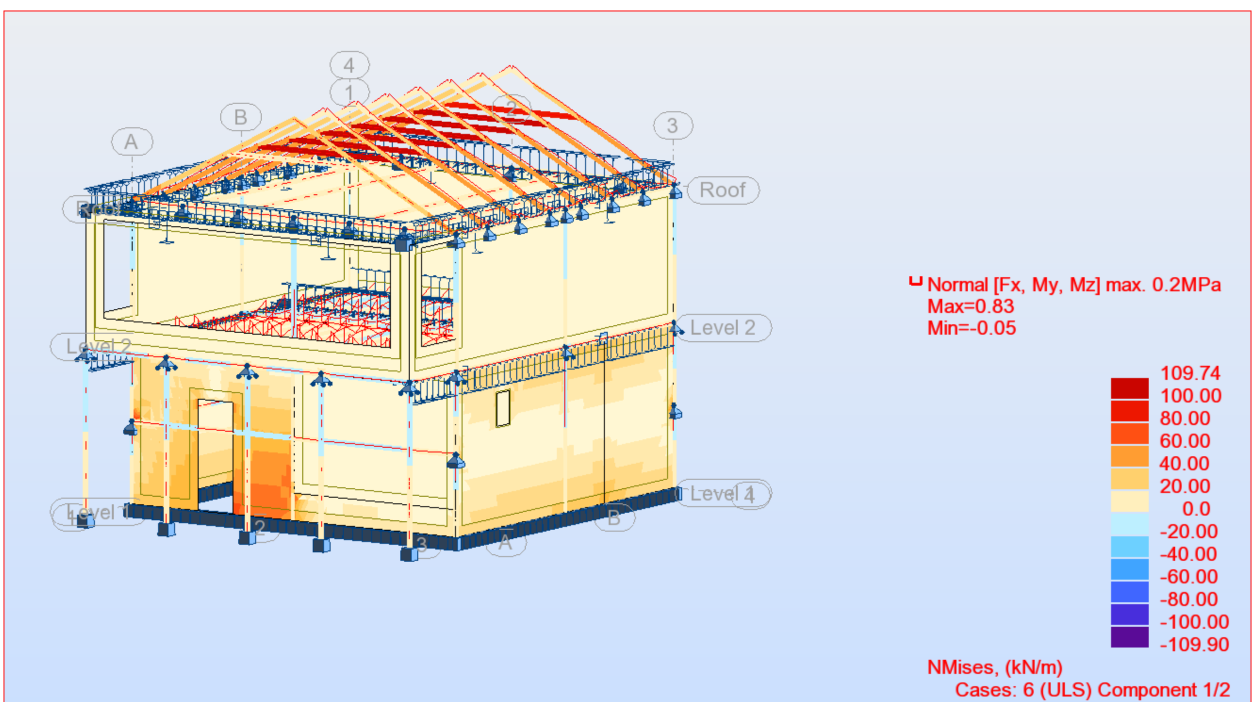

Figure 70: Membrane forces during ULS

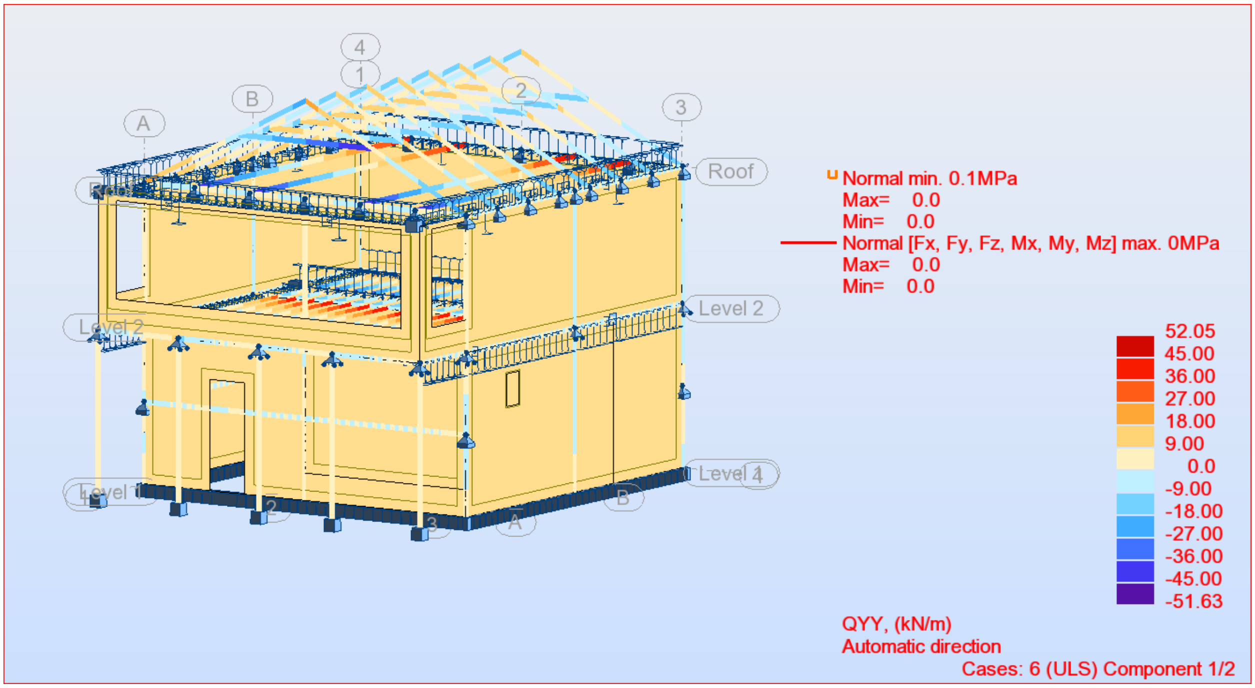

Figure 72: Shear stress during ULS

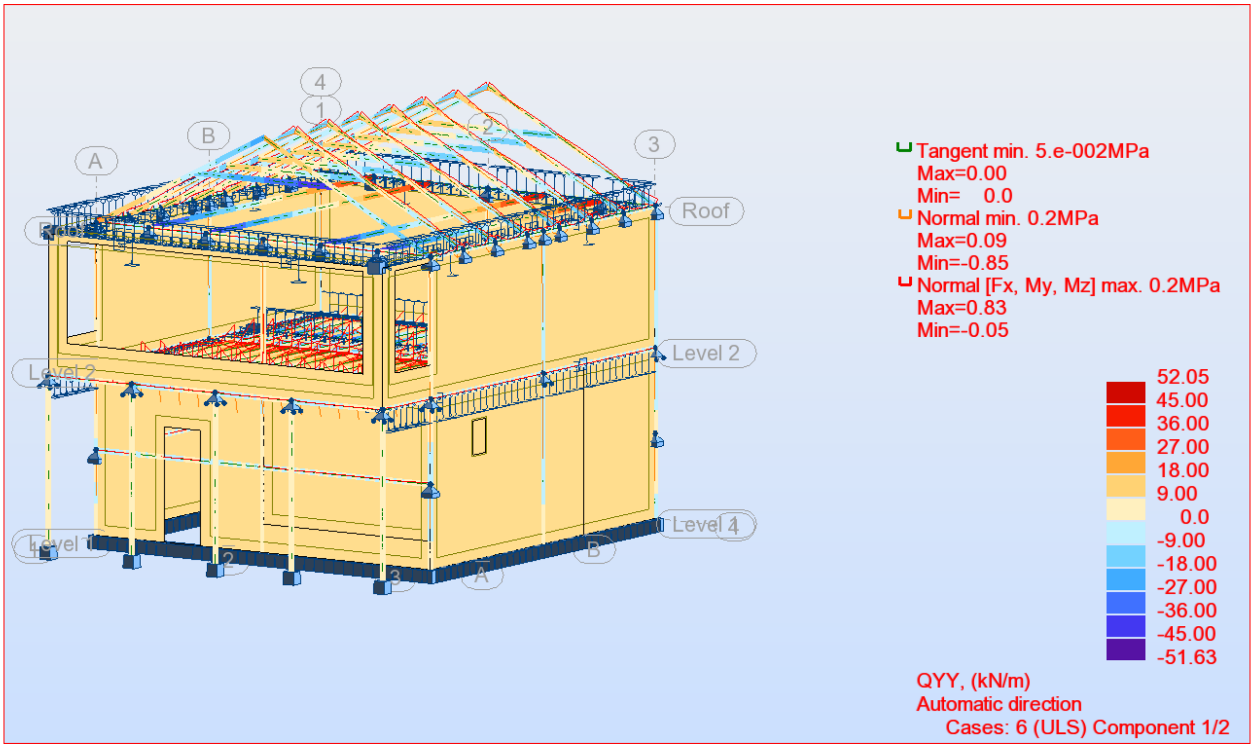

Figure 73: Tangential forces during ULS

The shear force diagram during SLS represents the lateral component of the forces on the structure. Overall the model behaves quite nicely in countering the shear force. The thick 450 millimeter plays its part to perfection in this case. The trusses show their vulnerability as the lower chord attracts the maximum force.

The membrane and tangential force diagram denotes another set of forces acting on the planar nodes. The modal analysis shows the just behavior of the model with only the panel with opening showing tendency of failure.

5.6 Conclusion

After analyzing the results of the structural analysis of 3D nonlinear finite element model of thathra style of architecture following conclusions can be made:

- The connection between beams and columns allows for both translational and rotational motion.
- Due to lack of anchorage out of plane movement of wall is not restricted.
- The stress diagram reflects the excessive torsional bending moment is present at the corners.
- Flexibility of roof and floor is evident through the shear force diagram in SLS load combination.
- Shear stress beyond permissible limit is prevalent in the load transfer system as evident through shear stress map during ULS loading combination.
- Deformation on the first floor level is exaggerated when loading under the prescribed live load thus pointing to lack of shear strength of the beam system supporting the floor.
- After analyzing the map of shear force due to wind load it can be predicted fairly that the walls will fail in shear if the wind pressure increases to 1.5kN/m3.
- The truss system shows signs of failure under various load combinations. This reflects that the pin joints in the truss need to be reinforced and its rotational motion needs to be restricted.
- Each truss behaves differently under the loads as these are not connected properly. Thus generating separate stress in each member which can further lead to failure.
- Openings in the wall attract moments due to closeness to the non-connected corners.

PROPOSAL

Low in-plane and out-of-plane strength of walls

Lack of shear strength in floor system

Lack of anchorage of roof beam and rafters with walls

Lack of lateral support to walls

6 Proposal

After analyzing the results failure of the superelements in the substructure were identified. These failures can be negated by provision of strengthening measures. Following are some structural deficiency and their counter retrofitting measures.

6.1 Low in-plane and out-of-plane strength of walls

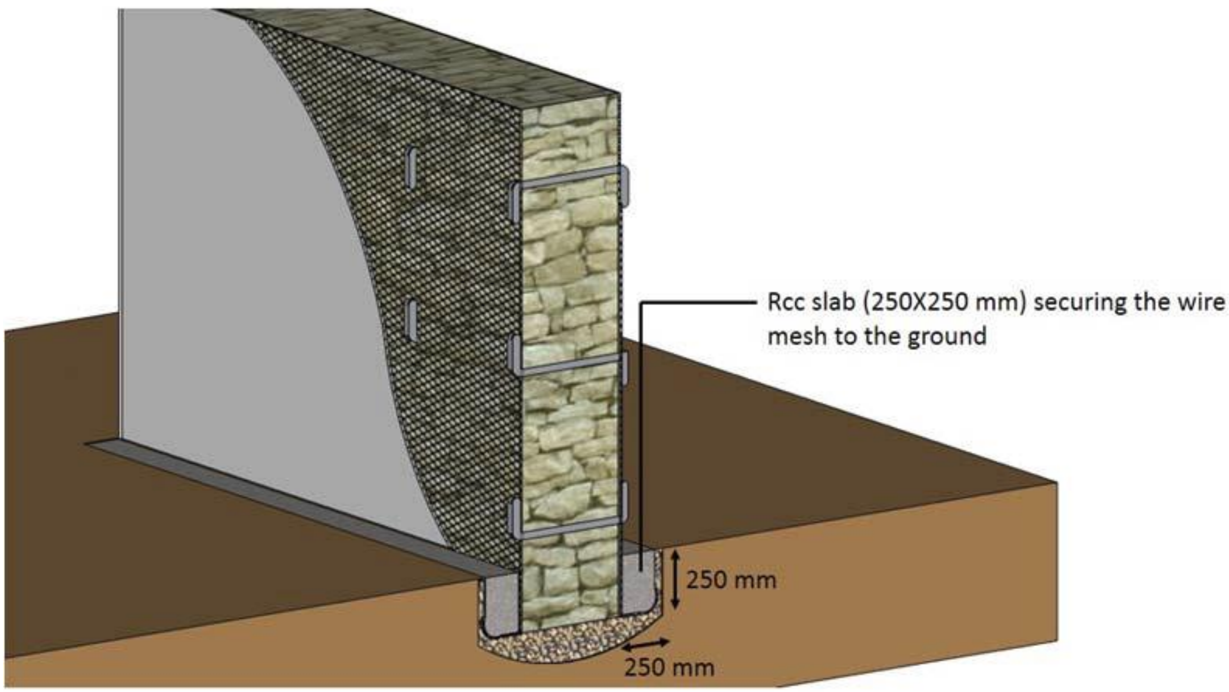

Figure 74: Wall section with retrofitting measures

The walls can be strengthened using ferro-cement (a layer of welded wire mesh grid sandwiched between two layers of cement-sand mortar or micro-concrete), applied on both faces of the dry stone walls, properly interconnected with the wall.

The composite action of the stone wall and ferro-cement is expected to provide adequate in-plane as well as out-of-plane strength. The steel connectors provided at regular intervals to interconnect the two layers of reinforcement on opposite faces of the wall will prevent the splitting of the dry stone walls.

6.2 Lack of shear strength in floor system

The floor system has shown tendency of failure in shear during loading. This can be avoided by providing a different beam system to support the floor. In this system the beams are arranged in criss cross manner thus providing reinforcement in both XX and YY direction.

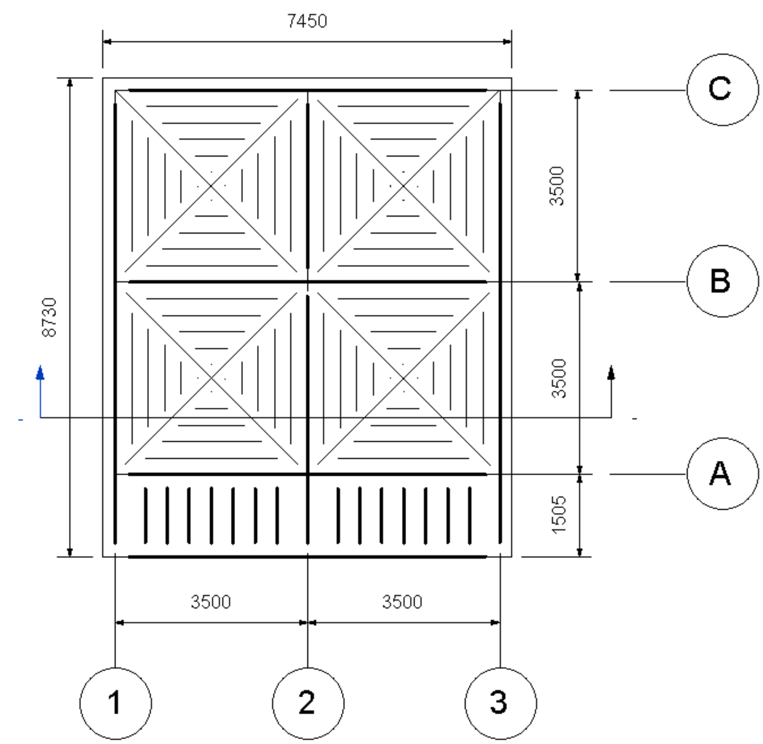

Figure 76: Plan of floor system with beams

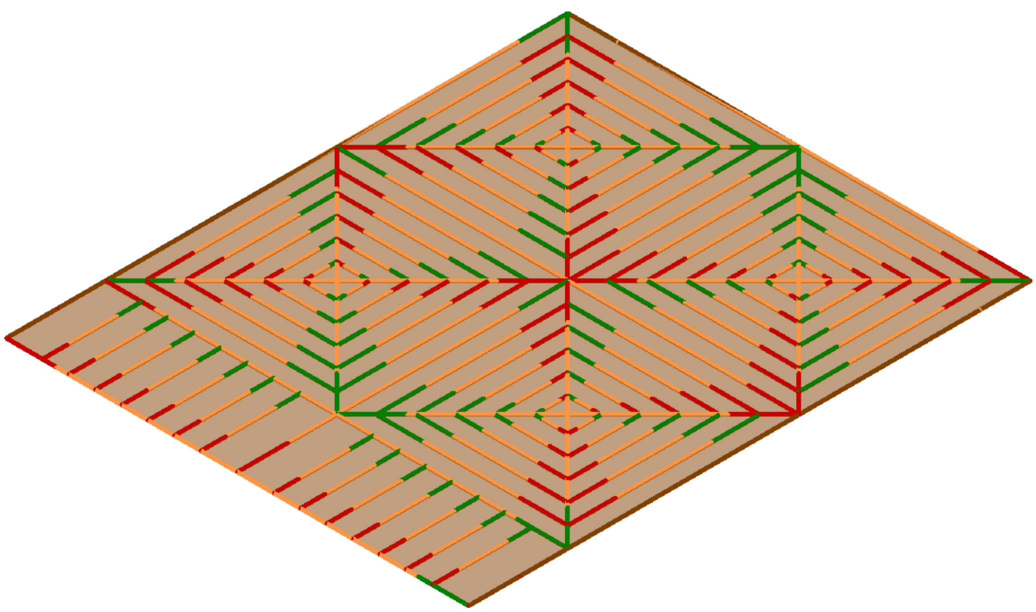

Figure 75: Analytical model of the floor

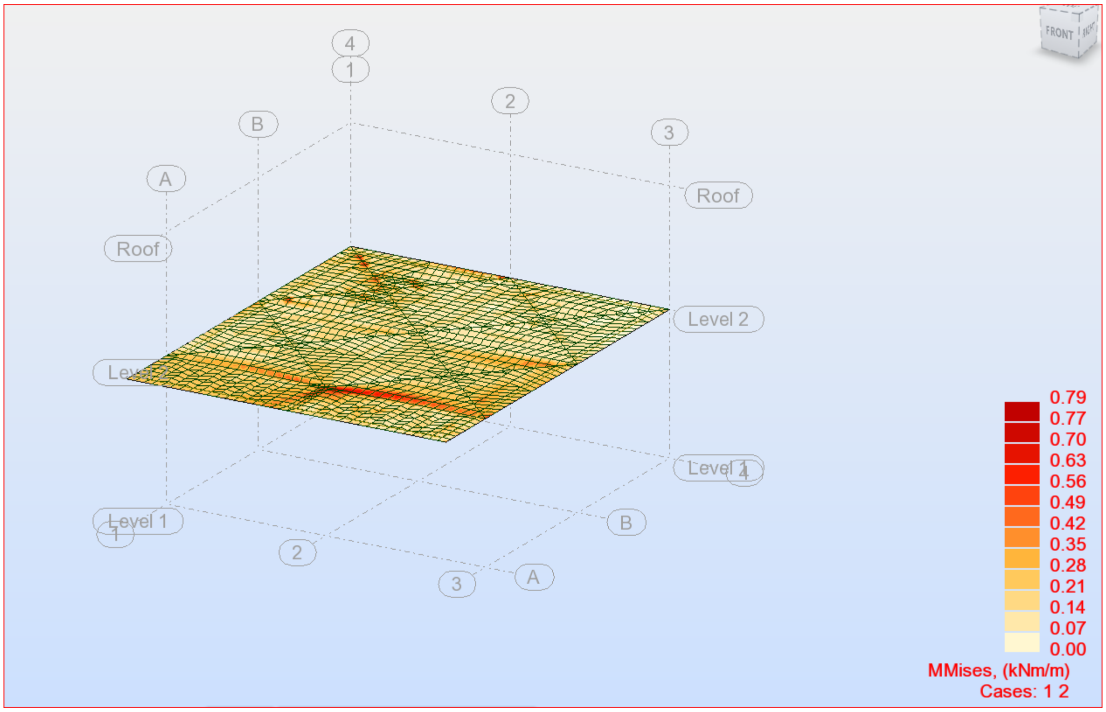

Figure 77: Moment in whole structure proposal

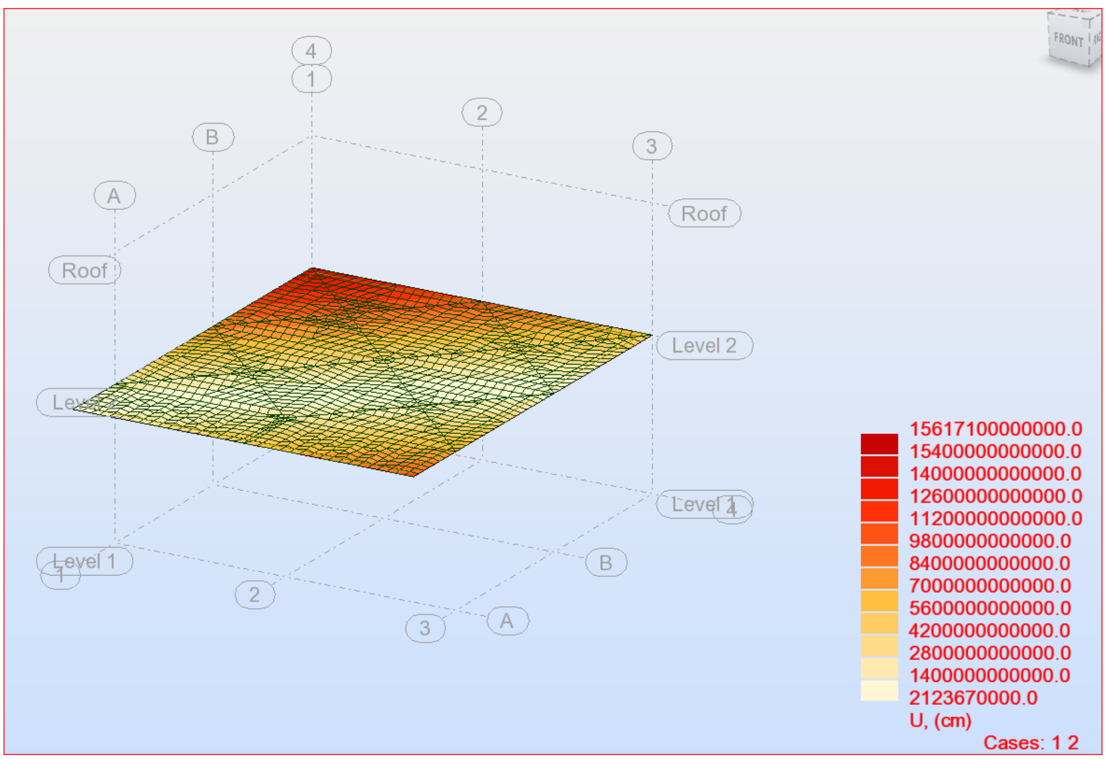

Figure 78: Total displacement in proposal

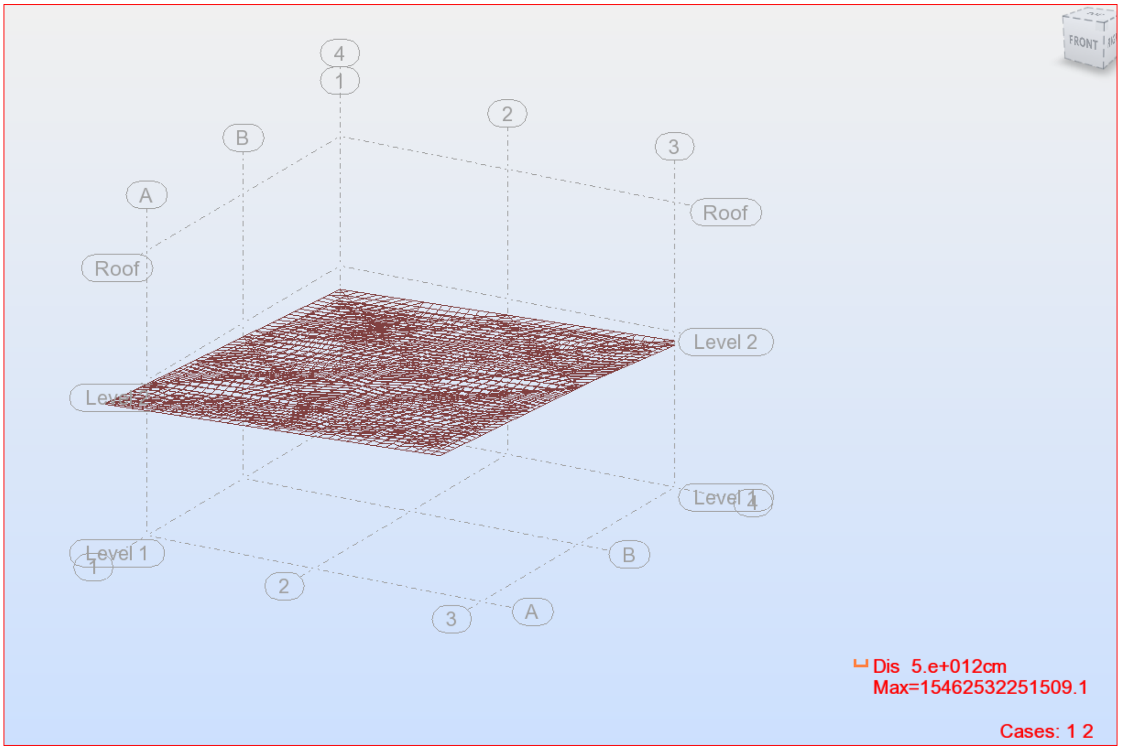

Figure 79: Total active deformation in proposal

After analyzing all the results it can be said that the new beam system has helped in reducing the chances of failure of the floor system.

6.3 Lack of anchorage of roof beam and rafters with walls

For existing construction, the ridge beam and rafters are simply placed on the walls without any positive anchorage. This will cause relative movement of these elements with respect to the walls, leading to the collapse of the roof. Therefore, these elements should be properly anchored to the walls. In the proposed strengthening scheme this can be achieved by using metallic connectors nailed with the wooden members and anchored into the ferro-cement layer or the roof/floor band.

6.4 Lack of lateral support to walls

External band (ties) can be provided at roof level (and also at floor level in case of flexible floor diaphragms) to provide out of-plane support to the walls. The bands may be provided in timber, steel or RC and have to be continuous on all internal and external walls.

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8 Appendix A – Survey Form

PLAN SKETCH

Elevation:

1. SURVEY DETAILS (Form Identifier)

(a) Form no.:

(b) Date:

2. BUILDING DETAILS

(a) Building Name:

(b) Address:____

____

Village/Town/City_______________District___________

________State__________________PIN________________

(c) Year of Construction:

(d) Width of adjoining main road (m):_____

(e) Predominant Use:

☐ Assembly ☐ Office ☐ School ☐ Emergency

☐ Residential ☐ Commercial ☐ Industrial

If Residential, no. of housing units: _______

Is the Building used for Lifeline function?

☐ No Give Details.........................

(f) Minimum distance from adjoining building (m): _____

(g) Visual Condition:

☐ Excellent ☐ Good ☐ Damage

(h) Building on stilts/Open ground floor:

☐ Yes ☐ No

☐ <25% ☐ 25% - 50% ☐ >50%

(i) Construction drawings available:

☐ Yes ☐ No

3. GENERAL INFORMATION

(a) Site Morphology

Site Morphology (Select All Applicable)					
☐ Flat	☐ Crest	☐ Embankment	☐ Downward Slope	☐ Trough	☐ Adjacent to Hill Slopes

(b) Soil

Soil Type				**Soil Nature**		
☐ Hard	☐ Medium	☐ Soft	☐ Not Known	☐ Expansive	☐ Not Expansive	☐ Not Known

Liquefaction Potential		
☐ Liquefiable	☐ Not Liquefiable	☐ Not Known

(c) Foundation

Foundation Type		
o No Foundation	o Trench filled with dry stone	o Dry stone packing
o Random rubble masonry strip	o Regular masonry strip in mud mortar	o Regular masonry strip in lime/cement mortar
o Pile/well foundation	o Beam on columns	o Other (specify)

(d) Typology

Material	Type of Load-Bearing Structure	Type of Roof/floor system	
Mud	Rammed Earth Wall	Heavy sloping roofs	
		Thatch/light weight sloping roof	
		Others (specify)-	
	Sundried brick/block walls	Heavy sloping roofs	
		Thatch/light weight sloping roof	
		Flat roof on wooden girders/planks	
		Others (specify)-	
	Stabilised Earth walls	Heavy sloping roof	
		Thatch/light weight sloping roof	
		Flat roof on wooden girders/planks	
		Others (specify)-	
	Mud walls with timber framing	Heavy sloping roof	
		Thatch/light weight sloping roof	
		Flat roof on wooden girders/planks	
		Others (specify)-	
Timber	Timber frame with timber plank partitions	Light weight sloping roof	
		Heavy/stone sloping roof	
		Others (specify)	
	Wooden frame with 'ekra'/bamboo/light partitions	Light weight sloping roof	
		Heavy /stone sloping roof	
		Others (specify)	
	Dhajji-Diwari	Light weight sloping roof	
		Heavy /stone sloping roof	
		Others (specify)	
	Thatra with timber plank partitions	Light weight sloping roof	
		Heavy /stone sloping roof	
		Others (specify)	
	Thatra with Dhajji-Diwari partitions	Light weight sloping roof	
		Heavy/ stone sloping roof	
		Others (specify)	
	Thatra with other partitions (specify)	Light weight sloping roof	
		Heavy /stone sloping roof	

		Others (specify)	
	Kath-Kunni walls with stone packing	Light weight sloping roof	
		Heavy/ stone sloping roof	
		Others (specify)	
Bamboo	Bamboo frames with Bamboo/Ekra/ straw partitions 'Bunga'	Thatch	
Stone	Dry stone walls	Light weight sloping roof	
		Heavy /stone sloping roof	
		Flat heavy mud roof on wooden/steel girders	
		Flat RC/RB roof	
		Others (specify)	
	Random Rubble in Mud mortar	Light weight sloping roof	
		Heavy/ stone sloping roof	
		Flat heavy mud roof on wooden/steel girders	
		Flat RC/RB roof	
		Others (specify)	
	Random Rubble in cement mortar	Light weight sloping roof	
		Heavy/ stone sloping roof	
		Flat heavy mud roof on wooden/steel girders	
		Flat RC/RB roof	
		Others (specify)	
	Dressed (Regular shaped) stone masonry in mud mortar	Light weight sloping roof	
		Heavy stone sloping roof	
		Flat heavy mud roof on wooden/steel girders	
		Flat RC/RB roof	
		Jack-Arch Roof	
		Others (specify)	
	Dressed (Regular shaped) stone masonry in lime mortar	Light weight sloping roof	
		Heavy/ stone sloping roof	
		Flat heavy mud roof on wooden/steel girders	
		Flat RC/RB roof	
		Jack-Arch Roof	
		Others (specify)	
	Dressed (Regular shaped) stone masonry in cement mortar	Light weight sloping roof	
		Heavy/ stone sloping roof	
		Flat heavy mud roof on wooden/steel girders	

		Flat RC/RB roof	
		Jack-Arch Roof	
		Others (specify)	
	Massive (thick) stone masonry walls/Historical buildings	Domes	
		Jack-Arch Roof	
		Others (specify)	
Brick	Brick masonry in mud mortar	Light weight sloping roof	
		Heavy stone/tiled sloping roof	
		Flat heavy mud roof on wooden/steel girders	
		Flat RC/RB roof	
		Jack-Arch Roof	
		Others (specify)	
	Brick masonry in Lime mortar	Light weight sloping roof	
		Heavy stone/tiled sloping roof	
		Flat heavy mud roof on wooden/steel girders	
		Flat RC/RB roof	
		Jack-Arch Roof	
		Others (specify)	
	Brick masonry in Cement mortar	Light weight sloping roof	
		Heavy stone/tiled sloping roof	
		Flat heavy mud roof on wooden/steel girders	
		Flat RC/RB roof	
		Others (specify)	

(e) Workmanship and maintenance

Workmanship			Maintenance		
☐ Good	☐ Poor	☐ Not Known	☐ Good	☐ Poor	o Abandoned

(f) Other

Building Built on Split level	☐ Yes	☐ On hill slopes	☐ Entrance on higher slope
		☐ Partial basement	☐ Entrance on lower slope
	☐ No		
Housing Type	☐ Row	☐ Middle	☐ Corner
	☐ Isolated		

Building in contact with soil	Yes	Depth of soil retaining		Structural system supporting Soil	
		Down-hill side	☐	RC Shear Wall	☐
		Up-hill side	☐	Masonry Shear Wall	☐
		Sides	☐	Other:	☐
	No				

4. STRUCTURAL FEATURES

Thickness of Walls (no. of withes)			Presence of through-stones in random rubble masonry		
O Single-leaf (wythe)	O Double-leaf (wythe)		O Yes	O No	O N/A
Presence of arches/vaults without ties or buttresses			**Presence of ties/braces in sloping roofs**		
O Yes	O No		O Yes	O No	O N/A
Stones/tiles anchored to purlins			**Diagonal planks/braces in wooden floor diaphragms**		
O Yes	O No	O N/A	O Yes	O No	O N/A

5. Connections

Good connections between walls at corners			Lintel band/horizontal wooden members present			Roof band/ties present			Good connections between roof/floor slab and walls		
Yes	**No**	**Not known**	**Yes**	**No**	**Not known**	**Yes**	**No**	**Not known**	**Yes**	**No**	**Not known**

6. ARCHITECTURAL FEATURES

Different storey heights present?		Large open central courtyard present		Large door/window openings present		Door/window openings close to corners present		Re-entrant corners present	
Yes	No	Yes	No	Yes	No	Yes	No	Yes	No

Walls distributed symmetrically about one axis		Walls symmetrically distributed about both axes		Irregular orientation of rooms		Floating walls about one axis present		Floating walls about both axes present	
Yes	No	Yes	No	Yes	No	Yes	No	Yes	No

Long walls without cross walls present		Location of staircase eccentric about one axis		Location of staircase eccentric about both axes		Staircase headroom present		Overhead water tank present		
Yes	No	Yes	No	Yes	No	Yes	No	Yes	No	Location:______________

7. BUILDING DESCRIPTION (Overall metrics, age, use and intensity of use)

No. of stories *		Average inter-storey height (m) Ground Floor	Average inter-storey height (m) Other Stories	Average floor * area (m^2)	Utilisation of floor area(%)	Maximum * Number of occupants
Stories below Road Level	**Stories above Road Level**	<2.5	<2.5		< 25	**Season**
1	1	2.5-3.0	2.5-3.0		25-50	Day:
2	2	3.0-3.5	3.0-3.5		50-75	Night:
3	3	3.5-5.0	3.5-5.0		> 75	
		> 5.0	> 5.0		Abandoned	**Unseason**
					Unfinished	Day:
						Night:

Period of Occupancy (y/m m/d d/h)

Year of last structural repair (if any)

8. GEOMETRICAL CHARACTERISTICS

a) Projections

- Chimneys
- Parapet
- Cladding
- Balconie
- Sunshade
- Communication Towers
- Other

If others, give details____________________

Largest horizontal projection= _____mLargest vertical projection = _____m

b) Plan irregularity

Parameter	
Lateral load-resisting elements not parallel to orthogonal axis	☐
Re-entrant corners (>15% of plan dimension)	☐
Diaphragm discontinuity (cut/open area > 50% of gross area, and/or floor to floor variation in diaphragm stiffness)	☐
Out-of-plane offsets of lateral load-resisting elements	☐
Lateral load-resisting elements not symmetric about orthogonal axis	☐

c) Vertical irregularity

Parameter	
Open storey (Relative)	☐
Mass irregularities	☐
Geometrical irregularity (horizontal dimension of lateral load-resisting system in a storey is >150% of adjacent storey)	☐
Discontinuity in vertical members over height	☐
Vertical stiffness irregularities	☐
Staggered floors (offset floor diaphragm)	☐

9. HISTORY OF EARLIER DAMAGE

Yes No Not Known **If Yes,**

☐ Any storey/building noticeably leaning

☐ Cracks at beam-column junction

☐ Beam Distress - Vertical/Diagonal cracks near supports/mid span

☐ Are the slabs exclusively deflected?

☐ Cantilever slabs are damaged?

☐ Cracks in Staircase

☐ Out of plane failure of infill walls

If others, give details______________________________

10. **STRUCTURAL STRENGTHENING/REHABILITATION**

☐ Yes ☐ No ☐ Not Known

11. **IMPORTANT OBSERVATIONS AND COMMENTS**

12. **Photograph Nos.**

STRUCTURAL METRICS

Wall index	Direction	Length (m) L_{wall}	Floor 1		Floor 2		Floor 3		Floor 4		No. floors with identical wall	No. floors adjoining wall	Buttresses/ Arches supported	No. internal walls connected	Good connection with lateral walls	No. of openings where $L_1<L_2$
			Thickness (m)	L_{pier}/L_{wall}	Thickness (m)	L_{pier}/L_{wall}	Thickness (m)	L_{pier}/L_{wall}	Thickness (m)	L_{pier}/L_{wall}						

Structural Plan Density:

X direction: $$\frac{\text{Total area of bearing walls in x direction}}{\text{Plinth area}} \times 100$$

Y direction: $$\frac{\text{Total area of bearing walls in y direction}}{\text{Plinth area}} \times 100$$