DESIGN OF MULT-STOREY BUILDING WITH TIMBER USING MANUAL AND TEKLA SOFTWARE: A COMPARATIVE ANALYSIS

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CERTIFICATION

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APPROVAL

This research work is presented to the Department of civil Engineering, Nnamdi Azikiwe university Awka for approval in partial fulfillment for the award of Bachelor's Degree In Engineering(B.ENG), in the Department of Civil Engineering.

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DEDICATION

This research is dedicated to the almighty God for his grace, wisdom, guidance strength provisions throughout the period of this work.

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ABSTRACT

This research was undertaken to evaluate the difference in the computational results obtained from software aided design and manual design method of designing multi-storey timber building. To accomplish this an office building plan, storing 30 offices was taken for the design. The Nigerian timber specie chosen was Danta specie, a hardwood of N2 strength class and for the CAD design, a timber of the hardwood class having a strength class of D50 was chosen. The design code employed for both method was Eurocode 5. The analysis of the main structural elements of the building, the joist, beam and columns, were analyzed and designed providing the required sections.The analysis and the result showed that the value of the structural analysis in the manual design are higher thereby providing higher sizes of sections and the cost analysis proved the CAD design to be more cost effective.

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CHAPTER ONE

INTRODUCTION

1.1 Background of Study

Timber, as a structural material, possesses distinct characteristics compared to steel, reinforced concrete, or other composites. It is a natural material originating from biological sources, exhibiting highly variable properties. Wood is orthotropic, meaning its properties vary in different directions. Additionally, wood is hygroscopic, indicating that its moisture content continually changes with the relative humidity of the surrounding environment. Unlike the well-established procedures used for manufacturing concrete or steel, where specific grades can be obtained with controlled variability, timber selection involves verifying grades through non-destructive strength grading of sawn timber. This process involves sorting existing materials that have formed over an extended period and often under diverse conditions, based on statistical relationships. Sawn timber is produced from a wide range of logs obtained from various trees, often grown in different cultural conditions and with long rotation periods.

Timber has a long history of use in construction predating the development of concrete and steel as structural components, dating back centuries. As time progressed, the use of timber in construction improved, leading to the dominance of modern engineered timber products in our present-day society. These engineered timber products enable structural engineers to achieve the performance and efficiency required in the 21st-century construction industry. The term "timber" refers to the structural product derived from wood, which can be categorized as either "softwood" or "hardwood." Softwood is obtained from coniferous trees, while hardwood comes from broad-leaved trees. The terms "softwood" and "hardwood" are botanical distinctions and do not necessarily indicate the density or hardness of the wood. Softwood is commonly used in timber structures due to its accessibility, efficient utilization, relatively low cost, and a constant supply from regenerated forest areas with high growth rates. Hardwoods, on the other hand, are typically utilized in exposed frameworks and cladding where durability, specific aesthetics

The strength of sawn timber depends on factors such as its nature, width, size, member shape, moisture content, and length of loading, as well as strength-reducing

characteristics like grain slope, knots, fissures, and wane. Strength grading methods have been developed to differentiate timber using visual force grading methods or machine strength grading methods. Timbers with similar strength properties are grouped into sets with resistance groups described in BS EN 338:2009. This simplifies the design and development process by allowing projects to be based on specified strength class limits without needing to classify and source a particular mixture of species and grades. The strength groups are described as 'C' (coniferous) for softwoods and 'D' (deciduous) for hardwoods.

1.2 Problem Statement

The increasing construction costs present a challenge for engineers to produce economically sound designs while maintaining standards. An economical design holds significant value in all fields of engineering and is highly desirable for implementing construction projects. Using manual design methods often yields different results compared to structural design software such as Tekla, Stadpro, and Prota Structure.

1.3 Aim and Objectives

This research aims to achieve the following objectives:

1. Outline the design procedures for the main structural elements using Eurocode 5.

2. Outline the design procedures for the main structural elements using Tekla software.

3. Highlight the deviations between the results obtained from manual design and Tekla software using Eurocode 5, specifically in terms of their economic viability.

1.4 Scope of Study

This research will focus on the design methods of Eurocode 5 and Tekla software for the structural elements in a multistory timber structure. To achieve the research objectives, the following scopes will be considered:

1. The same architectural drawing will be used for both design methods to compare the required timber sections, revealing which approach is more economical.
2. The differences in design procedures for structural elements using Eurocode 5 and Tekla software will be examined.
3. A comparison will be made between the design results of Eurocode 5 and Tekla software in terms of the required timber sections and connections.

1.4 Significance of the Study

This holds substantial significance as it sheds light on the outcomes obtained from comparing software aided design with manual design techniques for timber

1. provides engineers, especially those involved in consultancy, with valuable insights into the potential results achieved through software aided design and manual design of timber structures

2. Assist engineers in making informed decisions regarding the preferred approach for designing timber structures, enabling them to choose a more effective method.
3. Help to bridge the gap between theoretical knowledge and practical application for civil engineering students and researchers, offering them valuable resources to understand the practical implications of design methodologies for timber structures.

CHAPTER TWO

LITERATURE REVIEW

2.1 Timber as a Construction Material

Timber is a rigid fibrous material originating from plants, refers to wood that is specifically processed for use in construction. In the field of construction, timber is often used interchangeably with the term "wood." Timber is categorized into two main types: hardwood and softwood. Hardwood is sourced from broad-leaved trees like Iroko, Mahogany, and Danta, which belong to the angiosperm family. On the other hand, softwood is obtained from coniferous trees that have needle-shaped leaves. Common examples of softwood trees include Scots Pine, Norway Spruce, and Douglas Fir. (Abimaje and Adams 2014)

Timber, when used in construction, refers to wood that is suitable for carpentry, joinery, or manufacturing purposes. This includes both standing trees and felled trees that can be transformed for the aforementioned uses. Throughout human history, timber has played a vital role. In ancient times, it served as a primary resource for building shelters such as huts, crafting weapons like spears and bows, and facilitating transportation through canoes, ships, and log bridges. Even today, timber remains an essential material for structural purposes. Thanks to advancements in technology, development, and engineering, innovative methods have emerged for working with timber. (aguwa james,2016)

In recent years, there has been a growing trend in the building industry towards incorporating timber as a primary material in the construction of tall buildings. This shift is fueled by multiple factors, including the emergence of new engineered timber products and the enticing economic advantages of using prefabricated timber components and composite building systems. However, what truly motivates architects, building owners, government entities, and other stakeholders to embrace timber construction is the profound understanding of its potential sustainability benefits. The desire for environmentally friendly and sustainable architecture has become a driving force behind the increasing preference for timber in high-rise buildings. Ramage et al, (2017),

Timber stands as the most significant renewable resource within the realm of building materials. As a building material, timber possesses inherent characteristics that determine its application in construction.

2.1.2 Benefits of Timber to the Construction Industry

The following are some of the reasons for utilizing timber in building construction (Atlantic Cladding, 2018):

1. **Ecology and Sustainability:** Timber is a truly sustainable and renewable building material. It can be easily replaced due to the practice of replanting new trees whenever timber is harvested. This ensures a continuous availability of timber.
2. **Low Production Energy:** Converting trees into timber for construction requires minimal energy, resulting in low embodied energy in timber.
3. **Excellent Insulation**: Timber possesses natural insulating properties, allowing for more space for insulation compared to brick buildings. It acts as a natural thermal insulator.

4. **Ease of Work**: Timber is a versatile material that can be used in various ways and is easy to install.
5. **Design Flexibility**: There are no limitations to the design and size possibilities when using timber in construction.
6. **Durability and Easy Maintenance:** Treated timber is durable, easy to maintain, and allows for quick building times.
7. **Fire Retardant:** Certain types of treated timber are fire retardant, delaying the ignition time and slowing down the burning process.

In addition to the immediate performance benefits of timber construction, its popularity is driven by the growing awareness of climate change and environmental consequences. Government policies supporting timber construction encompass planning, forestry, sustainable development, and climate change. Using local timber supports the economy, woodland management, and increases the demand for timber products. Furthermore, the origin of the timber can generate additional interest in a building's design (Davis, 2016).

Regarding types of timber construction, there have been advancements over time. Light timber frame construction, typically used in low- and mid-rise residential buildings, involves smaller-section stud members forming wall and floor assemblies.

On the other hand, heavy timber frame construction, utilized in mid- to high-rise residential and commercial applications, incorporates larger-section engineered timber products for the building superstructure. Heavy timber frame construction allows for greater design flexibility, including longer unsupported spans, open-plan areas, and taller structures (Barber & Gerald, 2019).

Timber possesses various physical properties as a construction material. It exhibits thermal properties such as minimal expansion with heat, high specific heat, and sound absorption capabilities. Wood is also an excellent electrical insulator, especially when dry, and possesses remarkable mechanical strength. Aesthetic properties make wood a decorative material, and its oxidation characteristics differ from metals. Working with wood is easy in terms of repair and maintenance, and the variation in types of woods allows for suitability based on specific needs (Prof. Dr. Ramadan Ozen).

2.2 Design of Timber Structures and Buildings

Timber has the potential to play a vital role in creating energy-efficient and resource-conscious structures. This aligns perfectly with the principles of sustainable building design. However, it is important to use timber appropriately, as its mere inclusion does not automatically ensure a sustainable outcome.

When utilized incorrectly, timber can actually lead to less sustainable results compared to using steel or concrete in building design and construction. Nevertheless, timber offers significant advantages beyond its impact on embodied carbon, including reduced construction time and minimized labor costs. (Matthew Caldwell 2021)

1)Timber can be easily and accurately machined into precise elements.

2)Large timber components are comparatively lightweight.

3)Timber can be efficiently transported and quickly assembled.

4)Construction time on-site is drastically reduced.

5)The labor force required for erecting timber structures is relatively small.

6)The lower weight of timber structures saves materials in foundations, facilitates construction on challenging sites, or allows for additional stories

As one of the fastest-growing economies in the world, Nigeria is faced with numerous ongoing projects, both technological and infrastructural, undertaken by governments and the private sector. These projects range from small-scale to large-scale, all aiming to meet the high demand caused by the country's large population and the significant rate of rural-urban migration (Ede and Okundaye, 2014). Despite the efforts and massive investments made, the ultimate goal of providing affordable housing for the masses remains a challenge. Therefore, there is a need to explore new ideas that promote the adoption of locally available building materials, leading to more affordable rural and urban structures accessible to the masses.

This project aims to explore the feasibility of using timber as a reliable alternative to labor-intensive and environmentally unfriendly concrete technology, which has been predominantly used in residential building construction in Nigeria. This is particularly crucial for low-cost housing programs aimed at addressing the acute housing deficit (Ede and Okundaye, 2014).

2.3 Building with Nigerian Timber

In Nigerian timber markets, there is a wide range of timber species available. However, some of these species have not yet been characterized and graded for easy professional application in structures by Nigerian engineers. This lack of standardization has resulted in less emphasis on the design and construction of timber structures, as the strength properties of these species are not well-known. Additionally, there is a general belief that timber structures are not durable and are prone to fire destruction. However, when properly designed and constructed, timber structures can be aesthetically pleasing and durable.

To address this issue, twenty timber species have been characterized and graded according to the Nigerian grading rule, providing a basis for their professional application. In terms of lengths, internationally accepted spans for timber start at 1.80m and increase in increments of 0.50m. This results in a series of lengths such as 1.80m, 2.10m, 2.40m, 2.70m, 3.00m, 3.30m, 3.60m, 3.90m, 4.20m, 4.50m, 4.80m, 5.10m, 5.40m, 5.70m, 6.00m, 6.30m, and so on. Among these lengths, rectangular-shaped types are commonly found in commercial quantities, such as planks, in Nigeria. (Aguwa James 2016)

Moving on to the Nigerian code of practice for timber design, NCP2 (1973), it is primarily based on BS 5268 (2002). However, replacing BS 5268 (2002) with Eurocode 5 (ECS) has posed a major challenge in timber design using NCP2 (1973). Eurocode 5, based on a limit state format, offers more comprehensive guidelines for timber design. Unlike materials like steel and concrete, the properties of timber materials are not designed or produced using predefined recipes. Instead, quality control procedures known as grading are employed to ensure that timber fulfills specific requirements. Grading involves the classification of timber and certain manufactured products, such as plywood, according to their quality.

Estimating the exact quantity of wood and non-wood forest products in Nigeria is not easily accomplished. However, the flexible use of timber in constructing buildings, particularly in roof fabrication, has made it popular. This popularity stems from the fact that Nigeria spends significant resources on importing steel, even for the fabrication of long-span trusses in sophisticated structures. Yet, timber can achieve economy, strength, durability, aesthetic appeal, and time-saving benefits. Therefore, promoting the development of timber constructions can address the competitive challenges posed by modern architecture.

In Nigeria, traditional houses are predominantly found in rural areas, and their construction materials are determined by the local environment. Mud, wood, straw, palm fronds, and raffia matting are the primary building materials used in traditional Nigerian houses. Straw and raffia palm mats are commonly employed for roofing in the southern regions and some parts of the non-Muslim north. The Ijaw people, particularly those living near water areas, construct their houses using strong bamboo sticks and wood directly on top of the water.

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According to CO. Osasona (2015), two major types of traditional timber buildings are prevalent in Nigeria: Wattle-and

According to CO. Osasona (2015), the two primary traditional timber buildings in Nigeria are Wattle-and-daub construction and Riverine architecture. Wattle-and-daub construction, prevalent in Eastern Nigeria, involves reinforcing a lattice of wooden strips and coating it with a sticky material made of wet soil, clay, sand, animal dung, and straw. Riverine architecture, found among the Ijaw people in Nigeria, consists of buildings constructed solely from timber materials, except for the roofing, which may be made of corrugated iron sheets (CO. Osasona, 2015).

The Nigerian construction industry is predominantly using heavy weight materials like concrete blocks and steel for residential buildings, resulting in high costs due to the increasing cost of obtaining concrete materials. This situation is mainly because alternative building materials are not being adequately explored. There is a need to encourage stakeholders to consider other building materials that can compete with concrete in terms of cost, sustainability, maintenance, and client satisfaction. This will not only provide viable alternatives but also create healthy competition in the construction industry (Ede, A.N et al, 2014).

2.4 Design Philosophies

In designing buildings, countries typically adhere to specific design standards or codes of practice, which are based on various design philosophies.

A design philosophy refers to a set of assumptions and procedures used to address the requirements of serviceability, safety, economy, and functionality of a structure. Different design philosophies have been introduced from various parts of the world. Some of the design philosophies used by engineers include:

1. Working stress method (WSM) / Load Factor Method (LFM)
2. Limit State Method (LSM)
3. Probabilistic Design Method (PDM)

Working Stress Method: The working stress method, also known as the allowable or permissible stress method, is used to determine the strength of a timber structure. It involves evaluating the stresses induced under working conditions and comparing them to the permissible or admissible stress limits. The permissible stresses are calculated by multiplying the grade stress for the timber by various modifying factors. Hence, the equation can be expressed as follows:

$$\sigma m.// = \Sigma\_m.\ grade \times K_1 \times K_2 \times K_3 \times K_4 \times K_5 \quad (1)$$

where:

$\sigma m.//$ = calculated bending stress parallel to the grain

$\Sigma m.$ grade = grade bending stress parallel to the grain

K_1 = modification factor for moisture content of the timber

K_2 = modification factor for duration of the load

K_3 = modification factor for shape of the cross-section

K_4 = modification factor for the depth of the section

K_5 = modification factor for load sharing.

Ultimate Load Method: The ultimate load method, also known as the load factor method or ultimate strength method, involves determining the ultimate load stresses by multiplying the working stress by a factor of safety. This is then compared to the ultimate capacity of the timber sections at yield. Plastic methods are employed to determine the timber section capacities. The relationship can be expressed as:

Working Load X Factor of Safety ≤ Ultimate Strength of timber at failure.

Limit State Design: In limit state design, the load at structural collapse is divided by a selected margin of safety to determine the ultimate capacity of the structure. The ultimate design load is determined by multiplying the working load by a second selected safety margin. The ultimate design load should be less than or equal to the ultimate capacity of the structure. Hence, the condition can be expressed as:

Ultimate design load ≤ ultimate capacity.

Additionally, the working (characteristic) load is multiplied by a partial factor of safety, while the failure/collapse load is multiplied by a partial safety factor.

Probability Design: When using probability design, the designer no longer thinks of each variable as a single value. Instead, each variable is viewed as a probability distribution, and the design is adjusted to account for random variability and improve quality. Potential issues and variations can be predicted and addressed during the early design stages, minimizing potential costs.

2.5 Design of Timber Structures To Eurocodes

The design of timber structures in accordance with Eurocodes is abbreviated as EN 1995 or informally known as BCS. It provides guidelines for designing buildings and civil engineering works using the limit state design philosophy. EN 1995 was approved by the European Committee for Standardization (CEN) on April 16, 2004. It applies to civil engineering works constructed with solid timber or wood-based structural products (e.g., LVL) and wood-based panels joined together with adhesives or mechanical fasteners. The standard is divided into several parts, with EN 1995-1-1 specifically covering general design rules for timber structures and specific design rules for buildings. The contents of EN 1995-1-1 include general principles, basis of design, materials, durability, - Basis of Structural Analysis

a) Ultimate Limit States

b) Serviceability Limit States

c) Connection with Metal Fasteners

d) Components and Assemblies

e) Structural Detailing and ControlPart 1-2: General-Structural Fire Design

   a. EN 1995-1-2 addresses the design of timber structures for fire exposure.

   b. It should be used in conjunction with EN 1995-1-1 and EN 1991-1-1-2: 2002.

   c. EN 1995-1-2 focuses on passive methods of fire protection and identifies differences from normal temperature design.

   d. Active methods of fire protection is not covered.

Part 2: Bridges

EN 1995-2 provides general design rules for structural parts of bridges.

   a. It covers timber and wood-based materials used in bridge structures.

   b. The design rules apply to structural members that are crucial for the bridge's reliability.

2.6 LIMITATIONS OF TIMBER TACKLED BY EUROCODE 5

Timber is a versatile and sustainable material that has been used in construction for centuries. However, it also has certain limitations that need to be considered in structural design. Eurocode, a set of European standards for structural design, has addressed these limitations to ensure safe and efficient use of timber in construction.

Here are some common limitations of timber in construction and how Eurocode has tackled them:

1. Strength and Load-Bearing Capacity: Timber has lower strength compared to materials like steel and concrete. It is susceptible to bending, compression, and tension failures. Eurocode addresses this limitation by providing design rules and equations that consider the strength properties of timber, such as characteristic values for different timber grades, load duration factors, and partial safety factors. These factors ensure that the designed timber structures can withstand the expected loads and maintain the required safety margins. (section 2 .3, pg. 15)

2. Moisture and Durability: Timber is susceptible to decay, rot, and insect attacks when exposed to moisture for extended periods. Eurocode emphasizes the need for proper detailing, protection, and treatment of timber elements to enhance durability. It provides guidelines on moisture content limits, timber treatment methods, and appropriate design measures to prevent or control moisture ingress and ensure the long-term performance of timber structures. (cl 2.3.2, pg22)

3. Fire Resistance: Timber is a combustible material and can lose its load-bearing capacity when exposed to fire. Eurocode addresses this limitation by providing fire design rules and performance requirements for timber structures. It defines fire

resistance classes for different timber elements and specifies fire protection measures, such as the use of fire-resistant coatings or the inclusion of additional layers of fire-resistant materials, to ensure the desired fire performance. Eurocode 5 (EN 1995-1-2, cl 3.2)

4. Dimensional Stability: Timber has the tendency to shrink, swell, and warp due to changes in moisture content and environmental conditions. Eurocode provides guidelines for accounting for these dimensional changes in structural design. It specifies design values for timber deformation properties, recommends appropriate moisture content assumptions, and advises on detailing techniques to minimize the adverse effects of dimensional changes on the structural integrity of timber elements. Eurocode 5 (EN 1995-1-2, cl 4.2)

5. Connections and Joints: Timber structures heavily rely on connections and joints for transferring loads and maintaining overall stability. Eurocode offers design provisions for timber connections and joints, including design equations,

recommended fastener types, and detailing guidelines. It ensures that the connections are adequately designed to resist the applied loads and maintain the structural integrity of the timber system. Eurocode 5 (EN 1995-1-2, cl 6.2)

Overall, Eurocode has played a crucial role in addressing the limitations of timber in construction by providing comprehensive design rules and guidelines. It enables engineers and architects to effectively utilize timber as a reliable and safe construction material while considering its unique properties and challenges.

2.7 COMPUTER AIDED DESIGN OF TIMBER USING TEKLA STRUCTURAL DESIGNER

The following can be done using tekla structural designer in the design of timber

1. 3D Modeling of Timber Structures:

Tekla Structural Designer allows engineers to create detailed 3D models of timber structures. Various timber elements such as beams, columns, and custom components can be accurately represented in the model.

2. **Material Properties and Design Loads:**

Engineers can input specific material properties for the timber, such as density, strength, and stiffness.

Design loads, including dead loads, live loads, and other applicable loads, can be assigned to the timber elements.

3. CAD Design Tools for Timber Elements:

Tekla Structural Designer provides specialized CAD design tools for timber beams, columns, and other timber components.

Engineers can define cross-sections, lengths, and orientations of the timber elements using intuitive design interfaces.

4. Design Checks and Compliance:

The software performs design checks on the timber elements based on the specified material properties and design loads.

Structural capacity, deflection limits, and stability requirements are evaluated to ensure compliance with design standards.

5. Timber Connection Design:

Tekla Structural Designer supports the detailed design of timber connections.

Engineers can define connection types, specify fasteners or connectors, and ensure proper load transfer between timber members.

6. Generation of Drawings and Reports:

The software enables the generation of accurate drawings and reports for timber structures.

Detailed drawings, including plans, elevations, and sections, can be produced, along with comprehensive design reports.

7. Compliance with Local Design Codes:

It is essential to complement the CAD design process in Tekla Structural Designer with local design codes and standards. Engineers should ensure that the timber designs adhere to specific requirements and best practices in timber construction.

By utilizing Tekla Structural Designer for CAD design of timber structures, engineers can benefit from enhanced efficiency, accuracy, and documentation capabilities. The software streamlines the design process, facilitates collaboration, and helps deliver structurally sound timber designs.

CHAPTER THREE

MATERIALS AND METHODS(ANALYSIS)

SITE DESCRIPTION

Awka is a city located in southeastern Nigeria. It is the capital of Anambra State and is situated approximately 400 kilometers (250 miles) southeast of Abuja, the country's capital, and about 175 kilometers (110 miles) northwest of Port Harcourt, a major port city in Nigeria. The geographic coordinates of Awka are approximately 6.2100° N latitude and 7.0700° E longitude.

The terrain in and around Awka is generally characterized by low-lying plains and undulating hills. The city is located within the tropical rainforest zone, which influences its climate and weather patterns.

The prevailing wind speed in Awka can vary throughout the year. However, the region experiences a predominant easterly wind flow, particularly during the dry season. These winds are often influenced by the Inter-Tropical Convergence Zone (ITCZ) and the Harmattan, a dry and dusty trade wind that blows from the Sahara Desert during the winter months.

Awka has a tropical climate with distinct wet and dry seasons. The city experiences a wet season that typically spans from April to October, characterized by higher rainfall and increased humidity. The dry season typically occurs between November and March and is characterized by lower rainfall and drier conditions. The rainy season is characterized by frequent rainfall, while the dry season is relatively rain-free.

The average annual rainfall in Awka is approximately 1,800 to 2,000 millimeters (70 to 80 inches), with the highest precipitation occurring during the peak of the rainy season. The months of July and August generally receive the highest rainfall, while December and January tend to be drier.

In terms of temperature, Awka experiences high average temperatures throughout the year. The average daily temperature ranges between 26°C (79°F) and 31°C (88°F) in the hottest months, while the cooler months see average temperatures ranging from 22°C (72°F) to 28°C (82°F).

The average daily humidity in Awka varies between 65% and 85%, depending on the season. During the rainy season, humidity levels tend to be higher due to the increased moisture in the air, while the dry season experiences lower humidity levels.

Regarding average daily pressure, Awka generally experiences standard atmospheric pressure, which is around 1013 hPa (hectopascals) or 29.92 inches of mercury (inHg).

It's important to note that these climate and weather conditions can vary from year to year, and specific weather patterns may be subject to change. It's always advisable to refer to up-to-date weather forecasts and local meteorological sources for the most accurate and current information.

3.1.2 Site Preparation: Prior to commencing construction, a comprehensive soil analysis is conducted to assess the suitability of the site for building erection. This includes a thorough examination of the soil's bearing strata, bearing capacity, and settlement rate.

3.1.3 Setting Out:

To establish the precise location of the building's foundation, temporary pegs are driven into the ground at the four corners where the footings will be constructed. The building's layout is established using batter boards, consisting of two vertical stakes (measuring 50mm x 100mm x 100mm) driven into the ground at each corner. These stakes are positioned at least 1m beyond the corner foundation lines to prevent disturbance during excavation. A horizontal board (measuring 25mm x

150mm) is then securely fastened to the vertical stakes, ensuring the tops of all batter boards are aligned at the same level. The highest corner serves as the datum batter board, with all other batter boards leveled accordingly.

With the batter boards in place, taut strings are stretched between them to mark the precise positions. To ensure the string position on the batter boards, nails are driven into the boards as reference points for future string replacements if necessary. Intermediate points of the footings are marked using a steel tape and plumb bob. The measurements are initiated from the corners and progressed along the strings. Any inaccuracies in the initial stake placement can be rectified at this stage. Sequentially, the positions of all footings are determined based on the marked points.

3.2 Design Building:

The design encompasses three-story medium-rise timber structure featuring 30rooms, along with lobbies, offices, a janitor's room, a lounge.

The dimension of the building is 21.2 x 20.2 meters, it has total surface area of 300.177 square meters. The total building height is 9.45 meter excluding roof height which is divided over three storey height of 3.15 meters with a clear height of 3meters.

The ground condition is okay; the soil has a safe bearing capacity. The foundation will be designed based on the bearing capacity of the soil

The building will be constructed with large size prefabricated assemblies. The roof of trussed rafter roof and the structure rests on a reinforced concrete foundation. The foundation employed is pad footing.

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3.1 **DESIGN OF FOUNDATION (EN I992-1-1: EUROCODE 2)**

PAD FOOTING

Foundations are designed to withstand the maximum axial load transferred by the column to the soil bearing layer.

Loads from the super structure are calculated in the ultimate limit state and converted to the serviceability state. The shape of the foundation pad (base) is square

Shear failure could arise:

(a) at the face of the column

(b) at a distance d from the face of the column

(c) punching failure of the slab

FACE SHEAR (CL. 6.4.5, EC 2)

Design shear stress at the column perimeter,ν_{Ed}, is ν_{ED}

Effective depth d,

$$d = h - \text{cover} - \text{diameter of bar} \qquad (2)$$

$$\nu_{ED} = \beta \frac{V_{ED.}}{U_0 d} \qquad (3)$$

$$V_{Rd,\,max} = 0.5\nu f_{cd} \qquad (4)$$

$$\nu = 0.6[1 - (f_{ck}/250)] \qquad (5)$$

$$f_{cd} = \alpha_{cc} \frac{f_{ck}}{\gamma_c} \qquad (6)$$

$f_{cd} > \nu_{Ed}$ OK

TRANSVERSE SHEAR (CL. 6.4.4, EC 2)

From above, design shear force on footing, V_{Ed}

Earth pressure , $P_E = \frac{V_{ED}}{base\ area}$ (7)

Ultimate load at 1d area is,

Area = $\Delta VEd = P_E\ (l_1 \times [l_1 - l])$ (8)

Applied shear force is

$V_{Ed,red} = VEd - \Delta VEd$ (9)

Design transverse shear stress, V_{Ed}, is

$V_{Ed} = \frac{V_{Ed,red}}{bd}$ (10)

where b = width of footing = 0.9m

fck = 30 N mm^{-2}

$C_{Rd,c} = 0.18/\gamma_c = 0.18/1.5 = 0.12$ N mm^{-2}

$K = 1 + \sqrt{\frac{200}{d}} =$ (11)

$Area\ of\ steel = 1000 \times \frac{\pi d^2/4}{spacing}$

$\rho_1 = \sqrt{\rho 1y\ \rho 1x} = \sqrt{\frac{Asl,y}{bd} \times \frac{Asl,x}{bd}}$ (12)

$\sigma_{cp} = 0$

$\nu_{min} = 0.035k^{3/2}f_{ck}^{1/2}$

Design shear resistance of concrete, $V_{Rd,c}$, is given by

$V_{Rd,c} = [C_{Rd,ck}(100\rho_1 f_{ck})^{1/3} + k_1\sigma_{cp}] \times (2d/a) \geq [v_{min} + k_1\sigma_{cp}] \times (2d/a)$ (13)

$V_{Ed} < V_{Rd,c}$

PUNCHING SHEAR

punching shear is Checked at 2d from face of column.

Basic control perimeter, u1, is

$$u1 = \text{column perimeter} + 2\pi(2d) \qquad (14)$$

Area within critical perimeter,

$$A = 4 \times h(2 \times d) + h^2 + \pi\,(2 \times d)^2 \qquad (15)$$

Ultimate load on shaded area, $\Delta V_{Ed} = \rho_E \times A$ (16)

Applied shear force, $V_{Ed,\,red} = V_{Ed} - \Delta V_{Ed}$

Punching shear stress,

$$V_{Ed} = \frac{V_{Ed}}{U_1 d}\,2 < V_{Rd,} \qquad (17)$$

Design of flexural members

Flexural members are often called horizontal members which include beams of different types

Flexural members in a timber structure includes

1. Floor joist
2. Beam

Design of flexural members The design of flexural members principally involves consideration of the following actions which are discussed next:

1. Bending
2. Deflection
3. Vibration
4. Lateral buckling

5. Shear

6. Bearing

DESIGN ANALYSIS OF BEAMS AT ULTIMATE LIMIT STATE

.

BENDING (CL. 6.1.6, EC 5)

Bending moment

$M_{d,y} = \frac{f_d l}{8}$ (18)

Design bending strength about Y-Y axis

$F_{m,y,d} = K_h.K_{sys.}K_{mond.} \frac{f_{m,k}}{Y_m}$ (19)

If members are not to fail in bending, the following conditions should be satisfied:

$\frac{\delta_{m,y,d}}{F_{m,y,d}} + K_m \frac{\delta_{m,z,d}}{F_{m,z,d}} \leq 1$ (20)

where

$\sigma_{m,y,d}$ and $\sigma_{m,z,d}$ = design bending stresses about axes y–y and z–z

$f_{m, y, d}$ and $f_{m, z,}$ = design bending strengths

k_m = is a factor that allows for the redistribution of secondary bending stresses and assumes the following values:

– for rectangular or square sections; km = 0.7

– for other cross-sections; km = 1.0

It should be noted that in EC 5, the x–x axis is the axis along the member and that axes y–y and z–z are the major and minor axes respectively. These definitions are consistent with the other structural Eurocodes.

For beams with rectangular cross-sections

$$\boldsymbol{\sigma}_{\mathbf{m,y,d}} = \frac{M_Y}{W_Y} = \frac{M_Y}{bh^2/6} \quad (21)$$

$$\boldsymbol{\sigma}_{\mathbf{m,z,d}} = \frac{M_z}{W_{YZ}} = \frac{M_Y}{hb^2/6} \quad (22)$$

Where

$M_{y,\,d}$ and $M_{z,\,d.}$ = design bending moments about y–y (major axis) and z–z (minor axis)

W_y and W_z = elastic modulus about y–y and z–z

B = is the breadth of beam

H = is the depth of beam

The following are the necessary checks to be satisfied:

The sectional properties is obtained from a value equal or higher than the sectional modulus required from the expression below

$$W_{y,req} \geq \frac{M_{d,y}}{\delta_{m,y,d}} \quad (23)$$

DESIGN ANALYSIS OF BEAMS AT SERVICEABILITY LIMIT STATE

DEFLECTION (CL. 7.2, EC 5) To prevent the possibility of damage to surfacing materials, ceilings, partitions and finishes, and to the functional needs as well as aesthetic requirements, EC 5 recommends various limiting values of deflection for beams (see Table 7.2, EC 5).

The following deflection values are to be calculated:

A) instanteneous deflection due to permanent action, $U_{inst}G$ (from table 2.3, EC 5) Y_G for serviceability limit state = 1.0

Factored permanent load, $G = Y_G\, G_k$

$U_{inst,G}$ = bending deflection + shear deflection

$$\frac{5}{384} \times \frac{f_d l^3}{EI} + \frac{12}{5} \times \frac{F_d L}{EA} \qquad (24)$$

B) instantaneous deflection due to variable action, $U_{inst}Q$

Y_G for serviceability state = 1.0

Factored variable load, Q = $Y_Q Q_k$

Instantaneous deflection due to variable load, $U_{inst}Q$ is given by

$U_{inst}Q$ = bending deflection + shear deflection

$$\frac{5}{384} \times \frac{f_d l^3}{EI} + \frac{12}{5} \times \frac{F_d L}{EA} \qquad (25)$$

(C.) Final deflection due to permanent actions (table 3.2, EC5)

For solid timber timbers subject to service class load

Medium – term loading, K_{def} = 0.6

Final deflection due to permanent actions

$U_{fin,}\ G^1$ is given by

$$U_{fin,G} = U_{inG}\ (1+ K_{def}) \qquad (26)$$

(Cl, 2.4.1,Ec 2/cl 6) $\Psi_2 = 0.3$

Final deflection due to variable action, $U_{fin,\ Q,}$ is given by

$$U_{fin,Q,} = U_{insQ1I}\ (1 + \Psi_2\ K_{def}) \qquad (27)$$

D) check for final deflection

Total deflection, $U_{fin} = U_{finG} + U_{finQ}$

Permissible final deflection assuming the floor support bristle finishes). W_{fin}; is

$W_{fin} = \frac{1}{250}$ X span

For deflection to be satisfied the value of W_{fin} should be greater than the value of U_{fin}

$W_{fin} > U_{fin}$

LATERAL BUCKLING (CL 6.3.3, EC 5)

Critical stress is given bt the expression

$$\sigma_{m,crit} = \frac{0.78b^2}{hl_{ef}} E_{0,05} \tag{29}$$

The relative slenderness ratio is given by

$$\lambda_{rel,m} = \sqrt{\frac{f_{m,k}}{\sigma_{m,crit}}} \tag{30}$$

since $\lambda_{rel,m} < 0.75, k_{crit} = 1$

$$k_{crit}\, f_{m,yd} = k_{crit}(K_h.K_{sys.}K_{mod.} \frac{f_{m,k}}{Y_m}) \tag{31}$$

$$\sigma_{m,d} = \frac{M_{y,d}}{bh^2/_6} = \frac{1641}{100 \times 225^2/_6} \tag{32}$$

For a flexural member in timber structures, lateral buckling is satisfied when

$\sigma_{m,d} \leq k_{crit}\, f_{m,d}$

SHEAR (CL. 6.1.7 & 6.5, EC 5)

Design shear strength is

$$F_{v,d} = K_{sys}K_{mod} \frac{f_{y,k}}{\gamma_m} \tag{33}$$

Maximum shear force is

$V_{Ed} = va =$

Design shear stress at Neural axis is

$$\tau_d = \frac{1.5\, V_d}{A} \tag{34}$$

For shear to be satisfied for a flexural member of timber , the design shear strength must be greater than the design shear stress

BEARING (CL. 6.1.5, EC 5)

Design compressive stress

A) Design bearing force is

$F_{9o,d} = va = \max shear\ force\ at\ the\ support$

$$\sigma_{c,90,d} = \frac{F_{90,d}}{bl} \quad (35)$$

.

B) Design compressive strength

Design compressive strength parallel to grain $f_{c,90,\ d}$ is given by

$$F_{c,90,d} = K_{sys} K_{mod} \frac{f_{c,90,k}}{\gamma_m} \quad (36)$$

Bearing capacity

l = size of wall

h = height of beam

b = breadth of beam

$$K_{c,90} = \left(2.38 - \frac{l}{250}\right)\left(1 + \frac{h}{12l}\right) \quad (37)$$

$K_{c,} F_{G90,d} > \sigma_{c90,d}$

DESIGN OF COMPRESSION MEMBERS

Compression members in a timber structure includes

1. Column /posts
2. Studs in framing
3. Struts in truss members

Geometric properties

The geometric properties of the compression members are determined using the below expressions

Effective length= 1.0 *X* h

Area =bXh

Where ,h=

$$I = \frac{bh^3}{12} \quad (38)$$

$$Z = \frac{bh^2}{6} \quad (39)$$

$$L = l_{yy} = l_{zz} = \sqrt{\frac{I}{A}} \quad (40)$$

.

Effective slenderness ratio

$$\lambda = \frac{L_e}{i} \quad (41)$$

$$\lambda'_{rely} = \lambda'_{relz} = \frac{\lambda_y}{\pi}\sqrt{\frac{f_{c,0,k}}{E_{0.05}}} \quad (42)$$

$$\lambda'_{rely} > 0.3$$

BUCKLING (CL. 6.3.2, EC 5)

Design compressive stress

$$\sigma_{,c,0,d} = \frac{N}{A} \quad (43)$$

Applied bending moment M is

$$M = (N \times D) \quad (44)$$

Design Bending strength $f_{m,d}$ **is**

$$f_{m,d} = K_h K_{mod.} \frac{f_{m,k}}{Y_m} \quad (45)$$

Design compression strength $f_{c,0,d}$ **is**

$$f_{c,0,d} = K_{mod.} \frac{k_{C,0,k}}{Y_m} \quad (46)$$

Design bending stress about the y–y axis, $\sigma_{m,y,d}$

$$\sigma_{m,y,d} = \frac{M_Y}{Z_Y} \quad (47)$$

Design bending stress about the z–z axis, $\sigma_{m,z,d} = 0$**.**

$$k_y = k_z = 0.5(1 + \beta c(\lambda_{rel,y} - 0.3) + \lambda^2_{rel,y}) \quad (48)$$

$$k_{cy} = k_{cz} = \frac{1}{k_y + \sqrt{{k_y}^2 - \lambda^2_{rely}}} \quad (49)$$

$$\frac{\sigma_{c,0,d}}{k_{C,y,}f_{C,0,d}} + \frac{\sigma_{m,y,d}}{f_{m,y,d}} + k_m \frac{\sigma_{m,z,d}}{f_{m,z,d}} \leq 1 \quad (50)$$

LATERAL TORSIONAL STABILITY

$$\boldsymbol{\sigma}_{crit} = \frac{\mathbf{0.78}b^2}{hl_{ef}} E_{min} \quad (51)$$

$$\lambda_{rely} = \sqrt{\frac{f_{\mathrm{m,k}}}{\boldsymbol{\sigma}_{crit}}} \quad (52)$$

Check lateral torsional stability

$$\left(\frac{\sigma_{m,d}}{\boldsymbol{k}_{crit} f_{m,y,d}}\right)^2 + \frac{\sigma_{c,0,d}}{\boldsymbol{k}_{c,z} f_{c,0,d}} \leq 1 \quad (53)$$

COMPUTER AIDED DESIGN OF TIMBER USING TEKLA STRUCTURAL DESIGNER

The following can be done using tekla structural designer in the design of timber

1. 3D Modeling of Timber Structures:

The first approach to cad design of timber is the insertion of grid lines in both axis directions to help in the placement of structural members. The windows interface for gridding in tekla structural designer is given in the figure below

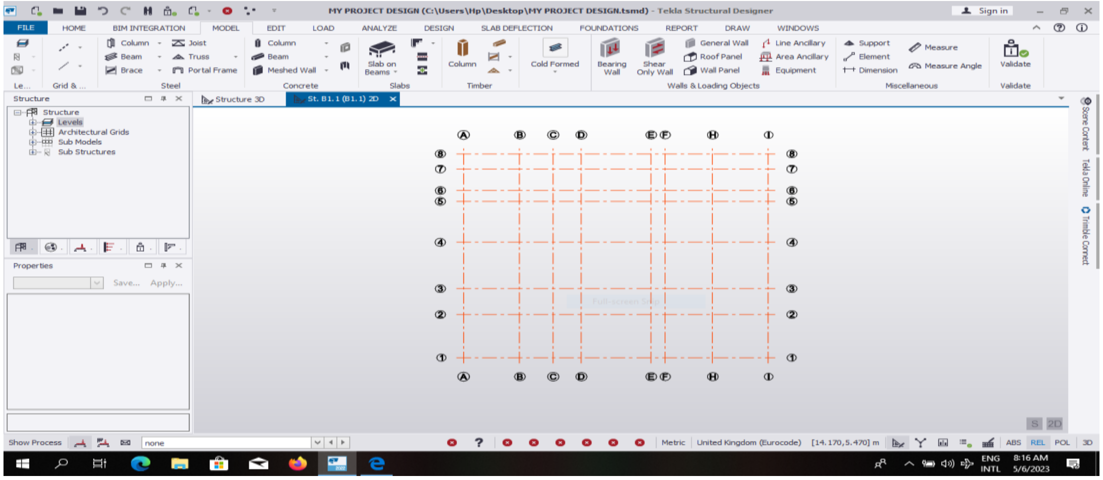


Table 3.1

Tekla Structural Designer allows engineers to create detailed 3D models of timber structures. Various timber elements such as beams, columns, and custom components can be accurately represented in the model.The full representation of the main structural elements is shown below

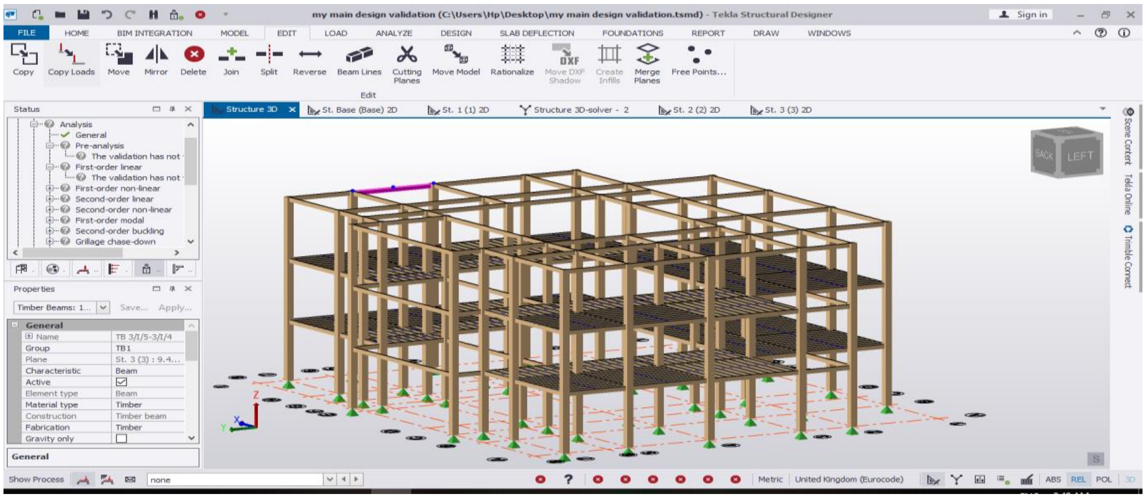


Figure 3.2

2. Material Properties and Design Loads:

Engineers can input specific material properties for the timber, such as density, strength, and stiffness. Design loads, including dead loads, live loads, and other applicable loads, can be assigned to the timber elements. The loading on each member is represented in the 3d view.

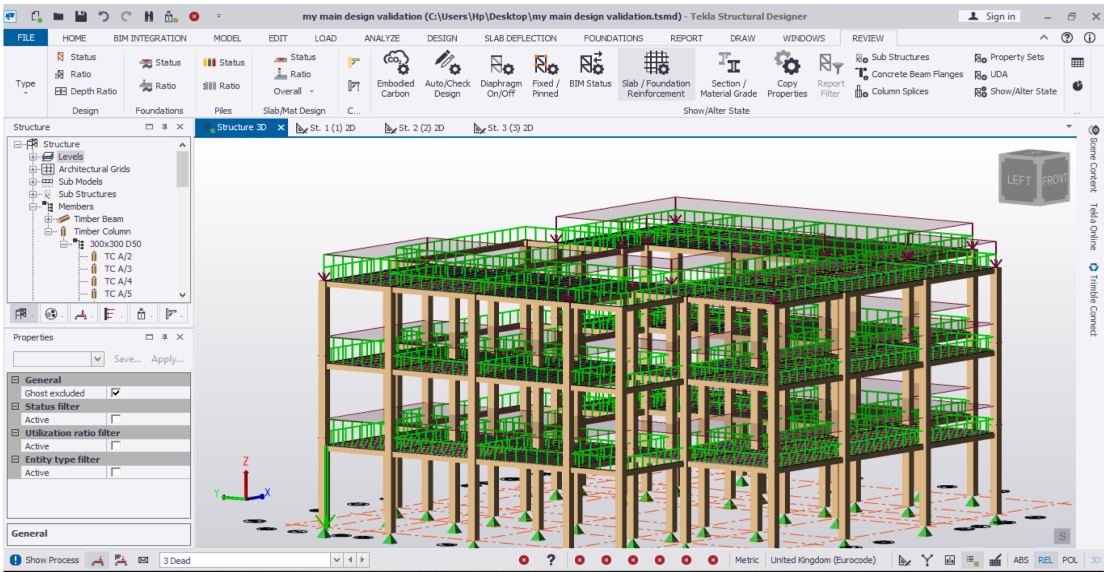


figure 3.3

3. CAD Design Tools for Timber Elements:

Tekla Structural Designer provides specialized CAD design tools for timber beams, columns, and other timber components.

Engineers can define cross-sections, lengths, and orientations of the timber elements using intuitive design interfaces.

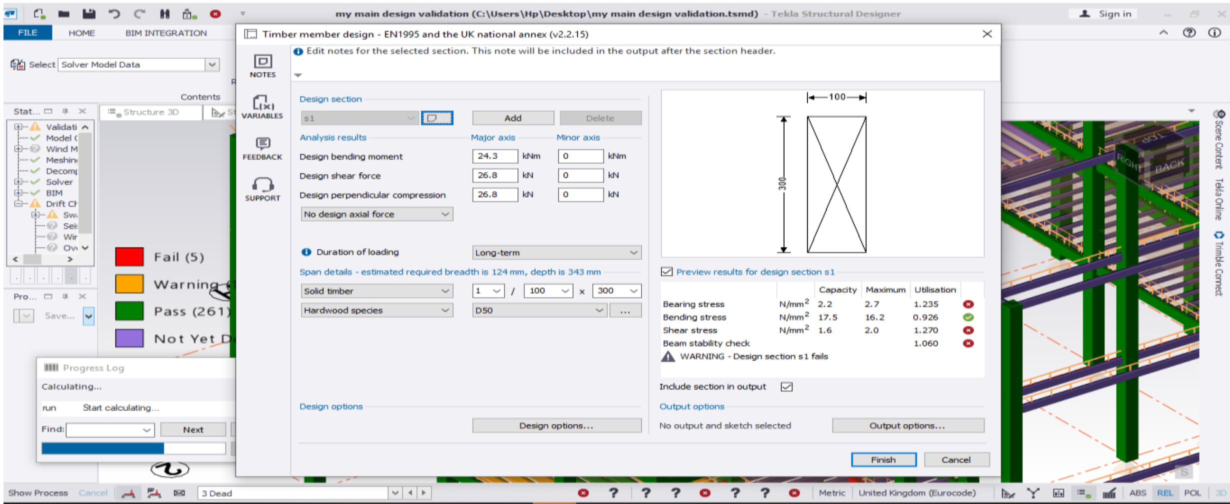


Figure3.4

4. Design Checks and Compliance:

The software performs design checks on the timber elements based on the specified material properties and design loads.

Structural capacity, deflection limits, and stability requirements are evaluated to ensure compliance with design standards.

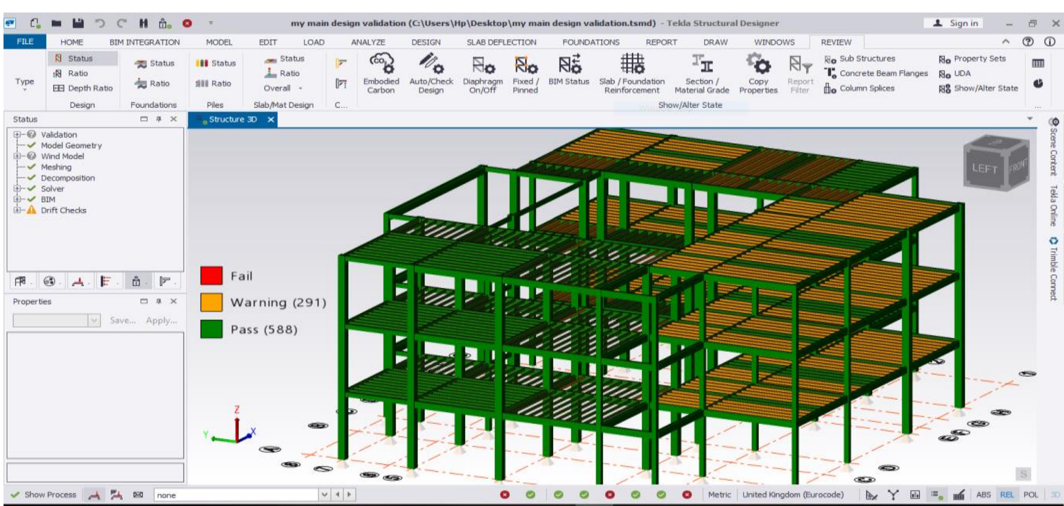


Figure 3.5

5. Generation of Drawings and Reports:

The software enables the generation of accurate drawings and reports for timber structures. Detailed drawings, including plans, elevations, and sections, can be produced, along with comprehensive design reports.

CHAPTER FOUR

DESIGN

.

4.0 INTRODUCTION

Design of builing structures is defined as the planning,determination of sizes ,and arrangement of structural members so that external forces or loadfs on tnhe Structure are transmitted to the foundation in the most economical manner consistent with the purpose of the structure (V.O.oyenuga,2011)

The nigerian timber adoted has the following properties(Aguwa james,2016)

Table 4.1

Description	Property
Name of timber	Danta
Stenght class	N2
Density	770kg/m^3
Wood type	Hardwood
Emean(N/mm^2)	12675
Emin(N/mm^2)	10302
Availability	South east asnd south south

FLOOR JOISTS DESIGN (EUROCODE -5:2004 IS EMPLOYED)

The design of timber floor joist for offices using strength of class N2, given that

.

i. The joists are spaced at 400mm centers
ii. The flooring is tongue and groove boarding with self-weight of $0.4KN/M^2$ and thickens of 25mm
iii. The ceiling is of asbestos ceiling board of 12mm and self-wt. of $0.2KN/M^2$
iv. The floor finish is of 37mm concrete topping panels.

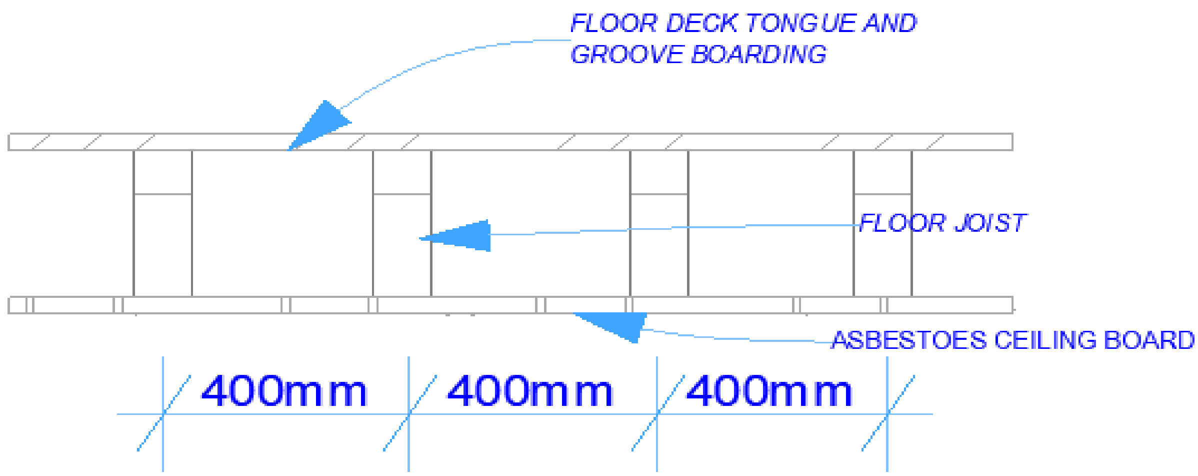


Figure 4.1

PANELS:

Panel 1: This includes all panels with effective span of 2,150mm

Panel 2: This includes all panels with effective span of 3,125mm

Panel 3: This includes all panels with effective span of 4,000mm

Panel 4: This includes all panels with effective span of 5,000mm

DESIGN LOADING

A). Permanent action (G_K);

Tongue and groove boarding	=	$0.1KN/M^2$
Asbestos Ceiling	=	$0.2KN/M^2$
Joist (Assumed)	=	$\underline{0.25KN/M^2}$
Total permanent action (G_k)	=	$\underline{0.55KN/M^2}$

B). Variable action, Q_K :

Imposed floor loads for offices (table 3, EN 1991-1-1) =$1.50KN/M^2$

DESIGN ACTION

Total design load is

$= \gamma_G G_k + \gamma_Q Q_k =$

$= 1.35 \text{ x } 0.55 + 1.5 \text{ x}1.5 = 2.99KN/M^2$

Design load on joist, F_d

$= F_d =$ joist spacing x effective span x load

$F_d = 0.4 \text{ xl } 2.15 \text{ x } 2.99 = \underline{\sim}2.57KN$

CACULATIONS

<u>Design parameters</u>

The grade of the seasoned Danta timber used is of N2 class

Characteristics strengths and modulus of elasticity for timber of strength class N2

Values in Nmm^{-2} are given below

Table 4.2

Bending strength ($F_{M,K}$)	Compression perpendicular to grain ($F_{C,90,K}$)	Shear parallel to grain (Fv,k)	Modules of elasticity (60, mean)
33.17	5.05	3.75	12675

K_3 = duration of loading = 1.0

K_8 = loading sharing system = 1.1

.

K_7 = depth factor = $\left(\frac{300}{h}\right)$ 0.11

- The joist will be designed for service class 1 (clause 2.3.1.3, EC5)
- The joist will carry both variable and permanent load, critical load duration class – medium term (table 1, EC5)
- K_{mod} (table 3.1, EC5) – 0.80
- Y_m (Table 2.3 EC 5) – 1.3
- The joist is from part of a load sharing system hence (K_{sys} = 1.1)
- - assure K_h = 1

Bending (Cl 6.1.6, EC5)

Bending moment

$M_{d,y} = \frac{f_d l}{8} = \frac{2.57x2.15}{8} = 0.69K_{NM}$

Design bending strength about Y-Y axis

$F_{m,y,d} = K_h.K_{sys.}K_{mond.} \frac{f_{m,k}}{Y_m} = \frac{1.0\times1.1\times0.8\times33.19}{1}$

$$\frac{1.0 \times 1.1 \times 0.8 \times 33.19}{1.3}$$

$F_{m,\ yd.}$ = 22.47N/mm^2

Bending (Cl 6.1.6, EC 5)

$\frac{\delta_{m,y,d}}{F_{m,y,d}} + K_m \frac{\delta_{m,z,d}}{F_{m,z,d}} \leq 1$

K_m = 0.7 – for rectangular / square timber section

$\frac{\delta_{m,y,d}}{22.47} + 0.7 \text{ x } \frac{0}{F_{m,z,d}} \leq 1$

$\delta_{m,y,d} \leq 22.47N/mm^2$

$$W_{y,req} \geq \frac{M_{d,y}}{\delta_{m,y,d}} = \frac{0.691 \times 10^6}{22.47}$$

$= 30.75 \times 10^3 mm^3$

From table for classification of Nigerian timber (Aguwa, 2016)

75mm X 100mm joist would be suitable

$W_y = 125 \text{ X } 10^3 \text{ mm}^3$, $I_y = 6.25 \text{ X } 10^6 \text{ MM}^4$, $A = 7.50 \text{ X } 10^3 \text{ mm}^2$

Deflection (Cl 7.2 , EC5)

A). instatenous deflection due to permanent action, $U_{inst}G$ (from table 2.3, EC 5) Y_G for serviceability limit state = 1.0

Factored permanent load, $G = Y_G G_k = 1.0 \text{ X } 0.\ 40$

$= 0.57N/m^2$

Factored permanent load per joist $f_{d,G}^1$ is

$F_{d,G}$ = total load X joist spacing X span length

= 0.57 X 0.4 X 2.15 = 0.49KN

$U_{inst,G}$ = bending deflection + sheer deflection

$$\frac{5}{384} \times \frac{f_d l^3}{EI} + \frac{12}{5} \times \frac{F_d L}{EA}$$

$$U_{inst,G} = \frac{5}{384}\left(\frac{(0.49 \times 10^3 (2.15 \times 10^3)^3}{12675 \times 6.2 \times 10^6}\right) + \frac{12}{5} \times \frac{0.49 \times 10^3 \times 2.15 \times 10^3}{12675 \times 7.5 \times 10^3}$$

0.80+ 0.027

= 1.07mm

B) instantaneous deflection due to variable action, $U_{inst}Q$

Y_G for serviceability state = 1.0

Factored variable load, $Q = Y_Q Q_k = 1.0 \text{ X } 1.5 = 1.5KN/M_2$

Factored variable action per joist, F_d,Q

F_d, Q = total load X joist spacing X span length

= 1.5 X 0.4 X 2.15 = 1.29KN

.

Instantaneous deflection due to variable load, $U_{inst}Q$ is given by

$U_{inst}Q$ = bending deflection + sheer deflection

$$\frac{5}{384} \times \frac{f_d l^3}{EI} + \frac{12}{5} \times \frac{F_d L}{EA}$$

$$\frac{5}{384}\left(\frac{1.29\times10^3(2.15\times10^3)^3}{12675\times6.2\times10^6}\right) + \frac{12}{5} \times \frac{1.29\times10^3\times2.15\times10^3}{12675\times7.5\times10^3}$$

= 2.12 + 0.07

= 2.82mm

(C.) Final deflection due to permanent actions (table 3.2, EC5)

For solid timber timbers subject to service class land

Medium – term loading, K_{def} = 0.6

Final deflection due to permanent actions

$U_{fin,}\ G^1$ is given by

$U_{fin,G}$ = U_{inG} (1+ K_{def})= 1.07 X (1 + 0.6) = 1.642mm

(Cl, 2.4.1,Ec2/cl6) Ψ_2 = 0.3

Final deflection due to variable action, $U_{fin,\ Q,}$ is given by

$U_{fin,Q,}$ = U_{insQ1I} (1 + Ψ_2 K_{def})

= 2.82 (1 + 0.3 x 0.6) = 3.33mm

D) check for final deflection

Total deflection, U_{fin} = U_{finG} + U_{finQ} = 1.6 42+ 3.33

= 4.97

Permissible final deflection assuming the floor support bristle finishes). W_{fin}; is

$W_{fin} = \frac{1}{250}$ X span

1/250 X 2.15 x 10^3 = 8.6

8.6mm > 4.97 OK

Therefore, 75mm X 100mm Joist are adequate in deflection

LATERAL BUCKLING (CL 6.3.3, EC 5)

The check is unnecessary as the compressive edge cannot move totally because the joist is attached to tongue and groove boarding.

SHEAR

Design shear strength is

$F_{v,d} = K_{sys}K_{mod}\frac{f_{y,k}}{\gamma_m} - 1.1 \times 0.8 \times \frac{3.75}{1.3}$

Maximum shear force is

$V_{Ed} = \frac{F_d}{2} = \frac{2.57 \times 10^3}{2} = 1.285 \times 10^3 N$

Design shear stress at Neural axis is

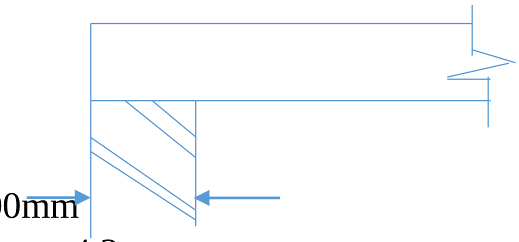
$\gamma_d = \frac{1.5\, V_d}{A} = \frac{1.5 \times 1.285 \times 10^3}{7.5 \times 10^3} = 0.257 N/mm^2 < f_{v,d}$ OK

BEARING

Design compressive stress

C) Design bearing force is

$F_{Go,d} = = \frac{F_d}{2} = \frac{2.57 \times 10^3}{2} = 1.285 \text{ X } 10^3$



100mm

Figure 4.2

Taking the floor joist to span assumptously onto 10mm wide walls as shown above, the bearing stress is given by:

$\delta_{c,90,d} = \frac{F_{90,d}}{bl} = \frac{1.2\ 85 \times 10^3}{75 \times 100} = 0.171\text{N/mm}^2$

D) Design compressive strength

Design compressive strength parallel to grain $f_{c,90,\,d}$ is given by

$F_{c,90,d} = K_{sys}\, K_{mod} \frac{f_{c,90,k}}{\gamma_m} = 1.1 \times 0.8 \times \frac{5.05}{1.3}$

$= 3.42\text{N/mm}^2$

Bearing capacity

The above diagram

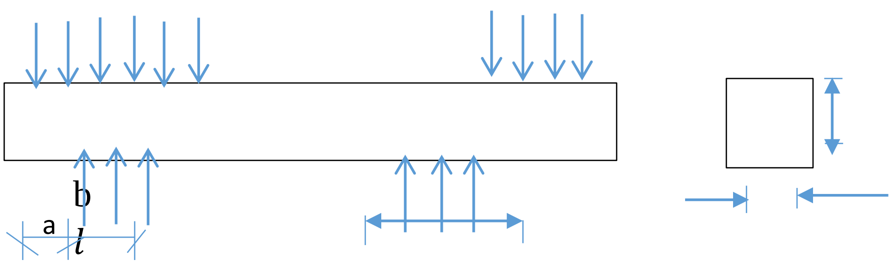


Figure 4.3 l

From the above diagram

a = overhanging length = O for the structure

l = size of wall = 100mm

h = height of joist = 100mm

b = breadth of joist = 75mm

a = o, L = 100m and h = 100mm

since a<h/3

$K_{c,90} = \left(2.38 - \frac{l}{250}\right)\left(1 + \frac{h}{12l}\right) = \left(2.38 - \frac{100}{250}\right)\left(1 + \frac{100}{12 \times 100}\right) = (1.98)(1.08)$

$= 2.13 < 4$ OK

.

$K_c, F_{G90,d} = 2.13 \text{ X } 3.42 = 7.28 N/mm^2\ \delta_{c90,d}$ OK

CHECK ASSUMED SELF WEIGHT OF JOISTS

From the Nigerian timber classification table

Density of class N2 timber is $770 kg/m^2$

$SW = \frac{b\ h\ p\ g}{spacing}$

$SW = \frac{(75 \times 100 \times 10^{-6}) \times 770 \times 9.8 \times \times 10^{-3}}{0.4} = 0.14 KNm^2$

Assumed self-weight > actual self-weight. hence assumed self-weight is ok

Design load on joist, F_d

$= F_d =$ joist spacing x effective span x load

$F_d = 0.4 \text{ x} 3.125 \text{ x } 2.99 = \underline{\sim} 3.74 KN$

CACULATIONS

<u>Design parameters</u>

The grade of the seasoned Danta timber used is of N2 class

Characteristics strengths and modulus of elasticity for timer of strength class N2

Values in Nmm^{-2} are given below

Table 4.3

Bending strength ($F_{M,K}$)	Compression perpendicular to grain ($F_{C,90,K}$)	Shear parallel to grain (Fv,k)	Modules of elasticity (60, mean)
33.17	5.05	3.75	12675

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K_7 = depth factor = $\left(\frac{300}{h}\right)$ 0.11

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- The joist will carry both variable and permanent load, critical load duration class – medium term (table 1, EC5)
- K_{mod} (table 3.1, EC5) – 0.80
- Y_m (Table 2.3 EC 5) – 1.3
- The joist is from part of a load sharing system hence (K_{sys} = 1.1)
- - assure K_h = 1

Bending (Cl 6.1.6, EC5)

Bending moment

$M_{d,y} = \frac{f_d l}{8} = \frac{3.74x3.125}{8} = 1.46K_{NM}$

Design bending strength about Y-Y axis

$F_{m,y,d} = K_h.K_{sys.}K_{mond.}\frac{f_{m,k}}{Y_m} = \frac{1.0\times1.1\times0.8\times33.19}{1.3}$

$F_{m,\ yd.}$ = 22.47N/mm^2

Bending (Cl 6.1.6, EC 5)

$$\frac{\delta_{m,y,d}}{F_{m,y,d}} + K_m\frac{\delta_{m,z,d}}{F_{m,z,d}} \leq 1$$

K_m = 0.7 – for rectangular / square timber section

$\frac{\delta_{m,y,d}}{22.47} + 0.7 \text{ x } \frac{0}{F_{m,z,d}} \leq 1$

$\delta_{m,y,d} \leq 22.47N/mm^2$

$W_{y,req} \geq \frac{M_{d,y}}{\delta_{m,y,d}} = \frac{1.46 \times 10^6}{22.47}$

$= 64.98 \times 10^3 mm^3$

From table for classification of Nigerian timber (Aguwa, 2016)

75mm X 150mm joist would be suitable

$W_y = 281X\ 10^3\ mm^3$, $I_y = 21.09\ X\ 10^6\ MM^4$, $A = 11.3X\ 10^3\ mm^2$

Deflection (Cl 7.2 , EC5)

A). instatenous deflection due to permanent action, $U_{inst}G$ (from table 2.3, EC 5) Y_G for serviceability limit state = 1.0

Factored permanent load, $G = Y_G\ G_k = 1.0\ X\ 0.\ 57$

$= 0.57N/m^2$

Factored permanent load per joist $f_{d,G}{}^1$ is

$F_{d,G}$ = total load X joist spacing X span length

= 0.57 X 0.4 X 3.125 = 0.71KN

$U_{inst,G}$ = bending deflection + sheer deflection

$$\frac{5}{384} \times \frac{f_d l^3}{EI} + \frac{12}{5} \times \frac{F_d L}{EA}$$

$U_{inst,G} = \frac{5}{384}\left(\frac{(0.71 \times 10^3 (3.125 \times 10^3)^3}{12675 \times 21.09 \times 10^6}\right) + \frac{12}{5} \times \frac{0.71 \times 10^3 \times 3.125 \times 10^3}{12675 \times 11.3 \times 10^3}$

= 1.09mm

B) instantaneous deflection due to variable action, $U_{inst}Q$

Y_G for serviceability state = 1.0

Factored variable load, Q = $Y_Q Q_k$ = 1.0 X 1.5 = 1.5KN/M_2

Factored variable action per joist, F_d,Q

F_d, Q = total load X joist spacing X span length

= 1.5 X 0.4 X 3.125 = 1.875KN

Instantaneous deflection due to variable load, U_{inst}Q is given by

U_{inst}Q = bending deflection + sheer deflection

$$\frac{5}{384} \times \frac{f_d l^3}{EI} + \frac{12}{5} \times \frac{F_d L}{EA}$$

$$\frac{5}{384}\left(\frac{1.875\times10^3(3.125\times10^3)^3}{12675\times21.09\times10^6}\right) + \frac{12}{5} \times \frac{1.875\times10^3\times3.125\times10^3}{12675\times11.3\times10^3}$$

=2.89mm

(C.) Final deflection due to permanent actions (table 3.2, EC5)

For solid timber timbers subject to service class land

Medium – term loading, K_{def} = 0.6

Final deflection due to permanent actions

$U_{fin,}$ G^1 is given by

$U_{fin,G}$ = U_{inG} (1+ K_{def})= 1 .09X (1 + 0.6) = 1.744mm

(Cl, 2.4.1,Ec2/cl6) Ψ_2 = 0.3

Final deflection due to variable action, $U_{fin, Q,}$ is given by

$U_{fin,Q,}$ = U_{insQ1I} (1 + Ψ_2 K_{def})

= 2.89 (1 + 0.3 x 0.6) = 3.410mm

D) check for final deflection

Total deflection, U_{fin} = U_{finG} + U_{finQ} = 1.774 + 3.410

= 5.18mm

Permissible final deflection assuming the floor support bristle finishes). W_{fin}; is

$W_{fin} = \frac{1}{250}$ X span

1/250 X 3.125 x 10^3 = 8.6

8.6mm > 5.18 **HENCE DEFLECTION OK**

Therefore, 75mm X 150mm Joist are adequate in deflection

LATERAL BUCKLING (CL 6.3.3, EC 5)

The check is unnecessary as the compressive edge cannot move totally because the joist is attached to tongue and groove boarding.

SHEAR

Design shear strength is

$F_{v,d} = K_{sys}K_{mod}\,\frac{f_{y,k}}{\gamma_m} - 1.1 \times 0.8 \times \frac{3.75}{1.3} = 2.53\text{N/mm}^2$

Maximum shear force is

$V_{Ed} = \frac{F_d}{2} = \frac{3.74 \times 10^3}{2} = 1.879 \times 10^3 N$

Design shear stress at Neural axis is

$\tau_d = \frac{1.5\,V_d}{A} = \frac{1.5 \times 1.879 \times 10^3}{11.3 \times 10^3} = 0.25\text{N/mm}^2 < f_{v,d}$ OK

BEARING

Design compressive stress

E) Design bearing force is

$F_{9o,d} = = \frac{F_d}{2} = \frac{3.74 \times 10^3}{2} = 1.870 \text{ X } 10^3$

.

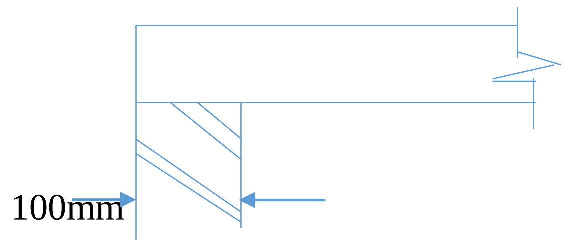


Figure 4.4

Taking the floor joist to span onto 100mm(assumed) wide walls as shown above, the bearing stress is given by:

$\delta_{c,90,d} = \frac{F_{90,d}}{bl} = \frac{1.870\times10^{3}}{75\times150} = 0.166\text{N/mm}^2$

F) Design compressive strength

Design compressive strength parallel to grain $f_{c,90,\,d}$ is given by

$F_{c,90,d} = K_{sys}\,K_{mod}\,\frac{f_{c,90,k}}{\gamma_m} = 1.1 \times 0.8 \times \frac{5.05}{1.3}$

$= 3.42\text{N/mm}^2$

Bearing capacity

The above diagram

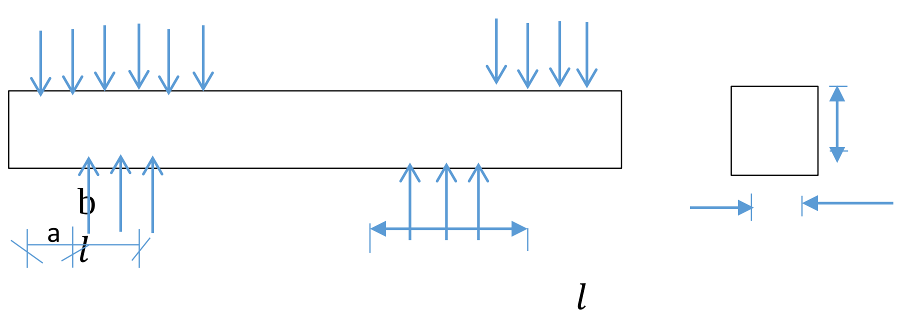


Figure 4.5

From the above diagram

a = overhanging length = O for the structure
l = size of wall = 100mm
h = height of joist = 150mm
b = breadth of joist = 75mm

.

a = o, L = 100m and h = 100mm

since a<h/3

$K_{c,90} = \left(2.38 - \frac{l}{250}\right)\left(1 + \frac{h}{12l}\right) = \left(2.38 - \frac{100}{250}\right)\left(1 + \frac{150}{12 \times 100}\right) = (1.98)(1.125)$

= 2.23 < 4 OK

$K_{c,} F_{G90,d} = 2.23 \text{ X } 3.42 = 7.6\text{N/mm}^2 > \delta_{c90,d}$ OK

CHECK ASSUMED SELF WEIGHT OF JOISTS
From the Nigerian timber classification table
Density of class N2 timber is 770kg/m^2

$$SW = \frac{b\ h\ p\ g}{spacing}$$
$$SW = \frac{(75 \times 150 \times 10^{-6}) \times 770 \times 9.8 \times \times 10^{-3}}{0.4} = 0.21KNm^2$$

Assumed self-weight > actual self-weight. hence assumed self-weight is ok

Design load on joist, F_d

= F_d = joist spacing x effective span x load

F_d = 0.4 x4x 2.99 = $\underset{-}{\sim}$4.78KN

CACULATIONS

Design parameters

The grade of the seasoned Danta timber used is of N2 class

Characteristics strengths and modulus of elasticity for timer of strength class N2

Values in Nmm^{-2} are given below

Table 4.4

Bending strength ($F_{M,K}$)	Compression perpendicular to grain ($F_{C,90,K}$)	Shear parallel to grain (Fv,k)	Modules of elasticity (60, mean)
33.17	5.05	3.75	12675

K_3 = duration of loading = 1.0

K_8 = loading sharing system = 1.1

K_7 = depth factor = $\left(\frac{300}{h}\right)$ 0.11

- The joist will be designed for service class 1 (clause 2.3.1.3, EC5)
- The joist will carry both variable and permanent load, critical load duration class – medium term (table 1, EC5)
- K_{mod} (table 3.1, EC5) – 0.80
- Y_m (Table 2.3 EC 5) – 1.3
- The joist is from part of a load sharing system hence (K_{sys} = 1.1)
- - assure K_h = 1

Bending (Cl 6.1.6, EC5)

Bending moment

$$M_{d,y} = \frac{f_d l}{8} = \frac{4.78x4}{8} = 2.39K_{NM}$$

Design bending strength about Y-Y axis

$$F_{m,y,d} = K_h.K_{sys.}K_{mond.}\frac{f_{m,k}}{Y_m} = \frac{1.0\times1.1\times0.8\times33.19}{1.3}$$

$F_{m,\ yd.}$ = 22.47N/mm^2

Bending (Cl 6.1.6, EC 5)

$\frac{\delta_{m,y,d}}{F_{m,y,d}}$ + $K_m \frac{\delta_{m,z,d}}{F_{m,z,d}} \leq 1$

.

K_m = 0.7 – for rectangular / square timber section

$$\frac{\delta_{m,y,d}}{22.47} + 0.7 \text{ x } \frac{0}{F_{m,z,d}} \leq 1$$

$\delta_{m,y,d} \leq 22.47N/mm^2$

$W_{y,req} \geq \frac{M_{d,y}}{\delta_{m,y,d}} = \frac{2.39 \times 10^6}{22.47}$

$=106.4 \times 10^3 mm^3$

From table for classification of Nigerian timber (Aguwa, 2016)

75mm X 150mm joist would be suitable

W_y = 281X 10^3 mm^3, I_y = 21.09 X 10^6 MM^4, A = 11.3X 10^3 mm^2

Deflection (Cl 7.2 , EC5)

A). instatenous deflection due to permanent action, $U_{inst}G$ (from table 2.3, EC 5) Y_G for serviceability limit state = 1.0

Factored permanent load, G = Y_G G_k = 1.0 X 0. 57

= 0.57N/m^2

Factored permanent load per joist $f_{d,G}^1$ is

$F_{d,G}$ = total load X joist spacing X span length

= 0.57 X 0.4 X 4 = 0.912KN

$U_{inst,G}$ = bending deflection + sheer deflection

$$\frac{5}{384} \times \frac{f_d l^3}{EI} + \frac{12}{5} \times \frac{F_d L}{EA}$$

$U_{inst,G} = \frac{5}{384} \left(\frac{(0.912 \times 10^3 (3.125 \times 10^3)^3}{12675 \times 21.09 \times 10^6}\right) + \frac{12}{5} \times \frac{0.912 \times 10^3 \times 3.125 \times 10^3}{12675 \times 11.3 \times 10^3}$

= 1.40mm

B) instantaneous deflection due to variable action, $U_{inst}Q$

Y_G for serviceability state = 1.0

.

Factored variable load, $Q = Y_Q Q_k$ = 1.0 X 1.5 = 1.5KN/M_2

Factored variable action per joist, F_d,Q

F_d, Q = total load X joist spacing X span length

= 1.5 X 0.4 X 4 = 2.4KN

Instantaneous deflection due to variable load, $U_{inst}Q$ is given by

$U_{inst}Q$ = bending deflection + sheer deflection

$$\frac{5}{384} \times \frac{f_d l^3}{EI} + \frac{12}{5} \times \frac{F_d L}{EA}$$

$$\frac{5}{384}\left(\frac{2.4\times 10^3(3.125\times 10^3)^3}{12675\times 21.09\times 10^6}\right) + \frac{12}{5} \times \frac{2.4\times 10^3\times 3.125\times 10^3}{12675\times 11.3\times 10^3}$$

=3.69mm

(C.) Final deflection due to permanent actions (table 3.2, EC5)

For solid timber timbers subject to service class land

Medium – term loading, K_{def} = 0.6

Final deflection due to permanent actions

$U_{fin,}\ G^1$ is given by

$U_{fin,G} = U_{inG}\ (1+ K_{def})$= 1.4X (1 + 0.6) = 2.24mm

(Cl, 2.4.1,Ec2/cl6) Ψ_2 = 0.3

Final deflection due to variable action, $U_{fin,\ Q,}$ is given by

$U_{fin,Q,} = U_{insQ1I}\ (1 + \Psi_2\ K_{def})$

= 3.69 (1 + 0.3 x 0.6) = 4.35mm

D) check for final deflection

Total deflection, $U_{fin} = U_{finG} + U_{finQ} = 2.24 + 4.35$

= 6.59mm

Permissible final deflection assuming the floor support bristle finishes). W_{fin}; is

$W_{fin} = \frac{1}{250}$ X span

1/250 X 4 x 10^3 = 16mm

16mm > 6.59mm OK

Therefore, 75mm X 150mm Joist are adequate in deflection

LATERAL BUCKLING (CL 6.3.3, EC 5)

The check is unnecessary as the compressive edge cannot move totally because the joist is attached to tongue and groove boarding.

SHEAR

Design shear strength is

$F_{v,d} = K_{sys}K_{mod} \frac{f_{y,k}}{\gamma_m} - 1.1 \times 0.8 \times \frac{3.75}{1.3}$

Maximum shear force is

$V_{Ed} = \frac{F_d}{2} = \frac{4.78 \times 10^3}{2} = 2.390 \times 10^3 N$

Design shear stress at Neural axis is

$\tau_d = \frac{1.5\, V_d}{A} = \frac{1.5 \times 2.390 \times 10^3}{11.3 \times 10^3} = 0.317 N/mm^2 < f_{v,d}$ OK

BEARING

Design compressive stress

G) Design bearing force is

$$F_{90,d} = = \frac{F_d}{2} = \frac{4.78 \times 10^3}{2} = 2.390 \text{ X } 10^3$$

100mm

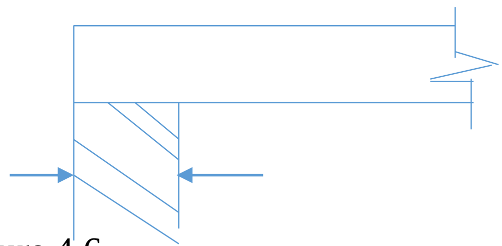


Figure 4.6

Taking the floor joist to span onto 10mm(assumed) wide walls as shown above, the bearing stress is given by:

$$\delta_{c,90,d} = \frac{F_{90,d}}{bl} = \frac{2.390 \times 10^3}{75 \times 150} = 0.21\text{N/mm}^2$$

H) Design compressive strength

Design compressive strength parallel to grain $f_{c,90,\, d}$ is given by

$$F_{c,90,d} = K_{sys}\, K_{mod}\, \frac{f_{c,90,k}}{\gamma_m} = 1.1 \times 0.8 \times \frac{5.05}{1.3}$$

$= 3.42\text{N/mm}^2$

Bearing capacity

The above diagram

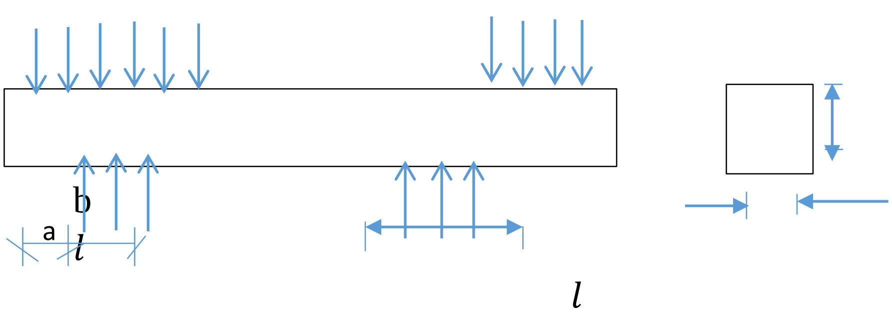


Figure 4.7

From the above diagram

a = overhanging length = O for the structure

l = size of wall = 100mm

h = height of joist = 150mm

b = breadth of joist = 75mm

a = o, L = 100m and h = 100mm

since a<h/3

.

$K_{c,90}$ = $\left(2.38 - \frac{l}{250}\right)\left(1 + \frac{h}{12l}\right) = \left(2.38 - \frac{100}{250}\right)\left(1 + \frac{150}{12 \times 100}\right) =$ $(1.98)(1.125)$

= 2.23 < 4 OK

$K_{c,}$ $F_{G90,d}$ = 2.23X 3.42 = 7.63N/mm^2 $> \delta_{c90,d}$ OK

CHECK ASSUMED SELF WEIGHT OF JOISTS

From the Nigerian timber classification table

Density of class N2 timber is 770kg/m^2

SW = $\frac{b\,h\,p\,g}{spacing}$

SW = $\frac{(75\times150\times10^{-6})\times770\times9.8\times\times10^{-3}}{0.4} = 0.21KNm^2$

Assumed self-weight > actual self-weight. hence assumed self-weight is ok

Design load on joist, F_d

= F_d = joist spacing x effective span x load

F_d = 0.4 x6.175x 2.99 = $\underset{-}{\sim}$7.39KN

CACULATIONS

<u>Design parameters</u>

The grade of the seasoned timber used is of N2 class

Characteristics strengths and modulus of elasticity for timer of strength class N2

Values in Nmm^{-2} are given below

Table 4.2

Bending strength ($F_{M,K}$)	Compression perpendicular to grain ($F_{C,90,K}$)	Shear parallel to grain (Fv,k)	Modules of elasticity (60, mean)
33.17	5.05	3.75	12675

K_3 = duration of loading = 1.0

K_8 = loading sharing system = 1.1

K_7 = depth factor = $\left(\frac{300}{h}\right)$ 0.11

- The joist will be designed for service class 1 (clause 2.3.1.3, EC5)
- The joist will carry both variable and permanent load, critical load duration class – medium term (table 1, EC5)
- K_{mod} (table 3.1, EC5) – 0.80
- Y_m (Table 2.3 EC 5) – 1.3
- The joist is from part of a load sharing system hence (K_{sys} = 1.1)
- - assure K_h = 1

Bending (Cl 6.1.6, EC5)

Bending moment

$M_{d,y} = \frac{f_d l}{8} = \frac{7.39x6.175}{8} = 5.704K_M$

Design bending strength about Y-Y axis

$F_{m,y,d} = K_h.K_{sys.}K_{mond.}\frac{f_{m,k}}{Y_m} = \frac{1.0\times1.1\times0.8\times33.19}{1.3}$

$F_{m,\ yd.}$ = 22.47N/mm^2

Bending (Cl 6.1.6, EC 5)

$\frac{\delta_{m,y,d}}{F_{m,y,d}}$ + $K_m \frac{\delta_{m,z,d}}{F_{m,z,d}} \leq 1$

K_m = 0.7 – for rectangular / square timber section

.

$\frac{\delta_{m,y,d}}{22.47}$ + 0.7 x $\frac{0}{F_{m,z,d}} \leq 1$

$\delta_{m,y,d} \leq 22.47 N/mm^2$

$$W_{y,req} \geq \frac{M_{d,y}}{\delta_{m,y,d}} = \frac{5.704 \ \times 10^6}{22.47}$$

$=253.84 \times 10^3 mm^3$

From table for classification of Nigerian timber (Aguwa, 2016)

75mm X 200mm joist would be suitable

W_y = 500X 10^3 mm^3, I_y = 50.X 10^6 MM^4, A = 15X 10^3 mm^2

Serviceability:

Deflection (Cl 7.2 , EC5)

A). instatenous deflection due to permanent action, $U_{inst}G$ (from table 2.3, EC 5) Y_G for serviceability limit state = 1.0

Factored permanent load, G = Y_G G_k = 1.0 X 0. 57

= 0.57N/m^2

Factored permanent load per joist $f_{d,G}{}^1$ is

$F_{d,G}$ = total load X joist spacing X span length

= 0.57 X 0.4 X 6.175 = 1.407KN

$U_{inst,G}$ = bending deflection + sheer deflection

$$\frac{5}{384} \times \frac{f_d l^3}{EI} + \frac{12}{5} \times \frac{F_d L}{EA}$$

$U_{inst,G} = \frac{5}{384}\left(\frac{(1.407\times 10^3(6.175\times 10^3)^3}{12675\times 50.0\times 10^6}\right) + \frac{12}{5} \times \frac{1.407\times 10^3\times 6.175\times 10^3}{12675\times 15.1\times 10^3}$ = 6.915mm

B) instantaneous deflection due to variable action, $U_{inst}Q$

Y_G for serviceability state = 1.0

Factored variable load, Q = Y_QQ_k = 1.0 X 1.5 = 1.5KN/M_2

Factored variable action per joist, F_d,Q

F_d, Q = total load X joist spacing X span length

= 1.5 X 0.4 X 6.175 = 3.705KN

Instantaneous deflection due to variable load, $U_{inst}Q$ is given by

$U_{inst}Q$ = bending deflection + sheer deflection

$$\frac{5}{384}\times\frac{f_d l^3}{EI}+\frac{12}{5}\times\frac{F_d L}{EA}$$

$$\frac{5}{384}\left(\frac{3.705\times 10^3(6.175\times 10^3)^3}{12675\times .50\times 10^6}\right)+\frac{12}{5}\times\frac{3.705\times 10^3\times 6.175\times 10^3}{12675\times 15.1\times 10^3}$$

=18.210mm

(C.) Final deflection due to permanent actions (table 3.2, EC5)

For solid timber timbers subject to service class land

Medium – term loading, K_{def} = 0.6

Final deflection due to permanent actions

$U_{fin,}$ G^1 is given by

$U_{fin,G}$ = U_{inG} (1+ K_{def})= 6.915X (1 + 0.6) = 11.064mm

(Cl, 2.4.1,Ec2/cl6) Ψ_2 = 0.3

Final deflection due to variable action, $U_{fin,Q,}$ is given by

$U_{fin,Q,}$ = U_{insQ1I} (1 + Ψ_2 K_{def})

= 18.210(1 + 0.3 x 0.6) = 21.48mm

D) check for final deflection

Total deflection, $U_{fin} = U_{finG} + U_{finQ}$ = 11.064+ 13.06

= 24.76mm

Permissible final deflection assuming the floor support bristle finishes). W_{fin}; is

$W_{fin} = \frac{1}{250}$ X span

1/250 X 6.175 x 10^3 = 24.7mm

16mm > 9mm OK

Therefore, 75mm X 125mm Joist are adequate in deflection

LATERAL BUCKLING (CL 6.3.3, EC 5)

The check is unnecessary as the compressive edge cannot move totally because the joist is attached to tongue and groove boarding.

SHEAR

Design shear strength is

$F_{v,d} = K_{sys}K_{mod} \frac{f_{y,k}}{\gamma_m} - 1.1 \times 0.8 \times \frac{3.75}{1.3}$

Maximum shear force is

$V_{Ed} = \frac{F_d}{2} = \frac{7.39 \times 10^3}{2} = 3.69 \times 10^3 N$

Design shear stress at Neural axis is

$\tau_d = \frac{1.5\, V_d}{A} = \frac{1.5 \times 3.69 \times 10^3}{15 \times 10^3} = 0.369 N/mm^2 < f_{v,d}$ OK

BEARING

Design compressive stress

I) Design bearing force is

$F_{90,d} = = \frac{F_d}{2} = \frac{7.39 \times 10^3}{2}$ $= 3.69 \text{ X } 10^3$

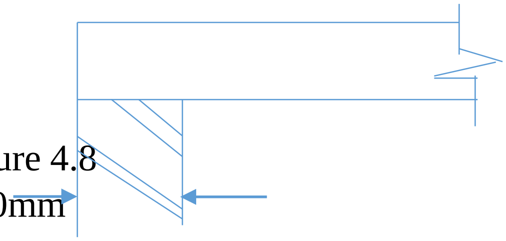


figure 4.8
100mm

Taking the floor joist to span assumptously onto 10mm wide walls as shown above, the bearing stress is given by:

$\delta_{c,90,d} = \frac{F_{90,d}}{bl} = \frac{3.69 \times 10^3}{75 \times 200}$ $= 0.246 \text{N/mm}^2$

J) Design compressive strength

Design compressive strength parallel to grain $f_{c,90,\,d}$ is given by

$F_{c,90,d} = K_{sys} K_{mod} \frac{f_{c,90,k}}{\gamma_m} = 1.1 \times 0.8 \times \frac{5.05}{1.3}$

$= 3.42 \text{N/mm}^2$

Bearing capacity

The above diagram

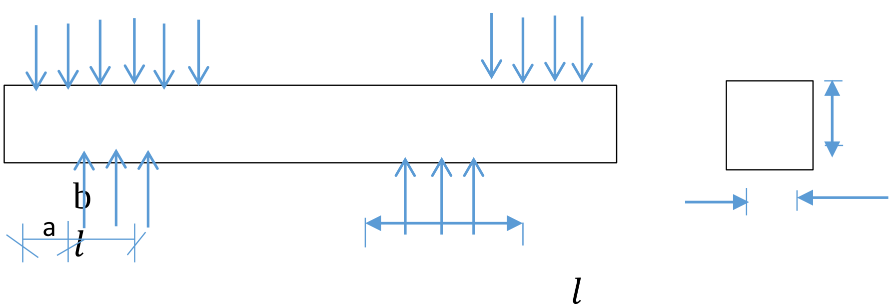


Figure 4.9

From the above diagram

a = overhanging length = O for the structure

l = size of wall = 100mm

h = height of joist = 125mm

b = breadth of joist = 75mm

a = o, L = 100m and h = 100mm

since a<h/3

$K_{c,90} = \left(2.38 - \frac{l}{250}\right)\left(1 + \frac{h}{12l}\right) = \left(2.38 - \frac{100}{250}\right)\left(1 + \frac{200}{12 \times 100}\right) =$ $(1.98)(1.166)$

$= 2.31 < 4$ OK

$K_c, F_{G90,d} = 2.31 \text{ X } 3.42 = 7.90\text{N/mm}^2 > \delta_{c90,d}$ OK

CHECK ASSUMED SELF WEIGHT OF JOISTS

From the Nigerian timber classification table

Density of class N2 timber is 770kg/m^2

$$SW = \frac{b\,h\,p\,g}{spacing}$$

$$SW = \frac{(75 \times 125 \times 10^{-6}) \times 770 \times 9.8 \times \times 10^{-3}}{0.4} = 0.17KNm^2$$

Assumed self-weight > actual self-weight. hence assumed self-weight is ok

4.2 BEAM DESIGN

DESIGN LOADING

A). Permanent action (G_K);

Roof load (live and dead)	=	1.50KN/m^2
Timber wall board load	=	0.30 KN/m^2
Self weight of beam	=	0.30 KN/m^2
Total permanent action (G_k)	=	2.1KN/M^2

B). Variable action, Q_K :

Imposed floor loads for offices (table 3, EN 1991-1-1) =1.50KN/M^2

DESIGN ACTION

Total design load is

$= \gamma_G G_k + \gamma_Q Q_k =$

$= 1.35 \times 2.1 + 1.5 \times 1.5 = 5.085 KN/M^2$

.

CACULATIONS

Design parameters

The grade of the seasoned timber used is of N2 class

Characteristics strengths and modulus of elasticity for timer of strength class N2

Values in Nmm^{-2} are given below

Table 4.5

Bending strength ($F_{M,K}$)	Compression perpendicular to grain ($F_{C,90,K}$)	Shear parallel to grain (Fv,k)	Modulus of elasticity (60, mean)
33.17	5.05	3.75	12675

K_3 = duration of loading = 1.0

K_8 = loading sharing system = 1.1

K_7 = depth factor = $\left(\frac{300}{h}\right)$ 0.11

- The beam will be designed for service class 1 (clause 2.3.1.3, EC5)
- The beam will carry both variable and permanent load, critical load duration class – medium term (table 1, EC5)
- K_{mod} (table 3.1, EC5) – 0.80
- Y_m (Table 2.3 EC 5) – 1.3
- The beam is from part of a load sharing system hence (K_{sys} = 1.1)
- - assure K_h = 1

BEAMS

Design of beams will equivalent spans are similar according to Engr V.O Oyenuga

the following beams will be designed to represent others:

a) **Beams supporting walls and floor**

Beam A to represent beams that span from 1800mm – 2500mm

Beam b to represent beams that span from 3500mm – 4000mm

Beam c to represent beams that span from 5500mm – 6000mm

b) **Beams supporting walls, floor and Beam**

Beam d to represent beans that span from 1500mm – 2000mm

Beam e to represent beans that span from 3500mm – 4000mm

Beam f to represent beans that span from 4500mm – 5000m

4.2.1 Beams supporting walls and floor

BEAM A

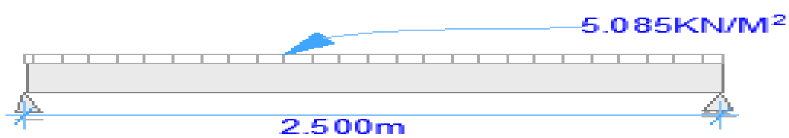


Figure 4.10

Bending (Cl 6.1.6, EC5)

Bending moment

$M_{d,y} = \frac{wl^2}{8} = \frac{5.085x2.5^2}{8} = 3.97K_{NM}$

Design bending strength about Y-Y axis

$F_{m,y,d} = K_h.K_{sys}.K_{mod}.\frac{f_{m,k}}{Y_m} = \frac{1.0\times1.1\times0.8\times33.19}{1.3}$

$F_{m,\ yd.} = 22.47N/mm^2$

Bending (Cl 6.1.6, EC 5)

$\frac{\sigma_{m,y,d}}{F_{m,y,d}}$ + $K_m \frac{\sigma_{m,z,d}}{F_{m,z,d}} \leq 1$

K_m = 0.7 – for rectangular / square timber section

$\frac{\sigma_{m,y,d}}{22.47}$ + 0.7 x $\frac{0}{F_{m,z,d}} \leq 1$

$\sigma_{m,y,d} \leq 22.47 N/mm^2$

$W_{y,req} \geq \frac{M_{d,y}}{\sigma_{m,y,d}} = \frac{3.97 \times 10^6}{22.47}$

$=176.68 \times 10^3 mm^3$

From table for classification of Nigerian timber (Aguwa, 2016)

75mm X 150mm Beam would be suitable

$W_y = 281 X\ 10^3\ mm^3$, $I_y = 21.09\ X\ 10^6\ MM^4$, $A = 11.3 X\ 10^3\ mm^2$

Deflection (Cl 7.2 , EC5)

A). instatenous deflection due to permanent action, $U_{inst}G$ (from table 2.3, EC 5) Y_G for serviceability limit state = 1.0

Factored permanent load, G = $Y_G\ G_k$ = 1.0 X 2.1

= 2.1kN/m^2

$U_{inst,G}$ = bending deflection + shear deflection

$$\frac{5}{384} \times \frac{f_d l^3}{EI} + \frac{12}{5} \times \frac{F_d L}{EA}$$

$U_{inst,G} = \frac{5}{384} \left(\frac{(2.1 \times 10^3 (2.5 \times 10^3)^3}{12675 \times 21.09 \times 10^6}\right) + \frac{12}{5} \times \frac{2.1 \times 10^3 \times 2.5 \times 10^3}{12675 \times 11.3 \times 10^3}$

= 1.68mm

B) instantaneous deflection due to variable action, $U_{inst}Q$

Y_G for serviceability state = 1.0

Factored variable load, Q = $Y_Q Q_k$ = 1.0 X 1.5 = 1.5KN/M_2

Instantaneous deflection due to variable load, $U_{inst}Q$ is given by

$U_{inst}Q$ = bending deflection + sheer deflection

$$\frac{5}{384} \times \frac{f_d l^3}{EI} + \frac{12}{5} \times \frac{F_d L}{EA}$$

$$\frac{5}{384}\left(\frac{1.5\times10^3(2.5\times10^3)^3}{12675\times21.09\times10^6}\right) + \frac{12}{5} \times \frac{1.5\times10^3\times2.5\times10^3}{12675\times11.3\times10^3}$$

=1.2mm

(C.) Final deflection due to permanent actions (table 3.2, EC5)

For solid timber timbers subject to service class land

Medium – term loading, K_{def} = 0.6

Final deflection due to permanent actions

$U_{fin,}$ G^1 is given by

$U_{fin,G} = U_{inG}$ (1+ K_{def})= 1.68X (1 + 0.6) = 2.69mm

(Cl, 2.4.1,Ec2/cl6) $\Psi_2 = 0.3$

Final deflection due to variable action, $U_{fin,\ Q,}$ is given by

$U_{fin,Q,} = U_{insQ1I}$ (1 + Ψ_2 K_{def})

= 1.2 (1 + 0.3 x 0.6) = 1.42mm

D) check for final deflection

Total deflection, $U_{fin} = U_{finG} + U_{finQ}$ = 2.69+ 1.42

= 4.11mm

Permissible final deflection assuming the floor support bristle finishes). W_{fin}; is

$W_{fin} = \frac{1}{250}$ X span

1/250 X 2.5 x 10^3 = 10mm

10mm > 4.11mm OK

Therefore, 75mm X 150mm beam are adequate in deflection

LATERAL BUCKLING (CL 6.3.3, EC 5)

$l_{ef} = 0.9l + 2h = 0.9 \times 2500 + 2 \times 150 = 2550$mm

.

$\sigma_{m,crit} = \frac{0.78b^2}{hl_{ef}} E_{0,05} \quad = \quad \frac{0.78 \times 75^2}{150 \times 2550} \times 5.4 \times 10^3 = 61.94$

$\lambda_{rel,m} = \sqrt{\frac{f_{m,k}}{\sigma_{m,crit}}} = \sqrt{\frac{33.17}{61.94}} = 0.731$

since $\lambda < 0.75, k_{crit} = 1$

$k_{crit}\, f_{m,yd} = k_{crit}(K_h.K_{sys.}K_{mod.} \frac{f_{m,k}}{\Upsilon_m}) = 1.0 \left(\frac{1.0 \times 1.1 \times 0.8 \times 33.19}{1.3}\right)$

= 22.4 N/mm

$\sigma_{m,d} = \frac{M_{y,d}}{bh^2/6} = \frac{3970}{75 \times 150^2/6} = 0.014 N/mm$

since $\sigma_{m,d} \leq \quad k_{crit}\, f_{m,d}$ it is ok

SHEAR

Design shear strength is

$F_{v,d} = K_{sys}K_{mod} \frac{f_{y,k}}{\gamma_m} - 1.1 \times 0.8 \times \frac{3.75}{1.3} = 2.54$

Maximum shear force is

$V_{Ed} = \frac{wl}{2} = \frac{5.085 \times 10^3 \times 2.5}{2} = 6.356 \times 10^3 N$

Design shear stress at Neural axis is

$\tau_d = \frac{1.5\, V_d}{A} = \frac{1.5 \times 6.356 \times 10^3}{11.3 \times 10^3} = 0.84$N/mm^2 <$f_{v,d}$ OK

BEARING

Design compressive stress

K) Design bearing force is

$$F_{90,d} = \frac{wl}{2} = \frac{5.085 \times 10^3 \times 2.5}{2} = 6.356 \times 10^3 N$$

Taking the beam to span onto 100mm(assumed) of nearby column as shown above, the bearing stress is given by:

$$\sigma_{c,90,d} = \frac{F_{90,d}}{bl} = \frac{6.356 \times 10^3}{75 \times 150} = 0.565\text{N/mm}^2$$

L) Design compressive strength

Design compressive strength parallel to grain $f_{c,90,\,d}$ is given by

$$F_{c,90,d} = K_{sys}\,K_{mod}\,\frac{f_{c,90,k}}{\gamma_m} = 1.1 \times 0.8 \times \frac{5.05}{1.3}$$

$= 3.42\text{N/mm}^2$

Bearing capacity

The above diagram

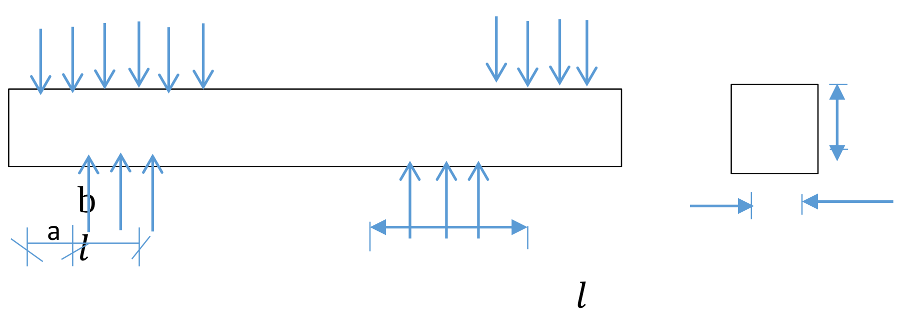


Figure 4.11

From the above diagram

a = overhanging length = O for the structure

l = size of wall = 100mm

h = height of BEAM = 150mm

b = breadth of BEAM = 75mm

a = o, L = 100m and h = 100mm

since a<h/3

$K_{c,90} = \left(2.38 - \frac{l}{250}\right)\left(1 + \frac{h}{12l}\right) = \left(2.38 - \frac{100}{250}\right)\left(1 + \frac{150}{12 \times 100}\right) = (1.98)(1.125)$

$= 2.23 < 4$ OK

.

$K_c, F_{G90,d} = 2.23X\ 3.42 = 7.63N/mm^2 > \sigma_{c90,d}$ OK

BEAM B

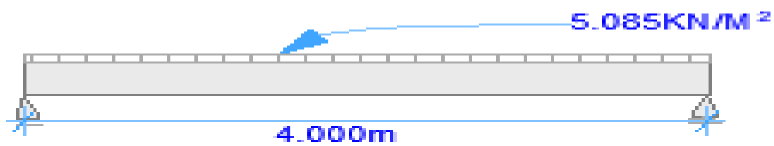


Figure 4.12

Bending (Cl 6.1.6, EC5)

Bending moment

$M_{d,y} = \frac{wl^2}{8} = \frac{5.085x4^2}{8} = 10.17K_{NM}$

Design bending strength about Y-Y axis

$F_{m,y,d} = K_h.K_{sys.}K_{mod.}\frac{f_{m,k}}{Y_m} = \frac{1.0 \times 1.1 \times 0.8 \times 33.19}{1.3}$

$F_{m,\ yd.} = 22.47N/mm^2$

Bending (Cl 6.1.6, EC 5)

$\frac{\sigma_{m,y,d}}{F_{m,y,d}} + K_m \frac{\sigma_{m,z,d}}{F_{m,z,d}} \leq 1$

$K_m = 0.7$ – for rectangular / square timber section

$\frac{\sigma_{m,y,d}}{22.47} + 0.7\ x\ \frac{0}{F_{m,z,d}} \leq 1$

$\sigma_{m,y,d} \leq 22.47N/mm^2$

$$W_{y,req} \geq \frac{M_{d,y}}{\sigma_{m,y,d}} = \frac{10.17 \times 10^6}{22.47}$$

$=452.60 \times 10^3 mm^3$

From table for classification of Nigerian timber (Aguwa, 2016)

100mm X 200mm Beam would be suitable

W_y = 667X 10^3 mm^3, I_y = 66.67 X 10^6 MM^4, A = 20 X 10^3 mm^2

Deflection (Cl 7.2 , EC5)

A). instatenous deflection due to permanent action, $U_{inst}G$ (from table 2.3, EC 5) Y_G for serviceability limit state = 1.0

Factored permanent load, G = Y_G G_k = 1.0 X 2.1

= 2.1kN/m^2

$U_{inst,G}$ = bending deflection + shear deflection

$$\frac{5}{384} \times \frac{f_d l^3}{EI} + \frac{12}{5} \times \frac{F_d L}{EA}$$

$$U_{inst,G} = \frac{5}{384}\left(\frac{(2.1 \times 10^3 (4 \times 10^3)^3}{12675 \times 66.67 \times 10^6}\right) + \frac{12}{5} \times \frac{2.1 \times 10^3 \times 4 \times 10^3}{12675 \times 20 \times 10^3}$$

= 2.15mm

B) instantaneous deflection due to variable action, $U_{inst}Q$

Y_G for serviceability state = 1.0

Factored variable load, Q = $Y_Q Q_k$ = 1.0 X 1.5 = 1.5KN/M_2

Instantaneous deflection due to variable load, $U_{inst}Q$ is given by

$U_{inst}Q$ = bending deflection + sheer deflection

$$\frac{5}{384} \times \frac{f_d l^3}{EI} + \frac{12}{5} \times \frac{F_d L}{EA}$$

$$\frac{5}{384}\left(\frac{1.5\times10^3(4\times10^3)^3}{12675\times66.67.09\times10^6}\right) + \frac{12}{5} \times \frac{1.5\times10^3\times4\times10^3}{12675\times20\times10^3}$$

.　=1.53mm

(C.) Final deflection due to permanent actions (table 3.2, EC5)

For solid timber timbers subject to service class land

Medium – term loading, K_{def} = 0.6

Final deflection due to permanent actions

$U_{fin,}$ G^1 is given by

$U_{fin,G} = U_{inG}(1 + K_{def}) = 2.15X(1 + 0.6) = 3.44$mm

(Cl, 2.4.1,Ec2/cl6) $\Psi_2 = 0.3$

Final deflection due to variable action, $U_{fin,Q,}$ is given by

$U_{fin,Q,} = U_{insQ1I}(1 + \Psi_2 K_{def})$

= 1.53(1 + 0.3 x 0.6) = 1.80mm

D) check for final deflection

Total deflection, $U_{fin} = U_{finG} + U_{finQ} = 3.44 + 1.80$

= 5.24mm

Permissible final deflection assuming the floor support bristle finishes). W_{fin}; is

$W_{fin} = \frac{1}{250}$ X span

1/250 X 4 x 10^3 = 16mm

16mm > 5.24mm OK

Therefore, 100mm X 200mm beam are adequate in deflection

LATERAL BUCKLING (CL 6.3.3, EC 5)

$l_{ef} = 0.9l + 2h = 0.9 \times 4000 + 2 \times 200 = 4000$mm

$\sigma_{m,crit} = \frac{0.78b^2}{hl_{ef}} E_{0,05} = \frac{0.78\times100^2}{200\times4000} \times 5.4 \times 10^3 = 52.6$

.

$\lambda_{rel,m} = \sqrt{\frac{f_{m,k}}{\sigma_{m,crit}}} = \sqrt{\frac{33.17}{52.6}} = 0.79$

since $\lambda_{rel,m} > 0.75, k_{crit} = 1.5$

$k_{crit}\, f_{m,yd} = k_{crit}(K_h.K_{sys.}K_{mod.} \frac{f_{m,k}}{\Upsilon_m}) = 1.5\,(\frac{1.0\times1.1\times0.8\times33.19}{1.3})$

= 33.6N/mm

$$\sigma_{m,d} = \frac{M_{y,d}}{bh^2/_6} = \frac{10170}{100\times200^2/_6} = 0.015N/mm$$

since $\sigma_{m,d} \leq k_{crit}\, f_{m,d}$ it is ok

The check is unnecessary as the compressive edge cannot move totally because the Beam is attached to tongue and groove boarding.

SHEAR

Design shear strength is

$F_{v,d} = K_{sys}K_{mod} \frac{f_{y,k}}{\gamma_m} - 1.1 \times 0.8 \times \frac{3.75}{1.3} = 2.54$

Maximum shear force is

$V_{Ed} = \frac{wl}{2} = \frac{5.085\times10^3\times4}{2} = 10.17 \times 10^3 N$

Design shear stress at Neural axis is

$\tau_d = \frac{1.5\,V_d}{A} = \frac{1.5\times10.17\times10^3}{20\times10^3} = 0.76N/mm^2 < f_{v,d}$ OK

BEARING

Design compressive stress

M) Design bearing force is

$$F_{90,d} = \frac{wl}{2} = \frac{5.085 \times 10^3 \times 4}{2} = 10.17 \times 10^3 N$$

100mm

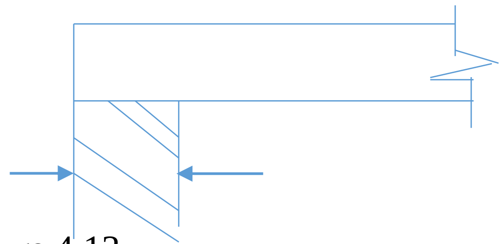


Figure 4.13

Taking the beam to span onto 100mm(assumed) of nearby column as shown above, the bearing stress is given by:

$$\sigma_{c,90,d} = \frac{F_{90,d}}{bl} = \frac{10.17 \times 10^3}{100 \times 200} = 0.509 \text{N/mm}^2$$

N) Design compressive strength

Design compressive strength parallel to grain $f_{c,90,d}$ is given by

$$F_{c,90,d} = K_{sys} K_{mod} \frac{f_{c,90,k}}{\gamma_m} = 1.1 \times 0.8 \times \frac{5.05}{1.3}$$

$= 3.42 \text{N/mm}^2$

Bearing capacity

The above diagram

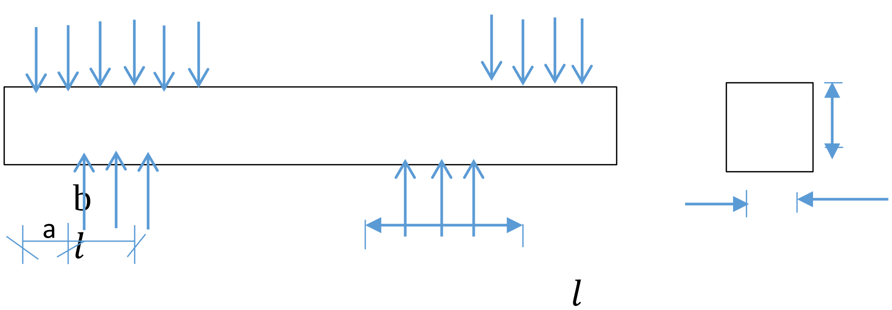


Figure 4.14

From the above diagram

a = overhanging length = O for the structure

l = size of wall = 100mm

h = height of beam = 200mm

b = breadth of beam = 100mm

a = o, L = 100m and h = 100mm

since a<h/3

$K_{c,90} = \left(2.38 - \frac{l}{250}\right)\left(1 + \frac{h}{12l}\right) = \left(2.38 - \frac{100}{250}\right)\left(1 + \frac{200}{12 \times 100}\right) = (1.98)(1.16)$

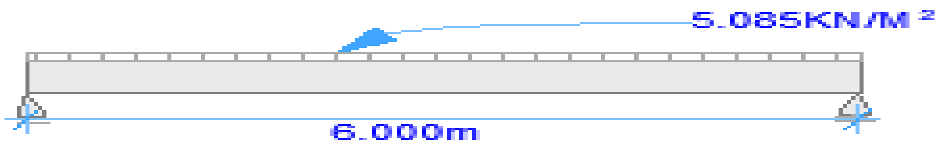
= 2.296 < 4 OK

$K_{c,} F_{G90,d}$ = 2.23X 3.42 = 7.63N/mm^2 $> \sigma_{c90,d}$ OK

Beams supporting walls and floor

BEAM C

Figure 4.15



Bending (Cl 6.1.6, EC5)

Bending moment

$M_{d,y} = \frac{wl^2}{8} = \frac{5.085x6^2}{8} = 22.885K_{NM}$

Design bending strength about Y-Y axis

$F_{m,y,d} = K_h.K_{sys.}K_{mod.} \frac{f_{m,k}}{Y_m} = \frac{1.0 \times 1.1 \times 0.8 \times 33.19}{1.3}$

$F_{m,\ yd.}$ = 22.47N/mm^2

Bending (Cl 6.1.6, EC 5)

$\frac{\sigma_{m,y,d}}{F_{m,y,d}} + K_m \frac{\sigma_{m,z,d}}{F_{m,z,d}} \leq 1$

K_m = 0.7 – for rectangular / square timber section

$\frac{\sigma_{m,y,d}}{22.47}$ + 0.7 x $\frac{0}{F_{m,z,d}} \leq 1$

$\sigma_{m,y,d} \leq 22.47N/mm^2$

$$W_{y,req} \geq \frac{M_{d,y}}{\sigma_{m,y,d}} = \frac{22.885 \times 10^6}{22.47}$$

$=1018.4 \times 10^3 mm^3$

From table for classification of Nigerian timber (Aguwa, 2016)

100mm X 300mm Beam would be suitable

W_y = 1500X 10^3 mm^3, I_y = 225 X 10^6 MM^4, A = 30 X 10^3 mm^2

Deflection (Cl 7.2 , EC5)

A). instatenous deflection due to permanent action, $U_{inst}G$ (from table 2.3, EC 5) Y_G for serviceability limit state = 1.0

Factored permanent load, G = Y_G G_k = 1.0 X 2.1

= 2.1kN/m^2

$U_{inst,G}$ = bending deflection + shear deflection

$$\frac{5}{384} \times \frac{f_d l^3}{EI} + \frac{12}{5} \times \frac{F_d L}{EA}$$

$U_{inst,G} = \frac{5}{384}\left(\frac{(2.1 \times 10^3 (6 \times 10^3)^3}{12675 \times 225 \times 10^6}\right) + \frac{12}{5} \times \frac{2.1 \times 10^3 \times 6 \times 10^3}{12675 \times 30 \times 10^3}$

= 2.07mm

B) instantaneous deflection due to variable action, $U_{inst}Q$

Y_G for serviceability state = 1.0

Factored variable load, Q = $Y_Q Q_k$ = 1.0 X 1.5 = 1.5KN/M_2

Instantaneous deflection due to variable load, $U_{inst}Q$ is given by

$U_{inst}Q$ = bending deflection + sheer deflection

$$\frac{5}{384} \times \frac{f_d l^3}{EI} + \frac{12}{5} \times \frac{F_d L}{EA}$$

$$\frac{5}{384}\left(\frac{1.5\times10^3(6\times10^3)^3}{12675\times225\times10^6}\right)+\frac{12}{5}\times\frac{1.5\times10^3\times6\times10^3}{12675\times30\times10^3}$$

=1.48mm

(C.) Final deflection due to permanent actions (table 3.2, EC5)

For solid timber timbers subject to service class land

Medium – term loading, K_{def} = 0.6

Final deflection due to permanent actions

$U_{fin,}$ G^1 is given by

$U_{fin,G} = U_{inG}$ (1+ K_{def})= 2.07X (1 + 0.6) = 3.312mm

(Cl, 2.4.1,Ec2/cl6) Ψ_2 = 0.3

Final deflection due to variable action, $U_{fin,\ Q,}$ is given by

$U_{fin,Q,} = U_{insQ1I}\,(1 + \Psi_2\, K_{def})$

= 1.48(1 + 0.3 x 0.6) = 1.75mm

D) check for final deflection

Total deflection, $U_{fin} = U_{finG} + U_{finQ}$ = 3.312+ 1.75

= 5.06mm

Permissible final deflection assuming the floor support bristle finishes). W_{fin}; is

$W_{fin} = \frac{1}{250}$ X span

1/250 X 6 x 10^3 = 16mm

24mm > 5.06mm OK

Therefore, 100mm X 300mm beam are adequate in deflection

LATERAL BUCKLING (CL 6.3.3, EC 5)

$l_{ef} = 0.9l + 2h = 0.9 \times 6000 + 2 \times 300 = 6000$mm

$\sigma_{\text{m,crit}} = \frac{0.78b^2}{hl_{ef}} E_{0,05} \quad = \frac{0.78\times100^2}{300\times3900} \times 5.4 \times 10^3 = 36$

$\lambda_{rel,m} = \sqrt{\frac{f_{m,k}}{\sigma_{m,crit}}} = \sqrt{\frac{33.17}{36}}$ =0.95

since $\lambda_{rel,m} > 0.75, k_{crit} = 1.5$

$k_{crit}\, f_{m,yd} = k_{crit}(\text{K}_{\text{h}}.\text{K}_{\text{sys.}}\text{K}_{\text{mod.}} \frac{f_{m,k}}{\Upsilon_m}) = 1.5\, (\frac{1.0\times1.1\times0.8\times33.17}{1.3})$

= 33.68N/mm

$$\sigma_{m,d} = \frac{M_{y,d}}{bh^2/6} = \frac{22885}{100\times300^2/6} = 0.0152N/mm$$

since $\sigma_{m,d} \leq \quad k_{crit}\, f_{m,d}$ it is ok

The check is unnecessary as the compressive edge cannot move totally because the Beam is attached to tongue and groove boarding.

SHEAR

Design shear strength is

$\text{F}_{\text{v,d}} = \text{K}_{\text{sys}}\text{K}_{\text{mod}} \frac{f_{y,k}}{\gamma_m} - 1.1 \times 0.8 \times \frac{3.75}{1.3} = 2.54$

Maximum shear force is

$\text{V}_{\text{Ed}} = \frac{wl}{2} = \frac{5.085 \times10^3\times6}{2} = 17.55 \times 10^3 N$

Design shear stress at Neural axis is

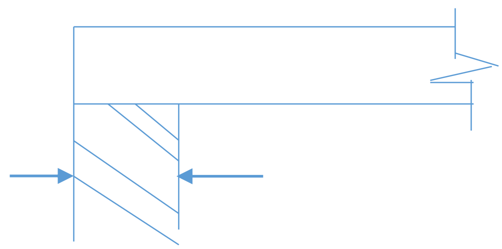
$\tau_{\text{d}} = \frac{1.5\, V_d}{A} = \frac{1.5 \times17.55\times10^3}{30 \times10^3} = 0.88\text{N/mm}^2 < \text{f}_{\text{v,d}}$ OK

BEARING

Design compressive stress

O) Design bearing force is

$\text{F}_{\text{9o,d}} = \frac{wl}{2} = \frac{5.085 \times10^3\times6}{2} = 15.255 \times 10^3 N$



100mm

figure 4.16

Taking the beam to span onto 100mm(assumed) of nearby column as shown above, the bearing stress is given by:

$$\sigma_{c,90,d} = \frac{F_{90,d}}{bl} = \frac{10.17 \times 10^3}{100 \times 300} = 0.339\text{N/mm}^2$$

P) Design compressive strength

Design compressive strength parallel to grain $f_{c,90,\,d}$ is given by

$$F_{c,90,d} = K_{sys}\,K_{mod}\,\frac{f_{c,90,k}}{\gamma_m} = 1.1 \times 0.8 \times \frac{5.05}{1.3}$$

$= 3.42\text{N/mm}^2$

Bearing capacity

The above diagram

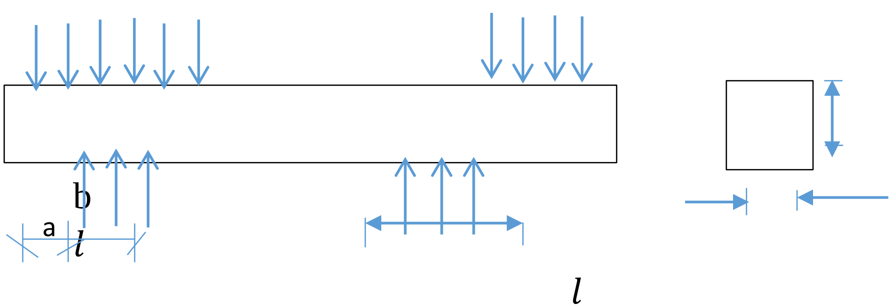


Figure 4.17

From the above diagram

a = overhanging length = O for the structure

l = size of wall = 100mm

h = height of beam = 300mm

b = breadth of beam = 100mm

a = o, L = 100m and h = 100mm

since a<h/3

.

$K_{c,90} = \left(2.38 - \frac{l}{250}\right)\left(1 + \frac{h}{12l}\right) = \left(2.38 - \frac{100}{250}\right)\left(1 + \frac{300}{12 \times 100}\right) =$ $(1.98)(1.25)$

$= 2.475 < 4$ OK

$K_{c,} F_{G90,d} = 2.475X\ 3.42 = 8.46N/mm^2 > \sigma_{c90,d}$ OK

4.2.2 Beams supporting walls, floor and Beam

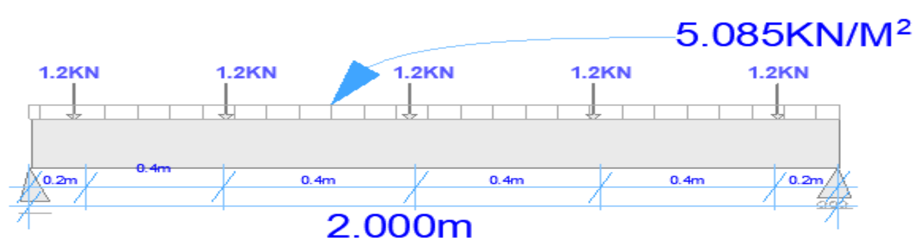
Beam D, to represent beans that span from 1500mm – 2000mm

Beam E, to represent beans that span from 3500mm – 4000mm

Beam F, to represent beans that span from 4500mm – 5000mm

BEAM D

Figure 4.18



Bending (Cl 6.1.6, EC5)

Bending moment

Considering the point loads due to the joists, singly

$Va = vb = 13.77KN$

.

$$Mmax = 2va - (1.2kn\ (0.6 + 0.2)m + 5.08\text{x}\frac{2^2}{2}) = 16.41knm$$

$$Md, y = 16.41knm$$

Design bending strength about Y-Y axis

$F_{m,y,d} = K_h.K_{sys.}K_{mod.}\ \frac{f_{m,k}}{Y_m} = \frac{1.0\times1.1\times0.8\times33.19}{1.3}$

$F_{m,\ yd.} = 22.47\text{N/mm}^2$

Bending (Cl 6.1.6, EC 5)

$\frac{\sigma_{m,y,d}}{F_{m,y,d}} + K_m\frac{\sigma_{m,z,d}}{F_{m,z,d}} \leq 1$

$K_m = 0.7$ – for rectangular / square timber section

$\frac{\sigma_{m,y,d}}{22.47} + 0.7 \text{ x } \frac{0}{F_{m,z,d}} \leq 1$

$\sigma_{m,y,d} \leq 22.47N/mm^2$

$W_{y,req} \geq \frac{M_{d,y}}{\sigma_{m,y,d}} = \frac{16.41knm\ \times 10^6}{22.47}$

$=730.30 \times 10^3 mm^3$

From table for classification of Nigerian timber (Aguwa, 2016)

100mm X 225mm Beam would be suitable

$W_y = 834\text{X } 10^3\ \text{mm}^3$, $I_y = 94.92\text{X } 10^6\ \text{MM}^4$, $A = 22.5\text{X } 10^3\ \text{mm}^2$

Deflection (Cl 7.2 , EC5)

A). instatenous deflection due to permanent action, $U_{inst}G$ (from table 2.3, EC 5) Y_G for serviceability limit state = 1.0

.

Factored permanent load, G = Y_G G_k = 1.0 X 2.1

= 2.1kN/m^2

$U_{inst,G}$ = bending deflection + shear deflection

$$\frac{5}{384} \times \frac{f_d l^3}{EI} + \frac{12}{5} \times \frac{F_d L}{EA}$$

$$U_{inst,G} = \frac{5}{384}\left(\frac{(2.1 \times 10^3(2\times10^3)^3}{12675 \times 94.92\times 10^6}\right) + \frac{12}{5} \times \frac{2.1 \times 10^3 \times 2\times10^3}{12675 \times 22.5 \times 10^3}$$

= 0.217mm

B) instantaneous deflection due to variable action, $U_{inst}Q$

Y_G for serviceability state = 1.0

Factored variable load, Q = $Y_Q Q_k$ = 1.0 X 1.5 = 1.5KN/M_2

Instantaneous deflection due to variable load, $U_{inst}Q$ is given by

$U_{inst}Q$ = bending deflection + sheer deflection

$$\frac{5}{384} \times \frac{f_d l^3}{EI} + \frac{12}{5} \times \frac{F_d L}{EA}$$

$$\frac{5}{384}\left(\frac{1.5\times10^3(2\times10^3)^3}{12675 \times 94.92 \times 10^6}\right) + \frac{12}{5} \times \frac{1.5 \times 10^3 \times 2\times10^3}{12675 \times 22.5\times10^3}$$

=0.357mm

(C.) Final deflection due to permanent actions (table 3.2, EC5)

For solid timber timbers subject to service class land

Medium – term loading, K_{def} = 0.6

Final deflection due to permanent actions

$U_{fin,}$ G^1 is given by

$U_{fin,G}$ = U_{inG} (1+ K_{def})= 0.217X (1 + 0.6) = 0.3472mm

(Cl, 2.4.1,Ec2/cl6) $\Psi_2 = 0.3$

Final deflection due to variable action, $U_{fin, Q,}$ is given by

$U_{fin,Q,} = U_{insQ1I} (1 + \Psi_2 K_{def})$

. = 0.357(1 + 0.3 x 0.6) = 0.421mm

D) check for final deflection

Total deflection, $U_{fin} = U_{finG} + U_{finQ}$ = 0.347+ 0.421

= 0.768mm

Permissible final deflection assuming the floor support bristle finishes). W_{fin}; is

$W_{fin} = \frac{1}{250}$ X span

1/250 X 2 x 10^3 = 8mm

8mm > 0.768mm OK

Therefore, 100mm X 225mm beam are adequate in deflection

LATERAL BUCKLING (CL 6.3.3, EC 5)

$l_{ef} = 0.9l + 2h = 0.9 \times 2000 + 2 \times 225 = 2150$mm

$\sigma_{m,crit} = \frac{0.78b^2}{hl_{ef}} E_{0,05} = \frac{0.78 \times 100^2}{225 \times 2150} \times 5.4 \times 10^3 = 87.069knm$

$\lambda_{rel,m} = \sqrt{\frac{f_{m,k}}{\sigma_{m,crit}}} = \sqrt{\frac{33.17}{87.069}} = 0.617$

since $\lambda_{rel,m} < 0.75, k_{crit} = 1$

$k_{crit}\, f_{m,yd} = k_{crit}(K_h.K_{sys}.K_{mod}. \frac{f_{m,k}}{Y_m}) = 1(\frac{1.0 \times 1.1 \times 0.8 \times 33.19}{1.3})$

= 22.46N/mm

$\sigma_{m,d} = \frac{M_{y,d}}{bh^2/6} = \frac{1641}{100 \times 225^2/6} = 1.944 \times 10^{-3}$

since $\sigma_{m,d} \leq \quad k_{crit}\, f_{m,d}$ it is ok

The check is unnecessary as the compressive edge cannot move totally because the Beam is attached to tongue and groove boarding.

SHEAR

Design shear strength is

$F_{v,d} = K_{sys}K_{mod}\,\frac{f_{y,k}}{\gamma_m} - 1.1\ \times 0.8\ \times\ \frac{3.75}{1.3} = 2.54$

Maximum shear force is

$V_{Ed} = va\ = 8.685 \times 10^3 N$

Design shear stress at Neural axis is

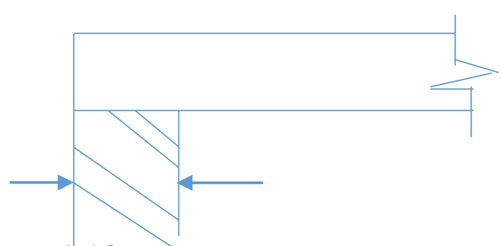
$\tau_d = \frac{1.5\,V_d}{A} = \frac{1.5\ \times 8.685 \times 10^3}{22.5\ \times 10^3}\ = 0.579\text{N/mm}^2 < f_{v,d}$ OK

BEARING

Design compressive stress

Q) Design bearing force is

$F_{90,d} = va\ = 8.685 \times 10^3 N$



100mm

Figure 4.19

Taking the beam to span onto 100mm(assumed) of nearby column as shown above, the bearing stress is given by:

$\sigma_{c,90,d} = \frac{F_{90,d}}{bl} = \frac{8.685 \times 10^3}{100 \times 225} = 0.386\text{N/mm}^2$

R) Design compressive strength

Design compressive strength parallel to grain $f_{c,90,\,d}$ is given by

$F_{c,90,d} = K_{sys}\,K_{mod}\,\frac{f_{c,90,k}}{\gamma_m} = 1.1 \times 0.8 \times \frac{5.05}{1.3}$

$= 3.42N/mm^2$

Bearing capacity

The above diagram

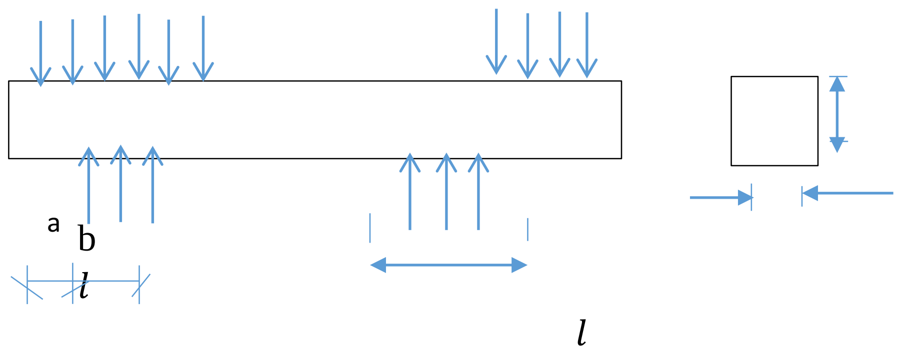


Figure 4.20

From the above diagram

a = overhanging length = O for the structure

l = size of wall = 100mm

h = height of beam = 225mm

b = breadth of beam = 100mm

a = o, L = 100m and h = 100mm

since a<h/3

$K_{c,90} = \left(2.38 - \frac{l}{250}\right)\left(1 + \frac{h}{12l}\right) = \left(2.38 - \frac{100}{250}\right)\left(1 + \frac{225}{12 \times 100}\right) = (1.98)(1.19)$

$= 2.35 < 4$ OK

$K_{c,}\,F_{G90,d} = 2.35X\ 3.42 = 8.06N/mm^2 > \sigma_{c90,d}$ OK

BEAM E

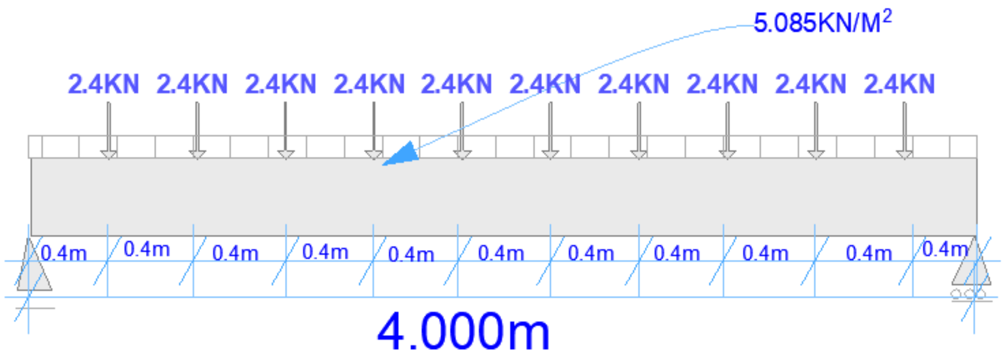


Figure 4.21

Bending (Cl 6.1.6, EC5)

Bending moment

Considering the point loads due to the joists, singly

$Va \ = \ vb \ = 23.7KN$

$$Mmax \ = 4va - (2.4kn \ (4)m + 5.08\text{x}\frac{4^2}{2}) = 26.97knm$$

$$Md, y \ = 44.52knm$$

Design bending strength about Y-Y axis

$F_{m,y,d} = K_h.K_{sys.}K_{mod.} \frac{f_{m,k}}{Y_m} = \frac{1.0 \times 1.1 \times 0.8 \times 33.19}{1.3}$

$F_{m,\ yd.} = 22.47 N/mm^2$

Bending (Cl 6.1.6, EC 5)

$\frac{\sigma_{m,y,d}}{F_{m,y,d}} + K_m \frac{\sigma_{m,z,d}}{F_{m,z,d}} \leq 1$

$K_m = 0.7$ – for rectangular / square timber section

$\frac{\sigma_{m,y,d}}{22.47} + 0.7 \text{ x } \frac{0}{F_{m,z,d}} \leq 1$

$\sigma_{m,y,d} \leq 22.47N/mm^2$

$$W_{y,req} \geq \frac{M_{d,y}}{\sigma_{m,y,d}} = \frac{44.52knm \times 10^6}{22.47}$$

$=1200.26 \times 10^3 mm^3$

From table for classification of Nigerian timber (Aguwa, 2016)

100mm X 300mm Beam would be suitable

W_y = 1500X 10^3 mm^3, I_y = 225X 10^6 MM^4, A = 30X 10^3 mm^2

Deflection (Cl 7.2 , EC5)

A). instatenous deflection due to permanent action, $U_{inst}G$ (from table 2.3, EC 5) Y_G for serviceability limit state = 1.0

Factored permanent load, G = Y_G G_k = 1.0 X 2.1

= 2.1kN/m^2

$U_{inst,G}$ = bending deflection + shear deflection

$$\frac{5}{384} \times \frac{f_d l^3}{EI} + \frac{12}{5} \times \frac{F_d L}{EA}$$

$U_{inst,G} = \frac{5}{384}\left(\frac{(2.1 \times 10^3 (4 \times 10^3)^3}{12675 \times 225 \times 10^6}\right) + \frac{12}{5} \times \frac{2.1 \times 10^3 \times 4 \times 10^3}{12675 \times 30 \times 10^3}$

= 0.614mm

B) instantaneous deflection due to variable action, $U_{inst}Q$

Y_G for serviceability state = 1.0

Factored variable load, Q = $Y_Q Q_k$ = 1.0 X 1.5 = 1.5KN/M_2

Instantaneous deflection due to variable load, $U_{inst}Q$ is given by

$U_{inst}Q$ = bending deflection + sheer deflection

$$\frac{5}{384} \times \frac{f_d l^3}{EI} + \frac{12}{5} \times \frac{F_d L}{EA}$$

$\frac{5}{384}\left(\frac{1.5 \times 10^3 (4 \times 10^3)^3}{12675 \times 225 \times 10^6}\right) + \frac{12}{5} \times \frac{1.5 \times 10^3 \times 4 \times 10^3}{12675 \times 30 \times 10^3}$

=0.438mm

(C.) Final deflection due to permanent actions (table 3.2, EC5)

For solid timber timbers subject to service class land

Medium – term loading, K_{def} = 0.6

Final deflection due to permanent actions

$U_{fin,}$ G^1 is given by

$U_{fin,G} = U_{inG}$ (1+ K_{def})= 0.614X (1 + 0.6) = 0.982mm

(Cl, 2.4.1,Ec2/cl6) Ψ_2 = 0.3

Final deflection due to variable action, $U_{fin, Q,}$ is given by

$U_{fin,Q,} = U_{insQ1I}$ (1 + Ψ_2 K_{def})

= 0.438(1 + 0.3 x 0.6) = 0.517mm

D) check for final deflection

Total deflection, $U_{fin} = U_{finG} + U_{finQ}$ = 0.982+ 0.517

= 0.768mm

Permissible final deflection assuming the floor support bristle finishes). W_{fin}; is

$W_{fin} = \frac{1}{250}$ X span

1/250 X 4 x 10^3 = 8mm

16mm > 0.768mm OK

Therefore, 100mm X 300mm beam are adequate in deflection

LATERAL BUCKLING (CL 6.3.3, EC 5)

$l_{ef} = 0.9l + 2h = 0.9 \times 4000 + 2 \times 300 = 4200$mm

$\sigma_{m,crit} = \frac{0.78b^2}{hl_{ef}} E_{0,05} = \frac{0.78 \times 100^2}{300 \times 2150} \times 5.4 \times 10^3 = 65.302knm$

$\lambda_{rel,m} = \sqrt{\frac{f_{m,k}}{\sigma_{m,crit}}} = \sqrt{\frac{33.17}{65.302}} = 0.712$

since $\lambda_{rel,m} < 0.75, k_{crit} = 1$

$k_{crit}\, f_{m,yd} \;=\; k_{crit}(K_h.K_{sys.}K_{mod.}\,\frac{f_{m,k}}{\gamma_m}) = 1(\frac{1.0\times1.1\times0.8\times33.19}{1.3})$

= 22.46N/mm

$$\sigma_{m,d} = \frac{M_{y,d}}{bh^2/6} = \frac{2697}{100\times300^2/6} = 1.798\times10^{-3}$$

since $\sigma_{m,d} \leq \; k_{crit}\, f_{m,d}$ it is ok

The check is unnecessary as the compressive edge cannot move totally because the Beam is attached to tongue and groove boarding.

SHEAR

Design shear strength is

$F_{v,d} = K_{sys}K_{mod}\,\frac{f_{y,k}}{\gamma_m} - 1.1 \times 0.8 \times \frac{3.75}{1.3} = 2.54$

Maximum shear force is

$V_{Ed} = va = 23.7 \times 10^3 N$

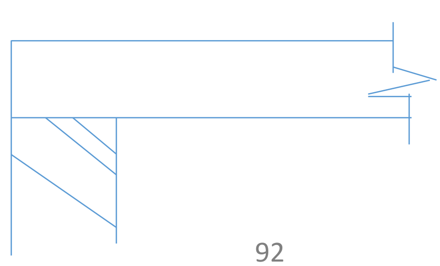
Design shear stress at Neural axis is

$\tau_d = \frac{1.5\, V_d}{A} = \frac{1.5 \times 23.7\times10^3}{30\times10^3} = 1.185\text{N/mm}^2 < f_{v,d}$ OK

BEARING

Design compressive stress

S) Design bearing force is
$F_{90,d} = va = 23.7 \times 10^3 N$



100mm

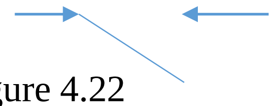


Figure 4.22

Taking the beam to span onto 100mm(assumed) of nearby column as shown above, the bearing stress is given by:

.

$$\sigma_{c,90,d} = \frac{F_{90,d}}{bl} = \frac{23.7\times10^3}{100\times300} = 0.79\text{N/mm}^2$$

T) Design compressive strength

Design compressive strength parallel to grain $f_{c,90,\,d}$ is given by

$$F_{c,90,d} = K_{sys}\,K_{mod}\,\frac{f_{c,90,k}}{\gamma_m} = 1.1 \times 0.8 \times \frac{5.05}{1.3}$$

$= 3.42\text{N/mm}^2$

Bearing capacity

The above diagram

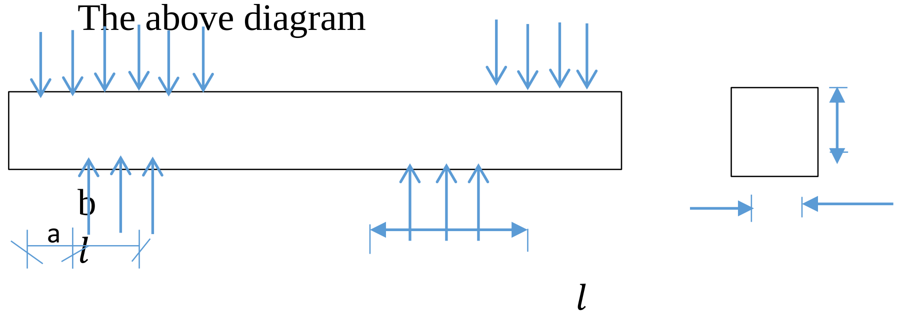


Figure 4.22

From the above diagram

a = overhanging length = O for the structure

l = size of wall = 100mm

h = height of beam = 300mm

b = breadth of beam = 100mm

a = o, L = 100m and h = 100mm

since a<h/3

$$K_{c,90} = \left(2.38 - \frac{l}{250}\right)\left(1 + \frac{h}{12l}\right) = \left(2.38 - \frac{100}{250}\right)\left(1 + \frac{300}{12\times100}\right) = (1.98)(1.25)$$

= 2.475 < 4 OK

$K_c, F_{G90,d} = 2.35X\ 3.42 = 8.06N/mm^2 > \sigma_{c90,d}$ OK

BEAM F

.

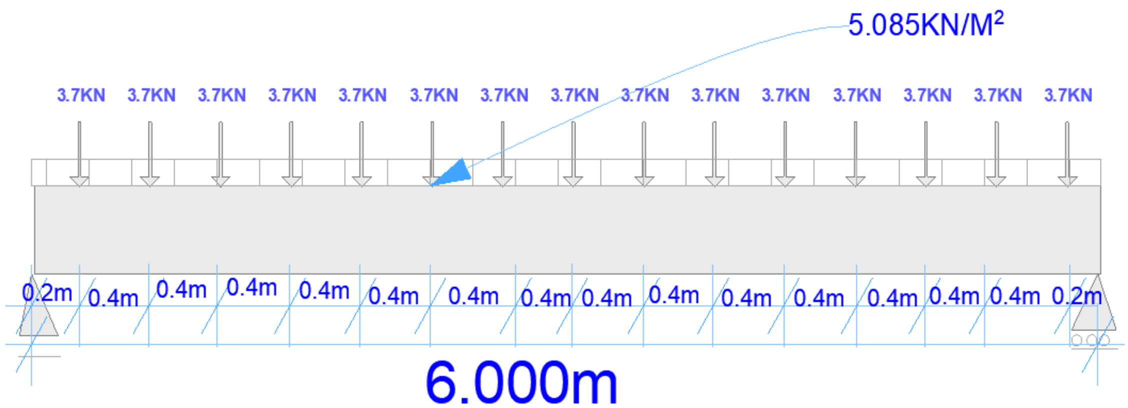


Figure 4.23

Bending (Cl 6.1.6, EC5)

Bending moment

Considering the point loads due to the joists, singly

$Va\ =\ vb\ = 55.855KN$

$$Mmax\ = 3va - (3.7kn\ (9.8)m + 5.08\text{x}\frac{3^2}{2}) = 86.83knm$$

$$Md, y\ = 86.83knm$$

Design bending strength about Y-Y axis

$F_{m,y,d} = K_h.K_{sys.}K_{mod.}\frac{f_{m,k}}{Y_m} = \frac{1.0\times1.1\times0.8\times33.19}{1.3}$

$F_{m,\ yd.} = 22.47N/mm^2$

Bending (Cl 6.1.6, EC 5)

$\frac{\sigma_{m,y,d}}{F_{m,y,d}} + K_m\frac{\sigma_{m,z,d}}{F_{m,z,d}} \leq 1$

$K_m = 0.7$ – for rectangular / square timber section

$\frac{\sigma_{m,y,d}}{22.47} + 0.7 \text{ x } \frac{0}{F_{m,z,d}} \leq 1$

$\sigma_{m,y,d} \leq 22.47 N/mm^2$

$$W_{y,req} \geq \frac{M_{d,y}}{\sigma_{m,y,d}} = \frac{86.83knm \times 10^6}{22.47}$$

$=3864 \times 10^3 mm^3$

From table for classification of Nigerian timber (Aguwa, 2016)

300mm X 300mm Beam would be suitable

$W_y = 4500 \text{X } 10^3 \text{ mm}^3$, $I_y = 675 \text{X } 10^6 \text{ MM}^4$, $A = 90 \text{X } 10^3 \text{ mm}^2$

Deflection (Cl 7.2 , EC5)

A). instatenous deflection due to permanent action, $U_{inst}G$ (from table 2.3, EC 5) Y_G for serviceability limit state = 1.0

Factored permanent load, $G = Y_G G_k = 1.0 \text{ X } 2.1$

$= 2.1 kN/m^2$

$U_{inst,G}$ = bending deflection + shear deflection

$$\frac{5}{384} \times \frac{f_d l^3}{EI} + \frac{12}{5} \times \frac{F_d L}{EA}$$

$U_{inst,G} = \frac{5}{384} \left(\frac{(2.1 \times 10^3 (6 \times 10^3)^3}{12675 \times 675 \times 10^6}\right) + \frac{12}{5} \times \frac{2.1 \times 10^3 \times 6 \times 10^3}{12675 \times 90 \times 10^3}$

= 0.69mm

B) instantaneous deflection due to variable action, $U_{inst}Q$

Y_G for serviceability state = 1.0

Factored variable load, $Q = Y_Q Q_k = 1.0 \text{ X } 1.5 = 1.5 KN/M_2$

Instantaneous deflection due to variable load, $U_{inst}Q$ is given by

$U_{inst}Q$ = bending deflection + sheer deflection

$$\frac{5}{384} \times \frac{f_d l^3}{EI} + \frac{12}{5} \times \frac{F_d L}{EA}$$

$$\frac{5}{384}\left(\frac{1.5\times10^3(4\times10^3)^3}{12675\times225\times10^6}\right) + \frac{12}{5} \times \frac{1.5\times10^3\times4\times10^3}{12675\times30\times10^3}$$

. =0.493mm

(C.) Final deflection due to permanent actions (table 3.2, EC5)

For solid timber timbers subject to service class land

Medium – term loading, $K_{def} = 0.6$

Final deflection due to permanent actions

$U_{fin,}$ G^1 is given by

$U_{fin,G} = U_{inG}(1 + K_{def}) = 0.69X(1 + 0.6) = 1.104mm$

(Cl, 2.4.1,Ec2/cl6) $\Psi_2 = 0.3$

Final deflection due to variable action, $U_{fin,Q,}$ is given by

$U_{fin,Q,} = U_{insQ1I}(1 + \Psi_2 K_{def})$

= 0.493(1 + 0.3 x 0.6) = 0.581mm

D) check for final deflection

Total deflection, $U_{fin} = U_{finG} + U_{finQ} = 1.104 + 0.581$

= 1.685mm

Permissible final deflection assuming the floor support bristle finishes). W_{fin}; is

$W_{fin} = \frac{1}{250}$ X span

1/250 X 6 x 10^3 = 24mm

16mm > 1.685mm OK

Therefore, 300mm X 300mm beam are adequate in deflection

LATERAL BUCKLING (CL 6.3.3, EC 5)

$l_{ef} = 0.9l + 2h = 0.9 \times 6000 + 2 \times 300 = 6000$mm

$\sigma_{m,crit} = \frac{0.78b^2}{hl_{ef}} E_{0,05} = \frac{0.78 \times 300^2}{300 \times 6000} \times 5.4 \times 10^3 = 210.6knm$

.

$\lambda_{rel,m} = \sqrt{\frac{f_{m,k}}{\sigma_{m,crit}}} = \sqrt{\frac{33.17}{210.6}} = 0.39$

since $\lambda_{rel,m} < 0.75, k_{crit} = 1$

$k_{crit}\, f_{m,yd} = k_{crit}(K_h.K_{sys.}K_{mod.} \frac{f_{m,k}}{Y_m}) = 1(\frac{1.0 \times 1.1 \times 0.8 \times 33.19}{1.3})$

= 22.46N/mm

$$\sigma_{m,d} = \frac{M_{y,d}}{bh^2/6} = \frac{8650}{300 \times 300^2/6} = 1.92 \times 10^{-3}$$

since $\sigma_{m,d} \leq k_{crit}\, f_{m,d}$ it is ok

The check is unnecessary as the compressive edge cannot move totally because the Beam is attached to tongue and groove boarding.

SHEAR

Design shear strength is

$F_{v,d} = K_{sys}K_{mod} \frac{f_{y,k}}{\gamma_m} - 1.1 \times 0.8 \times \frac{3.75}{1.3} = 2.54$

Maximum shear force is

$V_{Ed} = va = 55.85 \times 10^3 N$

Design shear stress at Neural axis is

$\tau_d = \frac{1.5\, V_d}{A} = \frac{1.5 \times 55.85 \times 10^3}{30 \times 10^3} = 2.792\text{N/mm}^2 < f_{v,d}$ OK

BEARING

Design compressive stress

U) Design bearing force is

$F_{90,d} = va = 55.85 \times 10^3 N$

100mm

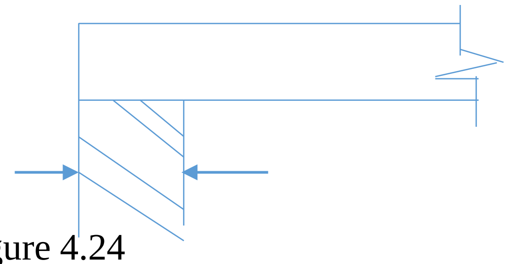


Figure 4.24

Taking the beam to span onto 100mm(assumed) of nearby column as shown above, the bearing stress is given by:

$$\sigma_{c,90,d} = \frac{F_{90,d}}{bl} = \frac{55.85 \times 10^3}{300 \times 300} = 0.62\text{N/mm}^2$$

V) Design compressive strength

Design compressive strength parallel to grain $f_{c,90,\,d}$ is given by

$$F_{c,90,d} = K_{sys} K_{mod} \frac{f_{c,90,k}}{\gamma_m} = 1.1 \times 0.8 \times \frac{5.05}{1.3}$$

$= 3.42\text{N/mm}^2$

Bearing capacity

The above diagram

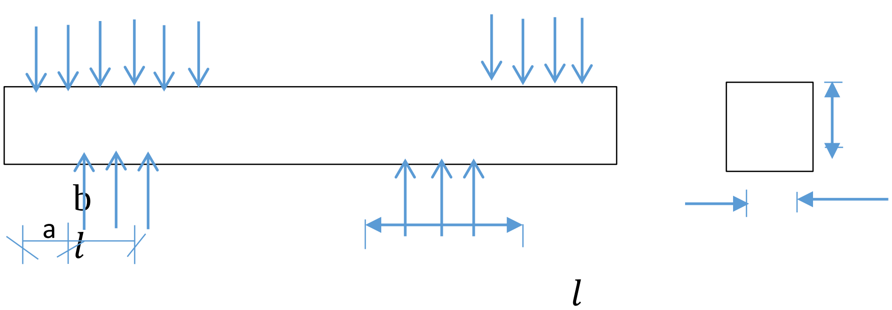


Figure 4.25

From the above diagram

a = overhanging length = O for the structure

l = size of wall = 100mm

h = height of beam = 300mm

b = breadth of beam = 300mm

a = o, L = 100m and h = 100mm

since a<h/3

$K_{c,90}$ = $\left(2.38 - \frac{l}{250}\right)\left(1 + \frac{h}{12l}\right) = \left(2.38 - \frac{100}{250}\right)\left(1 + \frac{300}{12 \times 100}\right) = (1.98)(1.25)$

= 2.475 < 4 OK

$K_c, F_{G90,d}$ = 2.475X 3.42 = 8.46N/mm^2 $> \sigma_{c90,d}$ OK

Table 4.6

4.3 DESIGN OF TIMBER COLUMN

Bending parallel to grain ($F_{M,K}$)	Compression parallel to grain ($F_{C,0,K}$)	Modules of elasticity (min)
33.17	17.91	10302

ROOF -2nd FLOOR

A) COLUMN C-1/F-1/E-8/I-8

Applied axial action

Axial loads (x-x) =17.55KN

Axial loads (y-y) =6.356KN

Self-wt. =0.1 X0.1 X3.15 X7.5=0.238KN X1.35=0.321KN

TOTAL LOAD =17.55+6.356+0.321= 24.233KN

L=3.15m

Effective length= 1.0 X3150=3150mm

B=h=150mm

Area =150 X150= 22500mm^2

$\mathbf{I} = \frac{bh^3}{12} = \frac{150\times150^3}{12} = \mathbf{42.1875}\times10^6\text{mm}^4$

$\mathbf{Z} = \frac{bh^2}{6} = \frac{150\times150^2}{6} = \mathbf{56.25}\times10^4\text{mm}^4$

.

$\boldsymbol{L} = \boldsymbol{l_{yy}} = \boldsymbol{l_{zz}} = \sqrt{\frac{I}{A}} = \sqrt{\frac{42.1875\times10^6}{22500}} = \mathbf{43.3mm}$

Effective slenderness ratio

$\boldsymbol{\lambda} = \frac{L_e}{i} = \frac{3150}{43.3} = \mathbf{72.74}$

$\lambda_{rely} = \lambda_{relz} = \frac{\lambda_y}{\pi}\sqrt{\frac{f_{c,0,k}}{E_{0.05}}} = \frac{72.74}{\pi}\sqrt{\frac{17.91}{10302}} = \mathbf{0.9mm}$

$\lambda_{rely} > 0.3$

BUCKLING (CL. 6.3.2, EC 5)

Design compressive stress

$\boldsymbol{\sigma}_{,c,0,d} = \frac{N}{A} = \frac{23.23\times10^3}{22500} = \mathbf{1.03 N/mm^2}$

Applied bending moment M is

$$M = (23.23 \times 75) = 1742.5KNmm$$

Design Bending strength $f_{m,d}$ **is**

$f_{m,d} = K_h K_{mod.} \frac{f_{m,k}}{Y_m} = \frac{1.0\times0.8\times33.19}{1.3} = 28.67Nmm^{-2}$

Design compression strength $f_{c,0,d}$ **is**

$f_{c,0,d} = K_{mod.} \frac{k_{C,0,k}}{Y_m} = \frac{0.8\times17.91}{1.3} = 11.02Nmm^{-2}$

Design bending stress about the y–y axis, $\boldsymbol{\sigma_{m,y,d}}$

$\boldsymbol{\sigma}_{m,y,d} = \frac{M_Y}{Z_Y} = \frac{17422\text{x}10^3}{562.5\text{x}10^4} = \mathbf{3.097Nmm^{-2}}$

Design bending stress about the z–z axis, $\boldsymbol{\sigma_{m,z,d} = 0}$**.**

$k_y = k_z = 0.5(1 + \beta c(\lambda_{rel,y} - 0.3) + \lambda^2_{rel,y})$

$$k_y = 0.5(1 + 0.2(0.9 - 0.3) + 0.9^2) = 0.965$$

$$k_{cy} = k_{cz} = \frac{1}{k_y + \sqrt{k_y{}^2 - \lambda^2{}_{rely}}} = \frac{1}{0.965 + \sqrt{0.965^2 - 0.9^2}} = 0.76$$

.

$$\frac{\sigma_{c,0,d}}{k_{C,y,}f_{C,0,d}} + \frac{\sigma_{m,y,d}}{f_{m,y,d}} + k_m \frac{\sigma_{m,z,d}}{f_{m,z,d}} \leq 1$$

$$\frac{1.03}{0.76\text{x}11.02} + \frac{3.097}{11.02} + 0.7 \text{ x } \frac{0}{F_{m,z,d}} \leq 1$$

0.136 + 0.281= 0.417 <1 ok

Therefore 150 × 150 column is adequate in buckling.

LATERAL TORSIONAL STABILITY

l_{ef} = l = 3150 mm

$$\sigma_{crit} = \frac{\mathbf{0.78}b^2}{hl_{ef}} E_{min} = \frac{0.78\text{x}150^2}{150\text{x}3150} \text{x}10302 = 382.64$$

$$\lambda_{rely} = \sqrt{\frac{f_{\mathrm{m,k}}}{\boldsymbol{\sigma}_{crit}}} = \sqrt{\frac{33.17}{\mathbf{382.64}}} = \mathbf{0.294}$$

Since $\lambda_{rel,m}$ < 0.75, k_{crit} = 1

Check lateral torsional stability

$$\left(\frac{\sigma_{m,d}}{\boldsymbol{k}_{crit}f_{m,y,d}}\right)^2 + \frac{\sigma_{c,0,d}}{\boldsymbol{k}_{c,z}f_{c,0,d}} \leq 1$$

$$\left(\frac{3.092}{1\text{x}28.67}\right)^2 + \frac{1.03}{0.76\text{x}11.02} = 0.13 < 1 \; ok$$

Hence the column is also adequate in lateral torsional buckling.

B) COLUMN A-2/ J-2 /A-7/J-7

Applied axial action

Axial loads (x-x) =23.75KN

Axial loads (y-y) =10.17KN

Self-wt. =0.15 X0.15 X3.15 X7.5=0.238KN X1.35=0.72KN

TOTAL LOAD =23.75+10.17+0.72= 34.64KN

Geometric properties

L=3.15m

Effective length= 1.0 X3150=3150mm

B=h=150mm

Area =150 X150= 22500mm^2

$\mathbf{I} = \frac{bh^3}{12} = \frac{150\times150^3}{12} = \mathbf{42.1875}\times10^6\text{mm}^4$

$\mathbf{W} = \frac{bh^2}{6} = \frac{150\times150^2}{6} = \mathbf{56.25}\times10^4\text{mm}^4$

$L = l_{yy} = l_{zz} = \sqrt{\frac{I}{A}} = \sqrt{\frac{42.1875\times10^6}{22500}}$ **=43.3mm**

Effective slenderness ratio

$\lambda = \frac{L_e}{i} = \frac{3150}{43.3} = \mathbf{72.74}$

$\lambda_{rely} = \lambda_{relz} = \frac{\lambda_y}{\pi}\sqrt{\frac{f_{c,0,k}}{E_{0.05}}} = \frac{72.74}{\pi}\sqrt{\frac{17.91}{10302}}$ **=0.9mm**

$\lambda_{rely} > 0.3$

BUCKLING (CL. 6.3.2, EC 5)

Design compressive stress

$$\boldsymbol{\sigma}_{,c,0,d} = \frac{N}{A} = \frac{34.64\times10^3}{22.5\times10^3} = \mathbf{1.53}\boldsymbol{N/mm^2}$$

Applied bending moment M is

$$M = (34.64 \times 75) = 2598KNmm$$

Design Bending strength $F_{m,d}$ **is**

$$f_{m,d} = K_h\, K_{mod.} \frac{f_{m,k}}{Y_m} = \frac{1.0\times0.8\times33.19}{1.3} = 28.67Nmm^{-2}$$

Design compression strength $f_{c,0,d}$ **is**

$$f_{c,0,d} = K_{mod.} \frac{k_{C,0,k}}{Y_m} = \frac{0.8\times17.91}{1.3} = 11.02Nmm^{-2}$$

Design bending stress about the y–y axis, $\boldsymbol{\sigma_{m,y,d}}$

$$\boldsymbol{\sigma}_{m,y,d} = \frac{M_Y}{Z_Y} = \frac{25980 \times 10^3}{562.5\times10^4} = \mathbf{4.62}\boldsymbol{Nmm^{-2}}$$

Design bending stress about the z–z axis, $\boldsymbol{\sigma_{m,z,d}}$ **= 0.**

$$k_y = k_z = 0.5(1 + \beta c(\lambda_{rel,y} - 0.3) + \lambda^2_{rel,y})$$

$$k_y = 0.5(1 + 0.2(0.9 - 0.3) + 0.9^2) = 0.965$$

$$k_{cy} = k_{cz} = \frac{1}{k_y+\sqrt{{k_y}^2-\lambda^2{}_{rely}}} = \frac{1}{0.965+\sqrt{0.965^2-0.9^2}} = 0.76$$

$$\frac{\sigma_{C,0,d}}{k_{C,y,}f_{C,0,d}} + \frac{\sigma_{m,y,d}}{f_{m,y,d}} + k_m \frac{\sigma_{m,z,d}}{f_{m,z,d}} \leq 1$$

$$\frac{1.53}{0.76\times11.02} + \frac{4.62}{28.67} + 0.7 \times \frac{0}{F_{m,z,d}} \leq 1$$

0.182+ 0.161= 0.343 <1 ok

Therefore 150 × 150 column is adequate in buckling.

LATERAL TORSIONAL STABILITY

$\boldsymbol{l_{ef} = l}$ **= 3150 mm**

$\boldsymbol{\sigma}_{crit} = \frac{\mathbf{0.78}b^2}{hl_{ef}} E_{min} = \frac{0.78\text{x}150^2}{150\text{x}3150} \text{x}10302 = 382.64$

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$\lambda_{rely} = \sqrt{\frac{f_{\mathrm{m,k}}}{\boldsymbol{\sigma}_{crit}}} = \sqrt{\frac{33.17}{\mathbf{382.64}}} = \mathbf{0.294}$

Since $\lambda_{rel,m}$ < 0.75, k_{crit} = 1

Check lateral torsional stability

$$\left(\frac{\sigma_{m,d}}{\boldsymbol{k}_{crit} f_{m,y,d}}\right)^2 + \frac{\sigma_{c,0,d}}{\boldsymbol{k}_{c,z} f_{c,0,d}} \leq 1$$

$$\left(\frac{4.62}{1\text{x}28.67}\right)^2 + \frac{1.03}{0.76\text{x}11.02} = 0.148 < 1\ ok$$

Hence the column is also adequate in lateral torsional buckling.

C) COLUMN B-2/ I-2

Applied axial action

Axial loads (x-x) =23.75KN

Axial loads (y-y) =6.37KN + 23.75KN = 30.12KN

Self-wt. =0.15 X0.15 X3.15 X7.5=0.238KN X1.35=0.72KN

TOTAL LOAD =23.75+6.37+23.75+0.72= 53.87KN

Geometric properties

L=3.15m

Effective length= 1.0 X3150=3150mm

B=h=150mm

Area =150 X150= 22500mm^2

$\mathbf{I} = \frac{bh^3}{12} = \frac{150\times150^3}{12} = \mathbf{42.1875}\times10^6\text{mm}^4$

$\mathbf{W} = \frac{bh^2}{6} = \frac{150\times150^2}{6} = \mathbf{56.25}\times10^4\text{mm}^4$

$L = l_{yy} = l_{zz} = \sqrt{\frac{I}{A}} = \sqrt{\frac{42.1875\times10^6}{22500}} = \mathbf{43.3mm}$

Effective slenderness ratio

$\lambda = \frac{L_e}{i} = \frac{3150}{43.3} = \mathbf{72.74}$

$\lambda_{rely} = \lambda_{relz} = \frac{\lambda_y}{\pi}\sqrt{\frac{f_{c,0,k}}{E_{0.05}}} = \frac{72.74}{\pi}\sqrt{\frac{17.91}{10302}} = \mathbf{0.9mm}$

$\lambda_{rely} > 0.3$

BUCKLING (CL. 6.3.2, EC 5)

Design compressive stress

$\sigma_{,c,0,d} = \frac{N}{A} = \frac{53.87\times10^3}{22.5\times10^3} = \mathbf{2.39 N/mm^2}$

Applied bending moment M is

$$M = (53.87 \times 75) = 4040.22KNmm$$

Design Bending strength $F_{m,d}$ **is**

$f_{m,d} = K_h K_{mod.} \frac{f_{m,k}}{Y_m} = \frac{1.0\times0.8\times33.19}{1.3} = 28.67Nmm^{-2}$

Design compression strength $f_{c,0,d}$ **is**

$f_{c,0,d} = K_{mod.} \frac{k_{C,0,k}}{Y_m} = \frac{0.8\times17.91}{1.3} = 11.02Nmm^{-2}$

Design bending stress about the y–y axis, $\boldsymbol{\sigma_{m,y,d}}$

$\boldsymbol{\sigma}_{m,y,d} = \frac{M_Y}{Z_Y} = \frac{40402.2 \text{ x } 10^3}{562.5\text{x}10^4} = \mathbf{7.18 Nmm^{-2}}$

Design bending stress about the z–z axis, $\sigma_{m,z,d}$ = 0.

$k_y = k_z = 0.5(1 + \beta c(\lambda_{rel,y} - 0.3) + \lambda^2_{rel,y})$

$k_y = 0.5(1 + 0.2(0.9 - 0.3) + 0.9^2) = 0.965$

$$k_{cy} = k_{cz} = \frac{1}{k_y + \sqrt{{k_y}^2 - \lambda^2{}_{rely}}} = \frac{1}{0.965 + \sqrt{0.965^2 - 0.9^2}} = 0.76$$

$$\frac{\sigma_{c,0,d}}{k_{C,y,}f_{C,0,d}} + \frac{\sigma_{m,y,d}}{f_{m,y,d}} + k_m \frac{\sigma_{m,z,d}}{f_{m,z,d}} \leq 1$$

$$\frac{2.39}{0.76\text{x}11.02} + \frac{7.18}{28.67} + 0.7 \text{ x } \frac{0}{F_{m,z,d}} \leq 1$$

0.28+ 0.25= 0.53 <1 ok

Therefore 150 × 150 column is adequate in buckling.

LATERAL TORSIONAL STABILITY

***l*$_{ef}$ = *l* = 3150 mm**

$$\boldsymbol{\sigma}_{crit} = \frac{\mathbf{0.78}b^2}{hl_{ef}} E_{min} = \frac{0.78\text{x}150^2}{150\text{x}3150} \text{x}10302 = 382.64$$

$$\lambda_{rely} = \sqrt{\frac{f_{\text{m,k}}}{\boldsymbol{\sigma}_{crit}}} = \sqrt{\frac{33.17}{\mathbf{382.64}}} = \mathbf{0.294}$$

Since $\lambda_{rel,m}$ < 0.75, k_{crit} = 1

Check lateral torsional stability

$$\left(\frac{\sigma_{m,d}}{\boldsymbol{k}_{crit} f_{m,y,d}}\right)^2 + \frac{\sigma_{c,0,d}}{\boldsymbol{k}_{c,z} f_{c,0,d}} \leq 1$$

$$\left(\frac{7.18}{1x28.67}\right)^2 + \frac{2.39}{0.76x11.02} = 0.348 < 1\ ok$$

Hence the column is also adequate in lateral torsional buckling.

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D) COLUMN C-2/ F-2

Applied axial action

Axial loads (x-x) =23.75KN

Axial loads (y-y) =6.37KN + 23.75KN = 30.12KN

Self-wt. =0.15 X0.15 X3.15 X7.5=0.238KN X1.35=0.72KN

TOTAL LOAD =23.75+6.37+23.75+0.72= 53.87KN

Geometric properties

L=3.15m

Effective length= 1.0 X3150=3150mm

B=h=150mm

Area =150 X150= 22500mm^2

$$\mathbf{I} = \frac{bh^3}{12} = \frac{150\times150^3}{12} = \mathbf{42.1875}\times10^6\text{mm}^4$$

$$\mathbf{W} = \frac{bh^2}{6} = \frac{150\times150^2}{6} = \mathbf{56.25}\times10^4\text{mm}^4$$

$$L = l_{yy} = l_{zz} = \sqrt{\frac{I}{A}} = \sqrt{\frac{42.1875\times10^6}{22500}} = \mathbf{43.3mm}$$

Effective slenderness ratio

$$\lambda = \frac{L_e}{i} = \frac{3150}{43.3} = \mathbf{72.74}$$

$$\lambda_{rely} = \lambda_{relz} = \frac{\lambda_y}{\pi}\sqrt{\frac{f_{c,0,k}}{E_{0.05}}} = \frac{72.74}{\pi}\sqrt{\frac{17.91}{10302}} = \mathbf{0.9mm}$$

$$\lambda_{rely} > 0.3$$

BUCKLING (CL. 6.3.2, EC 5)

Design compressive stress

$\boldsymbol{\sigma}_{,c,0,d} = \frac{N}{A} = \frac{53.87\times10^3}{22.5\times10^3} = \mathbf{2.39 N/mm^2}$

Applied bending moment M is

$$M = (53.87 \times 75) = 4040.22 KNmm$$

Design Bending strength $F_{m,d}$ **is**

$f_{m,d} = K_h K_{mod.} \frac{f_{m,k}}{Y_m} = \frac{1.0\times0.8\times33.19}{1.3} = 28.67 Nmm^{-2}$

Design compression strength $f_{c,0,d}$ **is**

$f_{c,0,d} = K_{mod.} \frac{k_{C,0,k}}{Y_m} = \frac{0.8\times17.91}{1.3} = 11.02 Nmm^{-2}$

Design bending stress about the y–y axis, $\boldsymbol{\sigma_{m,y,d}}$

$\boldsymbol{\sigma}_{m,y,d} = \frac{M_Y}{Z_Y} = \frac{40402.2 \text{ x } 10^3}{562.5\text{x}10^4} = \mathbf{7.18 Nmm^{-2}}$

Design bending stress about the z–z axis, $\boldsymbol{\sigma_{m,z,d} = 0}$**.**

$k_y = k_z = 0.5(1 + \beta c(\lambda_{rel,y} - 0.3) + \lambda^2_{rel,y})$

$k_y = 0.5(1 + 0.2(0.9 - 0.3) + 0.9^2) = 0.965$

$k_{cy} = k_{cz} = \frac{1}{k_y + \sqrt{k_y{}^2 - \lambda^2{}_{rely}}} = \frac{1}{O.965 + \sqrt{0.965^2 - 0.9^2}} = 0.76$

$$\frac{\sigma_{c,0,d}}{k_{C,y,} f_{C,0,d}} + \frac{\sigma_{m,y,d}}{f_{m,y,d}} + k_m \frac{\sigma_{m,z,d}}{f_{m,z,d}} \leq 1$$

$$\frac{2.39}{0.76x11.02} + \frac{7.18}{28.67} + 0.7 \text{ x } \frac{0}{F_{m,z,d}} \leq 1$$

0.28+ 0.25= 0.53 <1 ok

Therefore 150 × 150 column is adequate in buckling.

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LATERAL TORSIONAL STABILITY

***l_{ef}* = *l* = 3150 mm**

$$\boldsymbol{\sigma}_{crit} = \frac{\mathbf{0.78}b^2}{hl_{ef}} E_{min} = \frac{0.78x150^2}{150x3150} \text{x}10302 = 382.64$$

$$\lambda_{rely} = \sqrt{\frac{f_{\mathrm{m,k}}}{\boldsymbol{\sigma}_{\boldsymbol{crit}}}} = \sqrt{\frac{33.17}{\mathbf{382.64}}} = \mathbf{0.294}$$

Since $\lambda_{rel,m}$ < 0.75, k_{crit} = 1

Check lateral torsional stability

$$\left(\frac{\sigma_{m,d}}{\boldsymbol{k}_{crit} f_{m,y,d}}\right)^2 + \frac{\sigma_{c,0,d}}{\boldsymbol{k}_{c,z} f_{c,0,d}} \leq 1$$

$$\left(\frac{7.18}{1x28.67}\right)^2 + \frac{2.39}{0.76x11.02} = 0.348 < 1 \; ok$$

Hence the column is also adequate in lateral torsional buckling.

2^{nd} – 1^{st} FLOOR

A) COLUMN C-1/F-1/E-8/I-8

Applied axial action

Loading from above = 24.233KN

Axial loads (x-x) =17.55KN

Axial loads (y-y) =6.356KN

Self-wt. =0.2 X0.2 X3.15 X7.5=0.238KN X1.35=0.321KN

. TOTAL LOAD =17.55+6.356+0.321+24.233= 48.47KN

Geometric properties

L=3.15m

Effective length= 1.0 X3150=3150mm

B=h=150mm

Area =150 X150= 22500mm^2

$\mathbf{I} = \frac{bh^3}{12} = \frac{150 \times 150^3}{12} = \mathbf{42.1875} \times 10^6 \mathrm{mm}^4$

$\mathbf{Z} = \frac{bh^2}{6} = \frac{150 \times 150^2}{6} = \mathbf{56.25} \times 10^4 \mathrm{mm}^4$

$L = l_{yy} = l_{zz} = \sqrt{\frac{I}{A}} = \sqrt{\frac{42.1875 \times 10^6}{22500}} = \mathbf{43.3mm}$

Effective slenderness ratio

$\lambda = \frac{L_e}{i} = \frac{3150}{43.3} = \mathbf{72.74}$

$\lambda_{rely} = \lambda_{relz} = \frac{\lambda_y}{\pi} \sqrt{\frac{f_{c,0,k}}{E_{0.05}}} = \frac{72.74}{\pi} \sqrt{\frac{17.91}{10302}} = \mathbf{0.9mm}$

$\lambda_{rely} > 0.3$

BUCKLING (CL. 6.3.2, EC 5)

Design compressive stress

$\sigma_{,c,0,d} = \frac{N}{A} = \frac{48.466 \times 10^3}{22500} = \mathbf{2.15 N/mm^2}$

Applied bending moment M is

$$M = (48.466 \times 75) = 3634.95KNmm$$

Design Bending strength $f_{m,d}$ **is**

$f_{m,d} = K_h\, K_{mod.} \frac{f_{m,k}}{Y_m} = \frac{1.0\times0.8\times33.19}{1.3} = 28.67Nmm^{-2}$

Design compression strength $f_{c,0,d}$ **is**

$f_{c,0,d} = K_{mod.} \frac{k_{C,0,k}}{Y_m} = \frac{0.8\times17.91}{1.3} = 11.02Nmm^{-2}$

Design bending stress about the y–y axis, $\boldsymbol{\sigma_{m,y,d}}$

$\boldsymbol{\sigma_{m,y,d}} = \frac{\boldsymbol{M_Y}}{\boldsymbol{Z_Y}} = \frac{36349.5\text{x}\,10^3}{\mathbf{562.5}\text{x}10^4} = \mathbf{6.462Nmm^{-2}}$

Design bending stress about the z–z axis, $\boldsymbol{\sigma_{m,z,d}} = \mathbf{0}$**.**

$k_y = k_z = 0.5(1 + \beta c(\lambda_{rel,y} - 0.3) + \lambda^2_{rel,y})$

$k_y = 0.5(1 + 0.2(0.9 - 0.3) + 0.9^2) = 0.965$

$k_{cy} = k_{cz} = \frac{1}{k_y+\sqrt{{k_y}^2-\lambda^2{}_{rely}}} = \frac{1}{0.965+\sqrt{0.965^2-0.9^2}} = 0.76$

$$\frac{\sigma_{C,0,d}}{k_{C,y,}f_{C,0,d}} + \frac{\sigma_{m,y,d}}{f_{m,y,d}} + k_m \frac{\sigma_{m,z,d}}{f_{m,z,d}} \leq 1$$

$$\frac{2.15}{0.76\text{x}11.02} + \frac{6.462}{11.02} + 0.7 \text{ x } \frac{0}{F_{m,z,d}} \leq 1$$

$$0.257 + 0.586 = 0.843 < 1 \text{ ok}$$

Therefore 150 × 150 column is adequate in buckling.

LATERAL TORSIONAL STABILITY

$\boldsymbol{l_{ef} = l}$ **= 3150 mm**

$\boldsymbol{\sigma_{crit}} = \frac{\mathbf{0.78}b^2}{hl_{ef}} E_{min} = \frac{0.78\text{x}150^2}{150\text{x}3150} \text{x}10302 = 382.64$

$$\lambda_{rely} = \sqrt{\frac{f_{m,k}}{\sigma_{crit}}} = \sqrt{\frac{33.17}{382.64}} = \mathbf{0.294}$$

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Since $\lambda_{rel,m} < 0.75$, $k_{crit} = 1$

Check lateral torsional stability

$$\left(\frac{\sigma_{m,d}}{\boldsymbol{k}_{crit} f_{m,y,d}}\right)^2 + \frac{\sigma_{c,0,d}}{\boldsymbol{k}_{c,z} f_{c,0,d}} \leq 1$$

$$\left(\frac{6.462}{1\text{x}28.67}\right)^2 + \frac{2.15}{0.76\text{x}11.02} = 0.30 < 1\ ok$$

Hence the column is also adequate in lateral torsional buckling.

B) COLUMN A-2/ J-2 /A-7/J-7

Applied axial action

Loading from above = 34.64KN

Axial loads (x-x) =23.75KN

Axial loads (y-y) =10.17KN

Self-wt. =0.15 X0.15 X3.15 X7.5=0.238KN X1.35=0.72KN

TOTAL LOAD =23.75+10.17+0.72+34.64= 69.28KN

Geometric properties

L=3.15m

Effective length= 1.0 X3150=3150mm

B=h=150mm

Area =150 X150= 22500mm^2

$$\mathbf{I} = \frac{bh^3}{12} = \frac{150\times150^3}{12} = \mathbf{42.1875}\times10^6\text{mm}^4$$

$$\mathbf{W} = \frac{bh^2}{6} = \frac{150\times150^2}{6} = \mathbf{56.25}\times10^4\text{mm}^4$$

$$L = l_{yy} = l_{zz} = \sqrt{\frac{I}{A}} = \sqrt{\frac{42.1875\times10^6}{22500}} = \mathbf{43.3mm}$$

Effective slenderness ratio

$$\lambda = \frac{L_e}{i} = \frac{3150}{43.3} = \mathbf{72.74}$$

$$\lambda_{rely} = \lambda_{relz} = \frac{\lambda_y}{\pi}\sqrt{\frac{f_{c,0,k}}{E_{0.05}}} = \frac{72.74}{\pi}\sqrt{\frac{17.91}{10302}} = \mathbf{0.9mm}$$

$$\lambda_{rely} > 0.3$$

BUCKLING (CL. 6.3.2, EC 5)

Design compressive stress

$$\sigma_{,c,0,d} = \frac{N}{A} = \frac{69.28\times10^3}{22.5\times10^3} = \mathbf{3.079N/mm^2}$$

Applied bending moment M is

$$M = (69.28\times75) = 5196KNmm$$

Design Bending strength $F_{m,d}$ **is**

$$f_{m,d} = K_h K_{mod.}\frac{f_{m,k}}{Y_m} = \frac{1.0\times0.8\times33.19}{1.3} = 28.67Nmm^{-2}$$

Design compression strength $f_{c,0,d}$ **is**

$$f_{c,0,d} = K_{mod.}\frac{k_{C,0,k}}{Y_m} = \frac{0.8\times17.91}{1.3} = 11.02Nmm^{-2}$$

Design bending stress about the y–y axis, $\sigma_{m,y,d}$

$$\sigma_{m,y,d} = \frac{M_Y}{Z_Y} = \frac{51960\text{ x }10^3}{562.5\text{x}10^4} = \mathbf{9.237Nmm^{-2}}$$

Design bending stress about the z–z axis, $\sigma_{m,z,d}$ = 0.

$k_y = k_z = 0.5(1 + \beta c(\lambda_{rel,y} - 0.3) + \lambda^2_{rel,y})$

$$k_y = 0.5(1 + 0.2(0.9 - 0.3) + 0.9^2) = 0.965$$

.

$$k_{cy} = k_{cz} = \frac{1}{k_y + \sqrt{k_y{}^2 - \lambda^2{}_{rely}}} = \frac{1}{0.965 + \sqrt{0.965^2 - 0.9^2}} = 0.76$$

$$\frac{\sigma_{c,0,d}}{k_{C,y,}f_{C,0,d}} + \frac{\sigma_{m,y,d}}{f_{m,y,d}} + k_m \frac{\sigma_{m,z,d}}{f_{m,z,d}} \leq 1$$

$$\frac{3.079}{0.76\text{x}11.02} + \frac{9.237}{28.67} + 0.7 \text{ x } \frac{0}{F_{m,z,d}} \leq 1$$

$$0.367 + 0.322 = 0.698 < 1 \text{ ok}$$

Therefore 150 × 150 column is adequate in buckling.

LATERAL TORSIONAL STABILITY

$l_{ef} = l$ = 3150 mm

$$\boldsymbol{\sigma}_{crit} = \frac{\mathbf{0.78}b^2}{hl_{ef}} E_{min} = \frac{0.78\text{x}150^2}{150\text{x}3150} \text{ x}10302 = 382.64$$

$$\lambda_{rely} = \sqrt{\frac{f_{\text{m,k}}}{\boldsymbol{\sigma}_{crit}}} = \sqrt{\frac{33.17}{\mathbf{382.64}}} = \mathbf{0.294}$$

Since $\lambda_{rel,m}$ < 0.75, k_{crit} = 1

Check lateral torsional stability

$$\left(\frac{\sigma_{m,d}}{\boldsymbol{k}_{crit} f_{m,y,d}}\right)^2 + \frac{\sigma_{c,0,d}}{\boldsymbol{k}_{c,z} f_{c,0,d}} \leq 1$$

$$\left(\frac{9.237}{1x28.67}\right)^2 + \frac{3.079}{0.76x11.02} = 0.47 < 1\ ok$$

Hence the column is also adequate in lateral torsional buckling.

.

C) COLUMN B-2/ I-2

Applied axial action

Loading from above = 53.87KN

Axial loads (x-x) =23.75KN

Axial loads (y-y) =6.37KN + 23.75KN = 30.12KN

Self-wt. =0.2 X0.2 X3.15 X7.5=0.238KN X1.35=1.275KN

TOTAL LOAD =23.75+6.37+23.75+1.275 +53.87KN = 109.015KN

Geometric properties

L=3.15m

Effective length= 1.0 X3150=3150mm

B=h=200mm

Area =200 X200= 40000mm^2

$$\mathbf{I} = \frac{bh^3}{12} = \frac{200\times200^3}{12} = \mathbf{133.33}\times10^6\text{mm}^4$$

$$\mathbf{W} = \frac{bh^2}{6} = \frac{200\times200^2}{6} = \mathbf{133.33}\times10^4\text{mm}^4$$

$$\boldsymbol{L} = \boldsymbol{l_{yy}} = \boldsymbol{l_{zz}} = \sqrt{\frac{I}{A}} = \sqrt{\frac{133.33\times10^6}{22500}} = \mathbf{76.97mm}$$

Effective slenderness ratio

$$\boldsymbol{\lambda} = \frac{L_e}{i} = \frac{3150}{133.33} = \mathbf{23.625}$$

$$\lambda_{rely} = \lambda_{relz} = \frac{\lambda_y}{\pi}\sqrt{\frac{f_{c,0,k}}{E_{0.05}}} = \frac{23.625}{\pi}\sqrt{\frac{17.91}{10302}} = \mathbf{0.313mm}$$

$\lambda_{rely} > 0.3$

BUCKLING (CL. 6.3.2, EC 5)

Design compressive stress

$$\boldsymbol{\sigma_{,c,0,d} = \frac{N}{A} = \frac{109.015\times10^3}{40\times10^3} = 2.725 N/mm^2}$$

Applied bending moment M is

$$M = (109.015 \times 100) = 10901.5 KNmm$$

Design Bending strength $F_{m,d}$ **is**

$$f_{m,d} = K_h\, K_{mod.} \frac{f_{m,k}}{Y_m} = \frac{1.0\times0.8\times33.19}{1.3} = 28.67 Nmm^{-2}$$

Design compression strength $f_{c,0,d}$ **is**

$$f_{c,0,d} = K_{mod.} \frac{k_{C,0,k}}{Y_m} = \frac{0.8\times17.91}{1.3} = 11.02 Nmm^{-2}$$

Design bending stress about the y–y axis, $\boldsymbol{\sigma_{m,y,d}}$

$$\boldsymbol{\sigma_{m,y,d} = \frac{M_Y}{Z_Y} = \frac{109015 \text{ x } 10^3}{133.33\text{x}10^4} = 8.176 Nmm^{-2}}$$

Design bending stress about the z–z axis, $\boldsymbol{\sigma_{m,z,d} = 0}$**.**

$$k_y = k_z = 0.5(1 + \beta c(\lambda_{rel,y} - 0.3) + \lambda^2_{rel,y})$$

$$k_y = 0.5(1 + 0.2(0.9 - 0.3) + 0.9^2) = 0.965$$

$$k_{cy} = k_{cz} = \frac{1}{k_y + \sqrt{{k_y}^2 - \lambda^2_{rely}}} = \frac{1}{0.965 + \sqrt{0.965^2 - 0.9^2}} = 0.76$$

$$\frac{\sigma_{c,0,d}}{k_{C,y,}f_{C,0,d}} + \frac{\sigma_{m,y,d}}{f_{m,y,d}} + k_m \frac{\sigma_{m,z,d}}{f_{m,z,d}} \leq 1$$

$$\frac{2.724}{0.76\text{x}11.02} + \frac{8.176}{28.67} + 0.7 \text{ x } \frac{0}{F_{m,z,d}} \leq 1$$

0.325+ 0.285= 0.610 <1 ok

Therefore 200 × 200 column is adequate in buckling.

LATERAL TORSIONAL STABILITY

***l_{ef}* = *l* = 3150 mm**

$$\boldsymbol{\sigma}_{crit} = \frac{\mathbf{0.78}b^2}{hl_{ef}} E_{min} = \frac{0.78\text{x}200^2}{200\text{x}3150} \text{ x}10302 = 510.19$$

$$\lambda_{rely} = \sqrt{\frac{f_{\text{m,k}}}{\boldsymbol{\sigma}_{crit}}} = \sqrt{\frac{33.17}{\mathbf{510.19}}} = \mathbf{0.254}$$

Since $\lambda_{rel,m}$ < 0.75, k_{crit} = 1

Check lateral torsional stability

$$\left(\frac{\sigma_{m,d}}{\boldsymbol{k}_{crit}f_{m,y,d}}\right)^2 + \frac{\sigma_{c,0,d}}{\boldsymbol{k}_{c,z}f_{c,0,d}} \leq 1$$

$$\left(\frac{8.176}{1\text{x}28.67}\right)^2 + \frac{2.39}{0.76\text{x}11.02} = 0.366 < 1\ ok$$

Hence the column is also adequate in lateral torsional buckling.

D)COLUMN C-2/ F-2

Applied axial action

Load from above = 53.87KN

Axial loads (x-x) =23.75KN

Axial loads (y-y) =6.37KN + 23.75KN = 30.12KN

Self-wt. =0.20 X0.20 X3.15 X7.5=0.238KN X1.35=1.275KN

TOTAL LOAD =23.75+6.37+23.75+1.275+ 53.87KN = 109.015KN

Geometric properties

L=3.15m

Effective length= 1.0 X3150=3150mm

B=h=200mm

Area =200 X200= 40000mm^2

$$\mathbf{I} = \frac{bh^3}{12} = \frac{200\times200^3}{12} = \mathbf{133.33}\times10^6\text{mm}^4$$

$$\mathbf{W} = \frac{bh^2}{6} = \frac{200\times200^2}{6} = \mathbf{133.33}\times10^4\text{mm}^4$$

$$L = l_{yy} = l_{zz} = \sqrt{\frac{I}{A}} = \sqrt{\frac{133.33\times10^6}{22500}} = \mathbf{76.97mm}$$

Effective slenderness ratio

$$\lambda = \frac{L_e}{i} = \frac{3150}{133.33} = \mathbf{23.625}$$

$$\lambda_{rely} = \lambda_{relz} = \frac{\lambda_y}{\pi}\sqrt{\frac{f_{c,0,k}}{E_{0.05}}} = \frac{23.625}{\pi}\sqrt{\frac{17.91}{10302}} = \mathbf{0.313mm}$$

$$\lambda_{rely} > 0.3$$

BUCKLING (CL. 6.3.2, EC 5)

Design compressive stress

$$\sigma_{,c,0,d} = \frac{N}{A} = \frac{109.017 \times 10^3}{40. \times 10^3} = 2.73N/mm^2$$

Applied bending moment M is

$$M = (109.017 \times 100) = 10901.7KNmm$$

Design Bending strength $F_{m,d}$ **is**

$$f_{m,d} = K_h K_{mod.} \frac{f_{m,k}}{Y_m} = \frac{1.0 \times 0.8 \times 33.19}{1.3} = 28.67Nmm^{-2}$$

Design compression strength $f_{c,0,d}$ **is**

$$f_{c,0,d} = K_{mod.} \frac{k_{C,0,k}}{Y_m} = \frac{0.8 \times 17.91}{1.3} = 11.02Nmm^{-2}$$

Design bending stress about the y–y axis, $\sigma_{m,y,d}$

$$\sigma_{m,y,d} = \frac{M_Y}{Z_Y} = \frac{10901.7 \text{ x } 10^3}{133.33 \text{x} 10^4} = 8.176Nmm^{-2}$$

Design bending stress about the z–z axis, $\sigma_{m,z,d} = 0$**.**

$$k_y = k_z = 0.5(1 + \beta c(\lambda_{rel,y} - 0.3) + \lambda^2_{rel,y})$$

$$k_y = 0.5(1 + 0.2(0.313 - 0.3) + 0.313^2) = 0.55$$

$$k_{cy} = k_{cz} = \frac{1}{k_y + \sqrt{{k_y}^2 - \lambda^2{}_{rely}}} = \frac{1}{0.55 + \sqrt{0.55^2 - 0.313^2}} = 0.845$$

$$\frac{\sigma_{C,0,d}}{k_{C,y,}f_{C,0,d}} + \frac{\sigma_{m,y,d}}{f_{m,y,d}} + k_m \frac{\sigma_{m,z,d}}{f_{m,z,d}} \leq 1$$

$$\frac{2.73}{0.848 \text{x} 11.02} + \frac{8.176}{28.67} + 0.7 \text{ x } \frac{0}{F_{m,z,d}} \leq 1$$

0.292+ 0.285= 0.577 <1 ok

Therefore 200 × 200 column is adequate in buckling.

LATERAL TORSIONAL STABILITY

l_{ef} = l = 3150 mm

$\sigma_{crit} = \frac{0.78b^2}{hl_{ef}} E_{min} = \frac{0.78x200^2}{200x3150} x10302 = 510.19$

.

$\lambda_{rely} = \sqrt{\frac{f_{m,k}}{\sigma_{crit}}} = \sqrt{\frac{33.17}{510.19}} = 0.254$

Since $\lambda_{rel,m} < 0.75$, $k_{crit} = 1$

Check lateral torsional stability

$$\left(\frac{\sigma_{m,d}}{k_{crit} f_{m,y,d}}\right)^2 + \frac{\sigma_{c,0,d}}{k_{c,z} f_{c,0,d}} \leq 1$$

$$\left(\frac{8.176}{1x28.67}\right)^2 + \frac{2.73}{0.846x11.02} = 0.374 < 1 \ ok$$

Hence the column is also adequate in lateral torsional buckling.

1ST TO GROUND FLOOR

A) COLUMN C-1/F-1/E-8/I-8

.

Applied axial action

Loading from above = 48.47KN

Axial loads (x-x) =17.55KN

Axial loads (y-y) =6.356KN

Self-wt. =0.3 X0.3 X3.15 X7.5=0.238KN X1.35=2.87KN

TOTAL LOAD =17.55+6.356+2.87+24.233+ 48.47KN=99.479KN

Geometric properties

L=3.15m

Effective length= 1.0 X3150=3150mm

B=h=300mm

Area =300 X300= 90000mm^2

$$\mathbf{I} = \frac{bh^3}{12} = \frac{300\times300^3}{12} = \mathbf{675}\times10^6\text{mm}^4$$

$$\mathbf{Z} = \frac{bh^2}{6} = \frac{300\times300^2}{6} = \mathbf{450}\times10^4\text{mm}^4$$

$$L = l_{yy} = l_{zz} = \sqrt{\frac{I}{A}} = \sqrt{\frac{675\times10^6}{90000}} = \mathbf{86.6mm}$$

Effective slenderness ratio

$$\lambda = \frac{L_e}{i} = \frac{3150}{675} = \mathbf{4.66}$$

$$\lambda_{rely} = \lambda_{relz} = \frac{\lambda_y}{\pi}\sqrt{\frac{f_{c,0,k}}{E_{0.05}}} = \frac{4.66}{\pi}\sqrt{\frac{17.91}{10302}} = 0.06\text{mm}$$

BUCKLING (CL. 6.3.2, EC 5)

Design compressive stress

$$\sigma_{,c,0,d} = \frac{N}{A} = \frac{99.479\times10^3}{22500} = 4.421 N/mm^2$$

Applied bending moment M is

$$M = (99.479 \times 100) = 9947.9KNmm$$

Design Bending strength $f_{m,d}$ **is**

$$f_{m,d} = K_h K_{mod.} \frac{f_{m,k}}{Y_m} = \frac{1.0\times0.8\times33.19}{1.3} = 28.67Nmm^{-2}$$

Design compression strength $f_{c,0,d}$ **is**

$$f_{c,0,d} = K_{mod.} \frac{k_{C,0,k}}{Y_m} = \frac{0.8\times17.91}{1.3} = 11.02Nmm^{-2}$$

Design bending stress about the y–y axis, $\sigma_{m,y,d}$

$$\sigma_{m,y,d} = \frac{M_Y}{Z_Y} = \frac{99479\text{x } 10^3}{562.5\text{x}10^4} = 17.68Nmm^{-2}$$

Design bending stress about the z–z axis, $\sigma_{m,z,d} = 0$.

$$k_y = k_z = 0.5(1 + \beta c(\lambda_{rel,y} - 0.3) + \lambda^2_{rel,y})$$

$$k_y = 0.5(1 + 0.2(0.06 - 0.3) + 0.06^2) = 0.479$$

$$k_{cy} = k_{cz} = \frac{1}{k_y+\sqrt{k_y{}^2-\lambda^2{}_{rely}}} = \frac{1}{0.479+\sqrt{0.479^2-0.06^2}} = 1.04$$

$$\frac{\sigma_{C,0,d}}{k_{C,y,}f_{C,0,d}} + \frac{\sigma_{m,y,d}}{f_{m,y,d}} + k_m\frac{\sigma_{m,z,d}}{f_{m,z,d}} \leq 1$$

$$\frac{4.421}{0.76\text{x}11.02} + \frac{17.68}{11.02} + 0.7\text{ x }\frac{0}{F_{m,z,d}} \leq 1$$

0.257 + 0.586= 0.843 <1 ok

Therefore 300 × 300 column is adequate in buckling.

LATERAL TORSIONAL STABILITY

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$l_{ef} = l$ **= 3150 mm**

$\sigma_{crit} = \frac{0.78b^2}{hl_{ef}} E_{min} = \frac{0.78x300^2}{300x3150}$ x10302 = 765.29

$\lambda_{rely} = \sqrt{\frac{f_{m,k}}{\sigma_{crit}}} = \sqrt{\frac{33.17}{765.29}} = \mathbf{0.208}$

Since $\lambda_{rel,m}$ < 0.75, k_{crit} = 1

Check lateral torsional stability

$$\left(\frac{\sigma_{m,d}}{k_{crit} f_{m,y,d}}\right)^2 + \frac{\sigma_{c,0,d}}{k_{c,z} f_{c,0,d}} \leq 1$$

$$\left(\frac{17.68}{1x28.67}\right)^2 + \frac{4.421}{0.76x11.02} = 0.95 < 1\ ok$$

Hence the column is also adequate in lateral torsional buckling.

A) COLUMN A-2/ J-2 /A-7/J-7

Applied axial action

Loading from above = 69.28KN

Axial loads (x-x) =12.75KN

Axial loads (y-y) =10.17KN

Self-wt. =0.3 X0.3 X3.15 X7.5=2.12KN X1.35=2.87KN

TOTAL LOAD =12.75+10.17+2.87+69.28=201.8KN

Geometric properties

L=3.15m

Effective length= 1.0 X3150=3150mm

B=h=3000mm

.

Area =300 X300= 90000mm^2

I $=\frac{bh^3}{12}=\frac{300\times300^3}{12}$ = **675**$\times10^6$mm^4

W$=\frac{bh^2}{6}=\frac{300\times300^2}{6}$ = **450**$\times10^4$mm^4

$\boldsymbol{L} = \boldsymbol{l_{yy}} = \boldsymbol{l_{zz}} = \sqrt{\frac{I}{A}} = \sqrt{\frac{675\times10^6}{90000}}$ **=86.6.3mm**

Effective slenderness ratio

$\boldsymbol{\lambda} = \frac{L_e}{i} = \frac{3150}{675}$ **= 4.66**

$\lambda_{rely} = \lambda_{relz} = \frac{\lambda_y}{\pi}\sqrt{\frac{f_{c,0,k}}{E_{0.05}}} = \frac{4.66}{\pi}\sqrt{\frac{17.91}{10302}}$ **=1.521mm**

$\lambda_{rely} > 0.3$

BUCKLING (CL. 6.3.2, EC 5)

Design compressive stress

$\boldsymbol{\sigma}_{,c,0,d} = \frac{N}{A} = \frac{201.8\times10^3}{90\times10^3} = \mathbf{2.24}\boldsymbol{N/mm^2}$

Applied bending moment M is

$$M = (201.8 \times 100) = 20180KNmm$$

Design Bending strength $F_{m,d}$ **is**

$$f_{m,d} = K_h K_{mod.} \frac{f_{m,k}}{Y_m} = \frac{1.0\times0.8\times33.19}{1.3} = 28.67Nmm^{-2}$$

Design compression strength $f_{c,0,d}$ **is**

$f_{c,0,d} = K_{mod.} \frac{k_{C,0,k}}{Y_m} = \frac{0.8 \times 17.91}{1.3} = 11.02 Nmm^{-2}$

Design bending stress about the y–y axis, $\sigma_{m,y,d}$

$\boldsymbol{\sigma_{m,y,d}} = \frac{M_Y}{Z_Y} = \frac{201800 \times 10^3}{450 \times 10^4} = \mathbf{44.84 Nmm^{-2}}$

.

Design bending stress about the z–z axis, $\sigma_{m,z,d} = 0$.

$k_y = k_z = 0.5(1 + \beta c(\lambda_{rel,y} - 0.3) + \lambda^2_{rel,y})$

$k_y = 0.5(1 + 0.2(1.52 - 0.3) + 1.52^2) = 1.78$

$k_{cy} = k_{cz} = \frac{1}{k_y + \sqrt{k_y{}^2 - \lambda^2{}_{rely}}} = \frac{1}{1.78 + \sqrt{1.78^2 - 1.52^2}} = 0.36$

$$\frac{\sigma_{c,0,d}}{k_{C,y,}f_{C,0,d}} + \frac{\sigma_{m,y,d}}{f_{m,y,d}} + k_m \frac{\sigma_{m,z,d}}{f_{m,z,d}} \leq 1$$

$$\frac{2.24}{0.76 \times 11.02} + \frac{9.237}{28.67} + 0.7 \times \frac{0}{F_{m,z,d}} \leq 1$$

0.367+ 0.322= 0.698<1 ok

Therefore 300 × 300 column is adequate in buckling.

LATERAL TORSIONAL STABILITY

***l_{ef}* = *l* = 3150 mm**

$\boldsymbol{\sigma}_{crit} = \frac{0.78b^2}{hl_{ef}} E_{min} = \frac{0.78 \times 300^2}{300 \times 3150} \times 10302 = 382.64$

$\lambda_{rely} = \sqrt{\frac{f_{m,k}}{\sigma_{crit}}} = \sqrt{\frac{33.17}{382.64}} = \mathbf{0.294}$

Since $\lambda_{rel,m} < 0.75$, $k_{crit} = 1$

Check lateral torsional stability

$$\left(\frac{\sigma_{m,d}}{\boldsymbol{k}_{crit} f_{m,y,d}}\right)^2 + \frac{\sigma_{c,0,d}}{\boldsymbol{k}_{c,z} f_{c,0,d}} \leq 1$$

.

$$\left(\frac{9.237}{1\text{x}28.67}\right)^2 + \frac{3.079}{0.76\text{x}11.02} = 0.47 < 1\ ok$$

Hence the column is also adequate in lateral torsional buckling.

C) COLUMN C-2/ F-2

Applied axial action

Load from above =108.845KN

Axial loads (x-x) =14.75KN

Axial loads (y-y) = 23.75KN

Self-wt. =0.30 X0.30 X3.15 X7.5=0.238KN X1.35=1.275KN

TOTAL LOAD =23.75+14.75+1.275 + 53.87+108.845= 202.5KN

Geometric properties

L=3.15m

Effective length= 1.0 X3150=3150mm

B=h=3000mm

Area =300 X300= 90000mm^2

I $=\boldsymbol{\frac{bh^3}{12}} = \boldsymbol{\frac{300\times300^3}{12}} = \mathbf{675}\times10^6\text{mm}^4$

W$=\boldsymbol{\frac{bh^2}{6}} = \boldsymbol{\frac{300\times300^2}{6}} = \mathbf{450}\times10^4\text{mm}^4$

$\boldsymbol{L} = \boldsymbol{l_{yy}} = \boldsymbol{l_{zz}} = \sqrt{\boldsymbol{\frac{I}{A}}} = \sqrt{\frac{\mathbf{675\times10^6}}{90000}}$ **=86.6.3mm**

Effective slenderness ratio

$\boldsymbol{\lambda = \frac{L_e}{i} = \frac{3150}{675}} = \mathbf{4.66}$

$\lambda_{rely}^{\cdot} = \lambda_{relz}^{\cdot} = \frac{\lambda^{\cdot}_{y}}{\pi}\sqrt{\frac{f_{c,0,k}}{E_{0.05}}} = \frac{4.66}{\pi}\sqrt{\frac{17.91}{10302}}$ **=1.521mm**

$\lambda_{rely}^{\cdot} > 0.3$

BUCKLING (CL. 6.3.2, EC 5)

Design compressive stress

$\sigma_{,c,0,d} = \frac{N}{A} = \frac{202.5\times10^{3}}{90\times10^{3}} = \mathbf{2.24 N/mm^2}$

Applied bending moment M is

$$M = (108.845 \times 100) = 20250KNmm$$

Design Bending strength $F_{m,d}$ **is**

$f_{m,d} = K_h K_{mod.} \frac{f_{m,k}}{Y_m} = \frac{1.0\times0.8\times33.19}{1.3} = 28.67Nmm^{-2}$

Design compression strength $f_{c,0,d}$ **is**

$f_{c,0,d} = K_{mod.} \frac{k_{C,0,k}}{Y_m} = \frac{0.8\times17.91}{1.3} = 11.02Nmm^{-2}$

Design bending stress about the y–y axis, $\sigma_{m,y,d}$

$\sigma_{m,y,d} = \frac{M_Y}{Z_Y} = \frac{10884.5 \times 10^3}{133.33\times10^4} = \mathbf{8.16Nmm^{-2}}$

Design bending stress about the z–z axis, $\sigma_{m,z,d}$ **= 0.**

$k_y = k_z = 0.5(1 + \beta c(\lambda_{rel,y} - 0.3) + \lambda^2_{rel,y})$

$k_y = 0.5(1 + 0.2(1.52 - 0.3) + 1.52^2) = 1.78$

$k_{cy} = k_{cz} = \frac{1}{k_y + \sqrt{k_y{}^2 - \lambda^2_{rely}}} = \frac{1}{1.78 + \sqrt{1.78^2 - 1.52^2}} = 0.36$

$$\frac{\sigma_{c,0,d}}{k_{C,y,}f_{C,0,d}} + \frac{\sigma_{m,y,d}}{f_{m,y,d}} + k_m \frac{\sigma_{m,z,d}}{f_{m,z,d}} \leq 1$$

$$\frac{2.24}{0.76\text{x}11.02} + \frac{8.16}{28.67} + 0.7 \text{ x } \frac{0}{F_{m,z,d}} \leq 1$$

0.26+ 0.28= 0.54 <1 ok

.

Therefore 300 × 300 column is adequate in buckling.

LATERAL TORSIONAL STABILITY

l_{ef} = l = 3150 mm

$$\boldsymbol{\sigma}_{crit} = \frac{0.78b^2}{hl_{ef}} E_{min} = \frac{0.78x200^2}{200x3150} x10302 = 510.19$$

$$\boldsymbol{\lambda}_{rely} = \sqrt{\frac{f_{m,k}}{\sigma_{crit}}} = \sqrt{\frac{33.17}{510.19}} = \mathbf{0.254}$$

Since $\lambda_{rel,m}$ < 0.75, k_{crit} = 1

Check lateral torsional stability

$$\left(\frac{\sigma_{m,d}}{\boldsymbol{k}_{crit}f_{m,y,d}}\right)^2 + \frac{\sigma_{c,0,d}}{\boldsymbol{k}_{c,z}f_{c,0,d}} \leq 1$$

$$\left(\frac{8.16}{1\text{x}28.67}\right)^2 + \frac{2.24}{0.76\text{x}11.02} = 0.35 < 1\ ok$$

Hence the column is also adequate in lateral torsional buckling.

4.4 DESIGN OF FOUNDATION (PAD FOOTING)

PAD FOOTING FOR (CB -1)

Shear failure could arise: (a) at the face of the column (b) at a distance d from the face of the column (c) punching failure of the slab

FACE SHEAR (CL. 6.4.5, EC 2)

Load on footing due to column is 1.35 × 99.479= 134.30 kN

Design shear stress at the column perimeter, ν_{Ed}, is ν_{ED}

Effective depth d,

d = h − cover − diameter of bar = 300 − 50 − 8 = 242 mm.

$$\nu_{ED} = \beta \frac{V_{ED}}{U_0 d} = 1 \times \frac{134.30 \times 10^3}{(4 \times 200) \times 242} = 1.44 N/mm^2$$

$V_{Rd,\,max} = 0.5\nu f_{cd}$

$\nu = 0.6[1 - (f_{ck}/250)] = 0.6[1 - (30/250)] = 0.528$

$$f_{cd} = \alpha_{cc} \frac{f_{ck}}{\gamma_c} = 1 \times \frac{30}{1.5} = 20 N/mm^2$$

$= 0.5 \times 0.528 \times 20 = 5.28\ N\ mm^{-2} > \nu_{Ed}$ OK

TRANSVERSE SHEAR (CL. 6.4.4, EC 2)

From above, design shear force on footing, V_{Ed} = 134.30kN

Earth pressure , $P_E = \frac{V_{ED}}{base\ area} = \frac{134.30}{0.9^2} = 165.80\ kN/m^2$

Ultimate load at 1d area is,

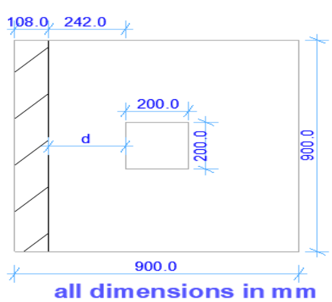

Figure 4.26

Area = ΔVEd = P_E (0.9 × [0.9 − 0.108]) = 117.61 kN

Applied shear force is

$V_{Ed,red}$ = VEd − ΔVEd = 134.30− 117.61 = 16.69 kN

Design transverse shear stress, V_{Ed}, is

$V_{Ed} = \frac{V_{Ed,red}}{bd} = \frac{16.69 \times 10^3}{900 \times 242} = 0.076\ N/mm^2$

where b = width of footing = 0.9m

fck = 30 N mm^{-2}

$C_{Rd,c} = 0.18/\gamma_c = 0.18/1.5 = 0.12$ N mm^{-2}

$$K = 1 + \sqrt{\frac{200}{d}} = 1 + \sqrt{\frac{200}{242}} = 1.9 < 2.0 \text{ OK}$$

$$Area\ of\ steel = 1000 \times \frac{201}{150} = 1340 mm^2$$

$$\rho_1 = \sqrt{\rho 1y\ \rho 1x} = \sqrt{\frac{Asl,y}{bd} \times \frac{Asl,x}{bd}} =$$

$$\sqrt{\frac{1340}{10^3 \times 242} \times \frac{1340}{10^3 \times 242}} = 0.0055 < 0.02 \text{ OK}$$

$$\sigma_{cp} = 0$$

$\nu_{min} = 0.035k^{3/2}f_{ck}^{1/2} = 0.035 \times 1.9^{3/2} \times 30^{1/2} = 1.8$

Design shear resistance of concrete, $V_{Rd,c}$, is given by

$V_{Rd,c} = [C_{Rd,c}k(100\rho_1 f_{ck})^{1/3} + k_1\sigma_{cp}] \times (2d/a)$

$= [0.12 \times 1.9(100 \times 0.0055 \times 30)^{1/3} + 0] \times 2$

$= 3.6$ N $mm^{-2} \geq [v_{min} + k_1\sigma_{cp}] \times (2d/a)$

$= 1.8 \times 2 = 3.6$ Nmm^{-2} Since V_{Ed} (= 0.076 Nmm^{-2}) < $V_{Rd,c}$ (= 3.6 Nmm^{-2})

shear reinforcement is NOT required.

PUNCHING SHEAR

Check punching shear at 2d from face of column.

Basic control perimeter, u1, is

u1 = column perimeter + 2π(2d)

= 4 × 300 + 2π (2 × 242) =4241.06 mm

Area within critical perimeter,

$A = 4 \times 300(2 \times 242) + 300^2 + \pi (2 \times 242)^2 = 1.406 \times 10^6$ mm^2

Ultimate load on shaded area, $\Delta V_{Ed} = \rho_E \times A = 165.80 \times 1.406 = 233.24$ kN

Applied shear force, $V_{Ed,red} = V_{Ed} - \Delta V_{Ed} = 134.30 - 23.32 = 110.98$ kN

Punching shear stress,

$$V_{Ed} = \frac{V_{Ed}}{U_1 d} = \frac{110.98 \times 10^3}{4241.06 \times 242} = 0.108 \text{ Nmm}^{-2} < V_{Rd,c} = 3.6 \text{ Nmm}^{-2} \text{ OK}$$

Hence, no shear reinforcement is required.

PAD FOOTING FOR (CB -2)

FACE SHEAR (CL. 6.4.5, EC 2)

Base for columns supporting a max axial load of 200KN

Load on footing due to column is 1.35 × 202.5= 273.4 kN

Design shear stress at the column perimeter,ν_{Ed}, is ν_{ED}

Effective depth d,

d = h − cover − diameter of bar = 300 − 50 − 8 = 242 mm.

$$\nu_{ED} = \beta \frac{V_{ED}}{U_0 d} = 1 \times \frac{273.4 \times 10^3}{(4 \times 200) \times 242} = 1.412 N/mm^2$$

$V_{Rd,\max} = 0.5 \nu f_{cd}$

$\nu = 0.6[1 - (f_{ck}/250)] = 0.6[1 - (30/250)] = 0.528$

$$f_{cd} = \alpha_{cc} \frac{f_{ck}}{\gamma_c} = 1 \times \frac{30}{1.5} = 20 N/mm^2$$

$= 0.5 \times 0.528 \times 20 = 5.28$ N mm$^{-2} > \nu_{Ed}$ OK

TRANSVERSE SHEAR (CL. 6.4.4, EC 2)

From above, design shear force on footing, V_{Ed} = 273.4kN

Earth pressure , P_E $= \frac{V_{ED}}{base\ area} = \frac{273.4}{0.9^2} = 337.53\ kN/m^2$

Ultimate load at 1d area is,

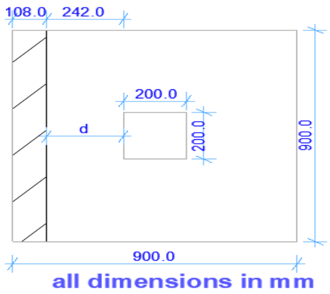

Figure 4.27

Area = ΔVEd = P_E (0.9 × [0.9 − 0.108]) = 240.59 kN

Applied shear force is

$V_{Ed,red}$ = VEd − ΔVEd = 273.4− 240.59 = 32.80 kN

Design transverse shear stress, V_{Ed}, is

$$V_{Ed} = \frac{V_{Ed,red}}{bd} = \frac{32.80 \times 10^3}{900 \times 242} = 0.15\ N/mm^2$$

where b = width of footing = 0.9m

fck = 30 N mm^{-2}

$C_{Rd,c}$ = 0.18/γ_c = 0.18/1.5 = 0.12 N mm^{-2}

$$K = 1 + \sqrt{\frac{200}{d}} = 1 + \sqrt{\frac{200}{242}} = 1.9 < 2.0 \text{ OK}$$

$$Area\ of\ steel = 1000 \times \frac{201}{150} = 1340\text{mm}^2$$

$$\rho_1 = \sqrt{\rho 1y\ \rho 1x} = \sqrt{\frac{Asl,y}{bd} \times \frac{Asl,x}{bd}} =$$

$\sqrt{\frac{1340}{10^3\times 242}\times\frac{1340}{10^3\times 242}}$ =0.0055 < 0.02 OK

$\sigma_{cp} = 0$

$\nu_{min} = 0.035k^{3/2}f_{ck}^{1/2} = 0.035 \times 1.9^{3/2} \times 30^{1/2} = 1.8$

Design shear resistance of concrete, $V_{Rd,c}$, is given by

$V_{Rd,c} = [C_{Rd,c}k(100\rho_1 f_{ck})^{1/3} + k_1\sigma_{cp}] \times (2d/a)$

$= [0.12 \times 1.9(100 \times 0.0055 \times 30)^{1/3} + 0] \times 2$

$= 3.6$ N mm$^{-2} \geq [v_{min} + k_1\sigma_{cp}] \times (2d/a)$

$= 1.8 \times 2 = 3.6$ Nmm^{-2} Since V_{Ed} (= 0.076 Nmm^{-2}) < $V_{Rd,c}$ (= 3.6 Nmm^{-2})

shear reinforcement is NOT required.

PUNCHING SHEAR

Check punching shear at 2d from face of column.

Basic control perimeter, u1, is

u1 = column perimeter + 2π(2d)

$= 4 \times 300 + 2\pi\ (2 \times 242)$ =4241.06 mm

Area within critical perimeter,

$A = 4 \times 300(2 \times 242) + 300^2 + \pi\ (2 \times 242)^2 = 1.406 \times 10^6$ mm^2

Ultimate load on shaded area, $\Delta V_{Ed} = \rho_E \times A = 337.53 \times 1.406 = 474.57$ kN

Applied shear force, $V_{Ed,red} = V_{Ed} - \Delta V_{Ed} = 273.4 - 47.45 = 225.95$ kN

Punching shear stress,

$V_{Ed} = \frac{V_{Ed}}{U_1 d} = \frac{225.95\times 10^3}{4241.06\times 242} = 0.22$ Nmm^{-2} $< V_{Rd,c}$ = 3.6 Nmm^{-2} OK

Hence, no shear reinforcement is required.

4.5 COMPUTER AIDED DESIGN(TEKLA STRUCTURALDESIGNER)

4.5 .1ROOF DESIGN (TRUSSES)

Geometry

Geometry (m) - D50 (EC5)

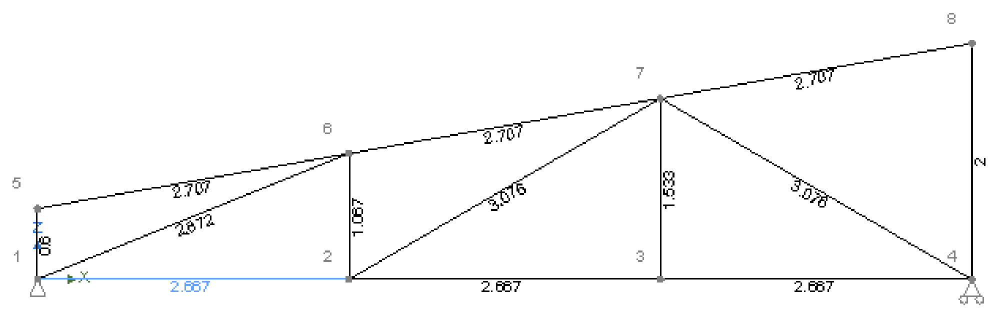

fig 4.28

table 4.6

Materials

Name	Density (kg/m^3)	Youngs Modulus kN/mm^2	Shear Modulus kN/mm^2	Thermal Coefficient °C^{-1}
D50 (EC5)	620	14	0.88	0

Sections

Name	Area	Moment of inertia		Shear area parallel to	
		Major	Minor	Minor	Major
	(cm^2)	(cm^4)	(cm^4)	(cm^2)	(cm^2)
Top - 50x100	50	416.7	104.2	41.7	41.7
Bottom - 50x150	75	1406.2	156.3	62.5	62.5
Internal - 50x75	37.5	175.8	78.1	31.3	31.3

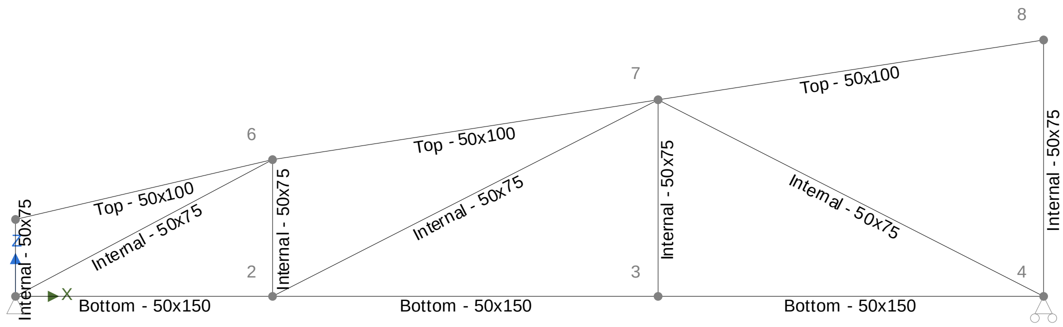

fig 4.29

table 4.7

Elements

Element	Length (m)	Nodes		Section	Material	Releases			Rotated
		Start	End			Start moment	End moment	Axial	
1	2	1	2	Bottom - 50x150	D50 (EC5)	Fixed	Fixed	Fixed	

Element	Length (m)	Nodes		Section	Material	Releases			Rotated
		Start	End			Start moment	End moment	Axial	
2	3	2	3	Bottom - 50x150	D50 (EC5)	Fixed	Fixed	Fixed	
3	3	3	4	Bottom - 50x150	D50 (EC5)	Fixed	Fixed	Fixed	
4	2.054	5	6	Top - 50x100	D50 (EC5)	Fixed	Fixed	Fixed	
5	3.036	6	7	Top - 50x100	D50 (EC5)	Fixed	Fixed	Fixed	
6	3.036	7	8	Top - 50x100	D50 (EC5)	Fixed	Fixed	Fixed	
7	0.6	1	5	Internal - 50x75	D50 (EC5)	Fixed	Fixed	Fixed	
8	1.067	2	6	Internal - 50x75	D50 (EC5)	Fixed	Fixed	Fixed	
9	1.533	3	7	Internal - 50x75	D50 (EC5)	Fixed	Fixed	Fixed	
10	2	4	8	Internal - 50x75	D50 (EC5)	Fixed	Fixed	Fixed	
11	2.267	1	6	Internal - 50x75	D50 (EC5)	Fixed	Fixed	Fixed	
12	3.369	2	7	Internal - 50x75	D50 (EC5)	Fixed	Fixed	Fixed	
13	3.369	7	4	Internal - 50x75	D50 (EC5)	Fixed	Fixed	Fixed	

Analysis Results

Forces Member results

Load combination: 1.0G + 1.0Q + 1.0RQ (Service)

Member	Axial force			
	Pos (m)	Max (kN)	Pos (m)	Min (kN)
Bottom Chord	2	-5.1	0	-5.2 (min)
Top Chord	2.054	5.5	8.126	0.1
member 13	3.369	5.9 (max)	0	5.4
Member4	0	-0.2	3.369	-0.2

Load combination: 1.0G + 1.0Q + 1.0RQ (Service)

Member	Axial deflection			
	Pos (m)	Max (mm)	Pos (m)	Min (mm)
Bottom Chord	8	0.4	0	0
Top Chord	0	0.1	8.126	-0.1
member 13	0	0.7 (max)	3.369	0.4
Member4	3.369	-0.5	0	-0.5 (min)

;

4.5.2DESIGN RESULT FROM TEKLA STRUCTURES(MATERIALS)

Table 4.8

Material Listing

Structure

Timber Beams

Section Size	Grade	No.	Total Length [m]	Mass [kg]	Embodied Carbon Mass [kgCO_2e]
75x100	D50	135	388.200	2154.51	1103
100x150	D50	27	102.600	1138.86	583
100x200	D50	42	78.900	1167.72	598
150x200	D50	108	205.200	4555.44	2332
150x300	D50	183	515.400	17162.82	8787
200x200	D50	48	144.000	4262.40	2182
200x300	D50	213	843.000	37429.20	19164
300x300	D50	3	24.000	1598.40	818
Total		**759**	**2301.300**	**69469.35**	**35568**

Timber Columns

Section Size	Grade	No.	Total Length [m]	Mass [kg]	Embodied Carbon Mass [kgCO_2e]
200x200	D50	42	132.300	3916.08	2005
300x300	D50	78	245.700	16363.62	8378
Total		**120**	**378.000**	**20279.70**	**10383**

Timber

Grade	Mass [kg]	Gross Surface Area [m^2]	Net Surface Area [m^2]	Volume [m^3]
D50	89749.05			121.3
Custom	0.00	716.9	716.9	71.7
Total	**89749.05**	**716.9**	**716.9**	**193.0**

Total

Material	Mass [kg]	Gross Surface Area	Net Surface Area	Volume [m^3]
Timber	89749.05	716.9	716.9	193.0
Total	**89749.05**	**716.9**	**716.9**	**193.0**

4.6 COMPARATIVE ANALYSIS OF THE TWO METHODS

4.6.1Structural analysis of members

Strength grades

manual = N2 -7700Kg/m^3

.

CAD = D50 -620Kg/m^3

BEAM				
member	parameter	MANUAL	CAD	Difference
beam(4m)	Max moment	10.17knm	8.3Knm	1.87
	Shear force	10.17kn	8.3Knm	1.87
Beam(6m)	Max moment	22.88knm	19.7knm	3.18
	Shear force	15.25kn	13.13kn	2.25

column				
member	parameter	MANUAL	CAD	Difference
COL C-1(3rd FLOOR)	Axial forces	24.23kn	21.3kn	2.93
COL C-1(2ndFLOOR)	Axial forces	48.47kn	43.5kn	4.97
COL C-1(1ST FLOOR)	Axial forces	99.47kn	87.8kn	11.67

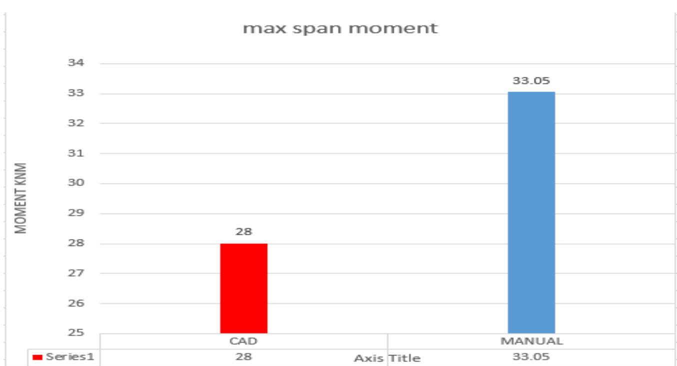

Fig 4.30 max moment bar chart

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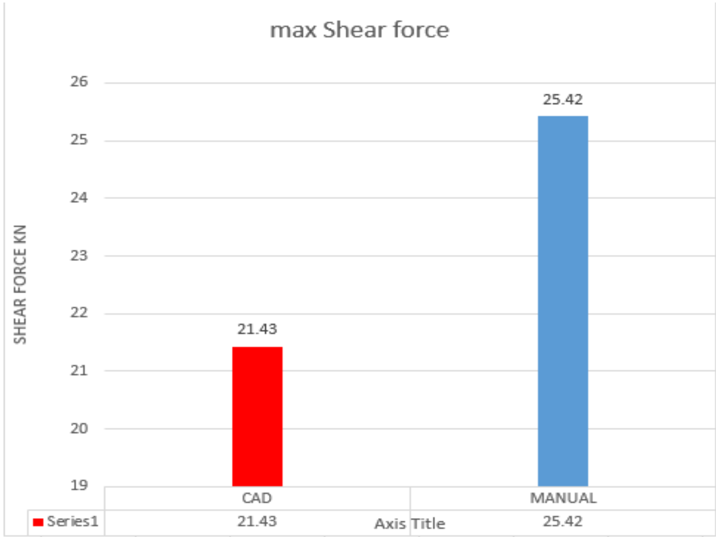

Fig 4.31 shear bar chart

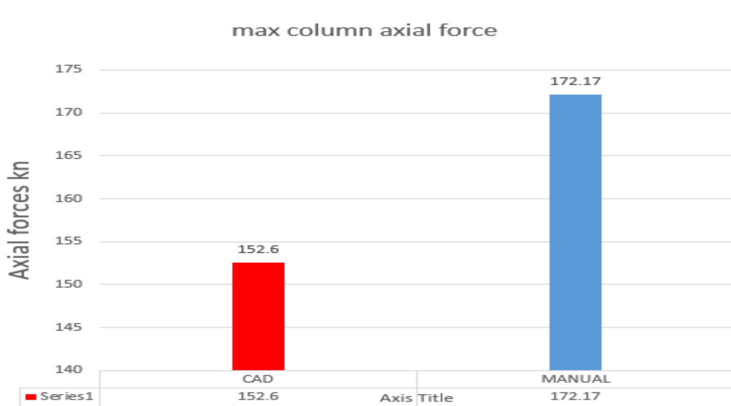

Fig 4.32 max axial forces bar chart

4.6.2 COST ANALYSIS

The approximate cost of a Nigerian timber of cross section 100mm x150mm (15000m^2) is ₦7000

General sections provided by tekla structural designer for beams and columns

Table 4.9

Section Size (mm)	Area mm^2	Grade	NO	TOTAL AREA
75x100	7500	D50	59	442500
100x150	1500	D50	22	330000
100x200	20000	D50	30	600000
150x200	30000	D50	65	1950000
150x300	45000	D50	15	675000
200x200	40000	D50	17	680000
200x300	60000	D50	40	2400000
300x300	90000	D50	45	4050000
Total	294000		293	8967500

From the above the total cost of the sections listed from tekla is given by

15000m^2 ----------------- ₦7000

8967500m^2 -------------------------- ₦ x

Hence the cost is given to be $= \frac{8967500\text{m2 x ₦}7000}{15000\text{m2}}$

= ₦4,184,833.3

General sections provided from the manual design

Table 4.10

Section Size (mm)	**Area mm^2**	**Grade**	**NO**	**TOTAL AREA**
75x100	7500	N2	30	225000
100x150	15000	N2	42	630000
100x200	20000	N2	35	700000
150x200	30000	N2	59	1770000
150x300	45000	N2	15	675000
200x200	40000	N2	13	520000
200x300	60000	N2	50	3000000
300x300	90000	N2	49	4410000
Total	294000		293	11930000

$15000m^2$ ----------------- ₦7000

$11930000m^2$ -------------------------- ₦ x

Hence the cost is given to be $= \frac{11930000m^2 \text{ x ₦}7000}{15000\text{m}2}$

= ₦5,567,333.3

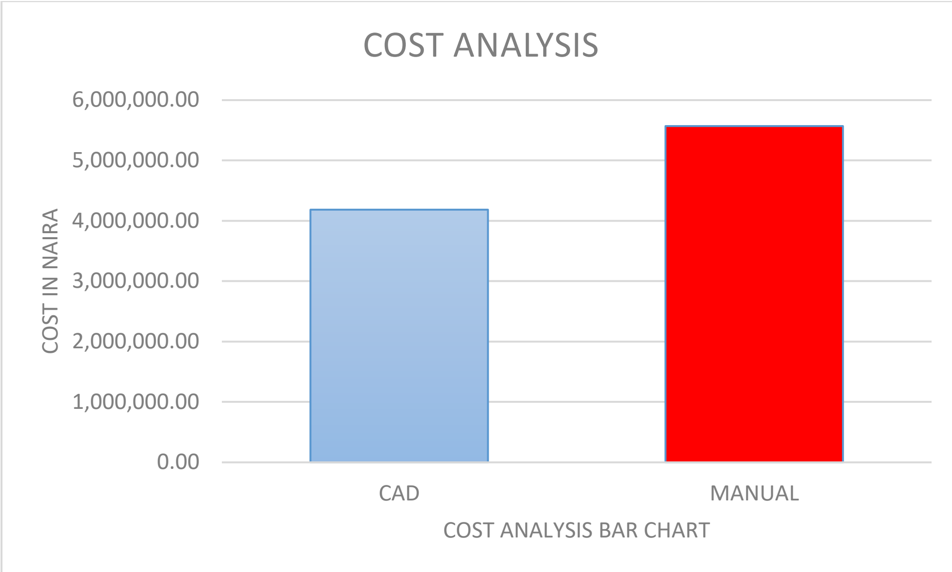

Figure 4.33

4.7 DISCUSSION

The comparative analysis done on the result of the structural analysis of the major structural elements showed that both method ensures stability which is the primary.

Aim of design. the much difference in the CAD design output to the manual output can be traced to the difference in the density of the timber being used being a result of the software no having built in section classification of Nigerian timber.

The sectional values produced from both is then calculated in terms of cost to show the cost effectiveness of each method hence it can be deduced that the CAD design produced sections lower in size to that of the manual designs. the cost of the total sectional area of the timber sections of the manual design are higher to a great extent to that of the manual design making the CAD design method more Economical and stress free while designing

CHAPTER FIVE

5.0 CONCLUSION AND RECOMENDATION

5.1 CONCLUSION

In summary, based on the findings presented in Chapter Four,

Timber has a great usefulness in the modern day construction industry having several properties which makes it ecofriendly. It has demonstrated its ability to withstand the various stresses.

1)The use of CAD designs in the design of timber structures provides sections which are more economical than the manual design calculation. The ease of software designs and time saving ability makes it necessary to be adopted fir economic design.

2)manual designs especially for Nigeria provides the designer the opportunity to apply the properties of timber species nearest to them

3)the software provided lower sectional area to the section produced by the manual

design making the computer aided design more cost effective

design making the computer aided design more cost effective

5.2 RECOMMENDATIONS

Based on the result generated through this research, the following recommendation can be given:

1)The structural design of a multi –story timber building can be carried out using either manual or computer aided design software built to design timber, since the result from the structural analysis both satisfies the stability requirement

2)While carrying out the structural analysis and design of timber structural members, the designer must seek to understand the full configuration of the software in other to make proper decision as to Arrive at the expected results

3)structural analysis of members can be done on the computer design software since it gives a lower value and the design done manually using a locally graded timber nearest to the Engineer

4)The design of timber structures as thought in tertiary institution in Nigeria should embrace teaching and training students to become confident in the use of computer aided software in designing timber.

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