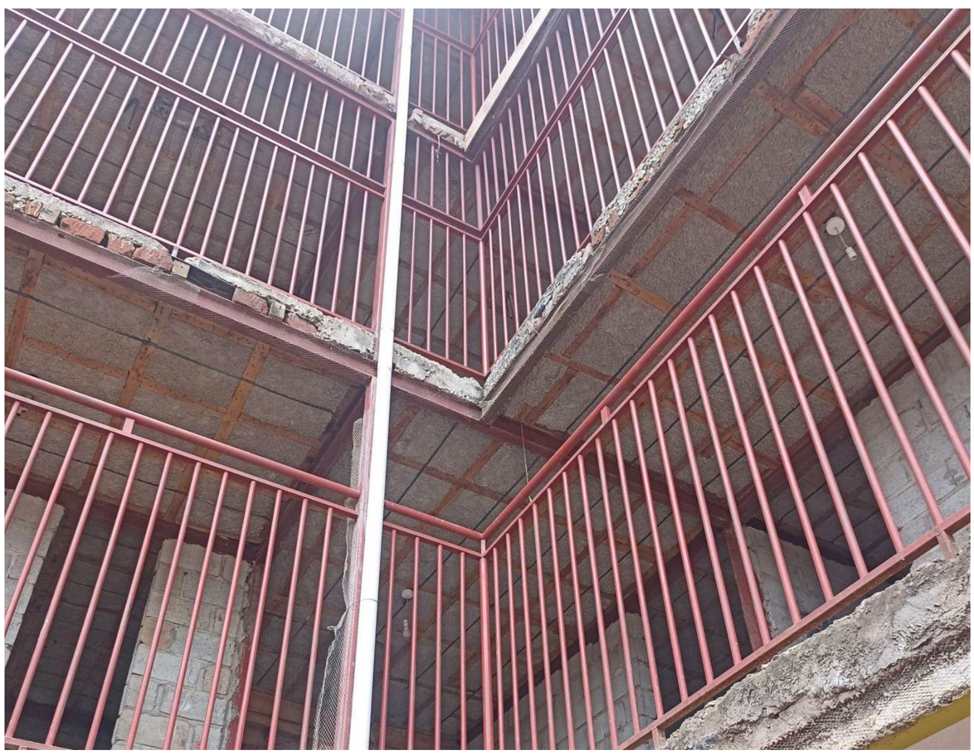
Steel-Timber-Concrete 'Composite' Building Method

TECHNICAL REPORT



National Building Review Board Technical Committee

APRIL 2024

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TERMS AND DEFINITIONS

AISC:	American Institute of Steel Construction
ARB:	Architects Registration Board
ASD:	Allowable Stress Design
ASME:	American Society of Mechanical Engineers
BCA:	Building Control Act
BCR:	Building Control Regulations
BFC:	Busoga Forestry Company
BRC:	British Reinforcing Cage
BS:	British Standard
CAES:	College of Agricultural and Environmental Sciences
CCA:	Chromated Copper Arsenate
CEDAT:	College of Engineering, Design, Art & Technology
CHS:	Circular Hollow Section
CLT:	Cross Laminated Timber
CPD:	Continuous Professional Development
DPC:	Damp Proof Course
DPM:	Damp Proof Membrane
EAS:	East African Standard
EASC:	East African Standards Committee
EN:	European Standard (Euro Norm)
ERB:	Engineers Registration Board
FCAW:	Flux Cored Arc Welding
FSC:	Forest Stewardship Council
FSSD:	Forest Sector Support Department
GMAW:	Gas Metal Arc Welding
GTAW:	Gas Tungsten Arc Welding
HVAC:	Heating, Ventilation and Air Conditioning
ISO:	International Organization for Standardization

MAK:	Makerere University
MC:	Moisture Content
MEP:	Mechanical, Electrical and Plumbing
MIG:	Metal Inert Gas
MoE, MOE:	Modulus of Elasticity
MoR, MOR:	Modulus of Rupture
MoWT:	Ministry of Works and Transport
MPa	Mega Pascal
MWE:	Ministry of Water and Environment
NBC:	National Building Code
NBC(SD):	National Building (Structural Design) Code
NBRB:	National Building Review Board
NDT:	Non-Destructive Testing
NFA:	National Forestry Authority
NFSC:	National Forest Stewardship Council
NWSC:	National Water and Sewerage Corporation
OPC:	Ordinary Portland Cement
PVC:	Poly Vinyl Chloride
SHS:	Square Hollow Section
SLS:	Serviceability Limit State
STC:	Steel-Timber-Concrete 'Composite' construction
STIL:	Steel and Tube Industries Limited
TC:	Technical Committee
TCC:	Timber-Concrete Composite
TIG:	Tungsten Inert Gas
UIPE:	Uganda Institution of Professional Engineers
ULS	Ultimate Limit State
UNBS:	Uganda National Bureau of Standards
URSB:	Uganda Registration Services Bureau
US:	Uganda Standard

UTGA:	Uganda Timber Growers Association
UTM:	Universal Testing Machine
W/C:	Water/Cement Ratio
WPS:	Welding Procedure Specification

EXECUTIVE SUMMARY

The Steel-Timber-Concrete (STC) 'Composite' construction method combines structural steel framing with a floor slab system of timber joists supported by steel beams. The timber joists are typically 100x100mm and spaced at 600mm intervals. An expanded metal lathe is nailed to the bottom of the timber framework, filled with pricking up mortar, and another layer is nailed to the top, creating a void between the concrete topping and ceiling soffit. The concrete topping, lightly reinforced with BRC mesh, is then cast on top of the mild steel expanded metal.

Due to the increasing use of the STC method and numerous complaints to NBRB, a study was conducted to assess its viability in Uganda. The study found significant flaws that made the method risky, leading to the issuance of Legal Notice No. 11 of 2022 on 23rd September 2022, prohibiting its use. Resulting from a recommendation from the first study, a Technical Committee was formed on 18th May 2023 to review the method and possibly develop guidelines for its safe use.

The overall objective of the Technical Committee was to "Assess the viability of the STC method, and if found viable, develop guidelines for the safe design, fabrication and erection of the structures (implementation) of STC buildings".

The specific objectives of the Technical Committee were to:

i. Review the 2022 NBRB Study report and other documents to form an opinion on the viability of the STC method as marketed before the prohibition.
ii. Undertake characterization of local Ugandan timber commonly used in structural members in construction in Uganda.
iii. Carry out a survey of structural steel on the Ugandan Market and check compliance to current design standards.
iv. Carry out a survey of concrete practices in the construction industry in Uganda to ascertain compliance with specifications for typical works.
v. Develop guidelines for the design and construction of structures with Steel, Timber and Concrete.
vi. Develop guidelines for welders of structural steel works in the building construction sector.

To satisfactorily answer the objectives, the following activities were undertaken;

i. Document Review

ii. Inspection of STC construction sites
iii. Meeting with Industry Players
iv. Timber Testing
v. Tests on Structural Steel
vi. Assessing welded steel connections on various sites
vii. Tests on Concrete and survey on common practices in the use of Concrete
viii. Structural analysis and capacity checks
ix. Formulation of guidelines for construction using steel, timber and concrete
x. Stakeholder engagement workshop

The technical committee concluded as follows on the six specific objectives:

i. Review of the NBRB Study report and other documents to form an opinion on the viability of the STC method as marketed before the prohibition.

Given the lack of adequate engineering basis, lack of composite action, lack of consideration for lateral resistance, lack of hogging moment reinforcement, poor welding quality, lack of specification of timber grades and properties for use, the poor timber on the open market; the lack of engineered connections; the method as marketed by the proponents and deployed in industry with its variations is generally unsafe and not viable from a structural point of view.

The Steel-Timber Concrete 'Composite' Building Method as marketed currently should remain prohibited. The prohibition should not extend to the possible use of the three materials of steel, timber and concrete in other combinations (for which minimum guidelines have been proposed).

ii. Characterization of local Ugandan timber commonly used in structural members in construction in Uganda.

The Characteristic bending strength of locally available timber as tested returned the following results: 26.3 MPa (Pine from Bwaise), 34.7 MPa (Eucalyptus from Bwaise), 15.7 MPa (Pine from Ndeeba), 35.9 MPa (Musambya from Ndeeba), 22 MPa (Mugavu from Ndeeba), 23.6 MPa (Musizi from Ndeeba) and 43 MPa (Eucalyptus from Ndeeba).

Results reveal considerable strength variations within the same timber species, indicating that assigning a strength class based solely on species is unreliable.

Factors such as age, growth conditions, defects, and post-harvest handling significantly influence timber strength.

The Moisture Content in the timber was also very high (up to 75.1%) as compared to what would be specified for structural purposes (15-22% at the time of erection).

For initial design/sizing, C14 and D18 classes can be used subject to confirmation through testing and statistical analysis to obtain characteristic values which are applicable to limit state design.

iii. Survey of structural steel on the Ugandan Market and check compliance to current design standards.

The results on yield strength were in the range of 270-520MPa, indicating that it is feasible to achieve the expected yield values, as per NBC, 2019. 33 of the 34 samples hit a yield strength value of more than 275MPa, with a characteristic value of 332 MPa.

Nine of the 34 samples (26.5%) failed to achieve the minimum tensile strength of 410MPa, which is the specified tensile strength as per US ISO 603-2.

16 of the 34 samples (47%) samples did not meet the minimum required for elongation (21-23% according to US ISO 603-2, for S275), which implies low ductility.

Five of the 29 samples had higher carbon content than the maximum recommended for ductility. 3 of the 35 samples had phosphorus and Sulphur amounts higher than the maximum recommended by the standards. Only two of the 29 samples returned Carbon Equivalent Values higher than the recommended. Generally, the structural steel as sampled is weldable.

iv. Survey of concrete practices in the construction industry in Uganda to ascertain compliance with specifications for typical works.

A survey of concrete practices in Uganda's construction industry revealed that different mix ratios were used in industry for target concrete Grade C25. The six mixes tested returned Compressive strength results ranging between 26 MPa and 29 MPa. This highlights the importance of project-specific mix design and trial mixes for each batch of materials. This approach is recommended rather than adopting a generic mix design ratio, which is often misinterpreted by artisans.

The 7-day concrete tests showed strength gains between 61% and 74% of the 28-day strength, challenging the assumption in practice of 70% strength gain.

Lack of material quality assurance procedures was common, with aggregates sourced by unqualified persons. Water quantity for concrete was often determined visually, with little consideration for the water-cement ratio. The informal sector lacks awareness of concrete grades, quality control, and proper mix proportioning.

v. Development of guidelines for the design and construction of structures with steel, timber and concrete

The use of Steel, Timber, and Concrete in structures can be successful when following scientifically tested guidelines and core engineering principles. Professionals engaged by developers are responsible for ensuring a safe design, satisfying themselves that it works at both ultimate and serviceability limit states, and issuing a certificate of good structural practice. Supervision during construction is essential and should ideally be conducted by the same person who designed the structure. It is the developer's responsibility to submit safe designs for approval by the planning authority, as developments without approved plans are illegal.

The Technical committee proposed guidelines on:

a) Structural Timber: Sourcing of good quality timber, selection & handling, keeping timber free from moisture content, Durability Treatment against termite attack, Grading & marking, and quality assurance.
b) Structural Steel: Section sizing, Durability of Structural Steel- Protection against Corrosion, Durability of Structural Steel-Protection against Fire, Standardization and Labelling/Marking, Quality assurance.
c) Concrete in Construction: Sourcing, selection & storage of materials, Concrete Mix Design and batching, Quality control, reinforcing steel quality assurance.
d) Connections: Welded connections, Bolted Connections, Timber to timber connections, Steel to Concrete connections, Concrete to Timber connections, Steel to Steel Connections
e) Reinforcement of the concrete slab: anti-cracking and flexural reinforcement Design & location, Concrete cover requirement.
f) Mechanical and Plumbing installations: Mechanical Ventilation and Air Conditioning Installation, Plumbing Installations, Fire Fighting Installations.

g) Electrical Installations: Proficiency level of technicians, Conduiting, protective devices, cabling.
h) Fire resistance: The Minimum fire rating should be at least 1 hour, corresponding to a minimum slab thickness of 100mm.
i) Minimum Design guidelines: Design by professionals, Checks for Stability, robustness, Durability & Strength at the very least, detail allowing for inspection & maintenance & buildability.
j) Post Construction and Maintenance: As-built drawings, occupation permits, maintenance needs.

vi. Guidelines for welders of structural steel works in the building construction sector.

Welding practitioners should be trained and certified to at least Level 4 for structural construction work, with training available at UBTEB-accredited institutions or through the Directorate of Industrial Training.

The Technical committee recommended as follows:

To Building Committees

i. To refrain from approving building plans without a design report, drawings showing connection details, sizing of structural elements, specifications for materials and geotechnical investigation reports.
ii. Should request developers of already existing STC structures to submit detailed Structural Integrity Assessment reports which should be carried out by registered engineers before an occupation permit is issued.
iii. Ensure that the STC structures which can be retrofitted, are strengthened and brought into compliance with the Building Control Regulatory Framework.

To Developers

i. Engage qualified professional engineers to carry out structural integrity assessments for existing STC constructions and make proposals for retrofitting.
ii. Apply for building permits for any proposed developments (including retrofits) and only start after issuance of the same from the appropriate building committees
iii. Engage qualified professionals to undertake all building operations, including design, construction and quality control.

iv. Apply for occupation permits from the appropriate building committees before occupying the buildings.

To Uganda National Bureau of Standards

i. Require steel suppliers to indicate grade and manufacturer on the structural steel sections on sale.
ii. Enforce the requirement for all timber producers to mark their products as required by the standards.
iii. Formulate timber standards for engineering purposes, and make them mandatory.

To Engineers Registration Board

i. To discipline professionals who negligently stamp drawings for which a design basis is not available and who fail in their duty of care to clients.
ii. Arrange for Continuous Professional Development courses for welding engineers, instructors and inspectors.
iii. Require each Professional Engineer to file projects that they have certified to ERB as pre-condition for renewal of practising license.

To National Building Review Board

i. Review the National Building (Structural Design) Code, 2019 to align with the current limit state design (Load and resistance factor design) philosophy.
ii. Plan to sensitise and roll out these guidelines with a timeline on review of the same.
iii. To consider the proposed guidelines as basis for a statutory instrument.
iv. Review the BCR, 2020 to include Welding Procedure Specification as a mandatory submission for steel structures for issuance of building permit.

To Uganda Registration Services Bureau

i. To require that Engineering and Construction firms should have at least one of the directors as a professional in a construction field.
ii. To consult the Minister in charge of works on any applications for patents or utility models that are of an engineering nature.

1 INTRODUCTION

1.1 Background

1.1.1 National Building Review Board

The Government of Uganda, through the Ministry of Works and Transport (MoWT) enacted the Building Control Act, 2013 in a bid to consolidate, harmonize and amend the law relating to the erection of buildings; to provide for building standards; to establish a National Building Review Board (NBRB) and Building committees; to promote and ensure planned, decent and safe building structures that are developed in harmony with the environment, and for other related matters.

Section 9 of the Building Control Act, 2013, (BCA, 2013) provides the functions of NBRB as to monitor building developments; to ensure that the design and construction of buildings and utilities to which the public is to have access to cater for persons with disabilities; to oversee, inspect and monitor operations of Building Committees; among others.

1.1.2 Steel-Timber-Concrete 'Composite' Construction

The Steel-Timber-Concrete (STC) 'Composite' method of construction incorporates Steel, Timber and Concrete in its primary structural elements.

The method consists of structural steel framing and a floor slab system comprising timber joists supported by steel beams. The 100x100mm timber joists are spaced at 600mm in orthogonal directions. At the bottom of the timber framework is nailed expanded metal lathe that is later filled with pricking up mortar. At the top of the timber framework, a second layer of expanded metal lathe is nailed, thus leaving a void between the concrete topping and the ceiling soffit. The concrete topping, lightly reinforced with BRC mesh, is placed on top of the mild steel expanded metal lathe.

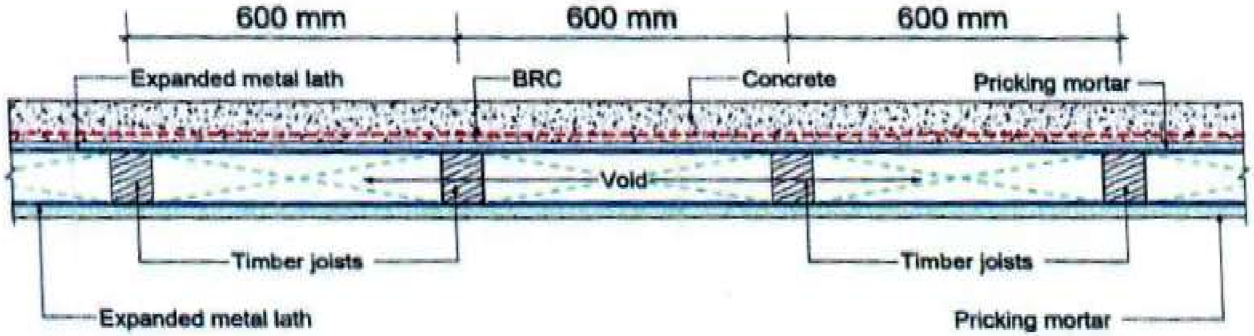

Figure 1-1: Section through typical STC Slab construction


Figure 1-2: Timber framework in typical STC Slab construction (Credit: Enock Kirabo)

Figure 1-3: STC Slab Steel-Timber layout and bottom metal lathe (Credit: Enock Kirabo)

Figure 1-4: Pricking up mortar is applied from the top (Credit: Enock Kirabo)

Figure 1-5: BRC mesh at the top and conduits underneath the mesh (Credit: Enock Kirabo)

Figure 1-6: STC Slab after construction

Of the 2,954 ongoing building operations in Kampala, Mukono, Mpigi and Wakiso monitored by the National Building Review Board from 1st July 2022 to June 30, 2023, 4.7% (138 sites) were using the unconventional Steel-Timber-Concrete 'composite' method.

1.1.3 Prohibition of the STC Method

Due to the rapid adoption of the STC method in the building sector and the several complaints received by NBRB, a study of its viability in the Ugandan context was carried out and concluded that there were significant barriers that rendered the method risky. Among the barriers identified was the lack of standards to guide the design and implementation of the method. Subsequently, the Minister of Works and Transport issued Legal Notice N0.11 of 2022 of 23rd September 2022 prohibiting the use of the STC method.

LEGAL NOTICES
SUPPLEMENT No. 9 **23rd September, 2022.**

LEGAL NOTICES SUPPLEMENT

to The Uganda Gazette No. 60, Volume CXV, dated 23rd September, 2022.

Printed by UPPC, Entebbe, by Order of the Government.

Legal Notice No. 11 of 2022.

THE BUILDING CONTROL ACT, 2013

The Building Control (Prohibition of Steel-Timber Concrete Composite Building Method) Notice, 2022

(Under section 42 of the Building Control Act, 2013, Act No. 10 of 2013)

Figure 1-7: Prohibition of the STC Method notice

Other barriers that were identified included the following:

i. No consideration of safety aspects in the design and implementation of this technology for example fire and earthquake resistance.

ii. Local timber was not graded, and therefore, the design parameters were unknown.
iii. The quality of work on all the sites inspected was lacking, and there were no quality control mechanisms in place.
iv. There was no involvement of professionals on all the sites inspected.
v. There was no skilled labour carrying out the implementation of this method.
vi. The drawings did not have approvals from both the professionals and the local authorities on all the sites inspected (the few that had drawings, the construction significantly deviated from the drawings).
vii. Lack of clarity on whether composite action was possible as marketed.

One of the recommendations after the prohibition was the formation of a Technical Committee to review the method with a view of issuing guidelines on the safe use of the method. This report is the output of the Technical Committee that carried out its work from 18th May 2023 to 30th April 2024.

1.2 Objectives of the Technical Committee

The overall objective was to "Assess the viability of the STC method, and if found viable, develop guidelines for the safe design, fabrication and erection of the structures (implementation) of STC buildings".

The specific objectives of the Technical committee were to:

a) Review the 2022 NBRB Study report and other documents to form an opinion on the viability of the STC method as marketed before the prohibition.
b) Undertake characterization of local Ugandan timber commonly used in structural members in construction in Uganda.
c) Carry out a survey of structural steel on the Ugandan Market and check compliance to current design standards.
d) Carry out a survey of concrete practices in the construction industry in Uganda to ascertain compliance with specifications for typical works.

e) Develop guidelines for the design and construction of structures with Steel, Timber and Concrete.
f) Develop guidelines for welders of structural steel works in the building construction sector.

1.3 Scope

The study involved the following key activities:

a) A desk-based examination of documents including previous research studies and existing standards; and a detailed analysis of the STC method as marketed in Uganda before the prohibition
b) Market surveys on common timber species locally available in Uganda and testing the same with a view of obtaining basic design parameters
c) Market surveys on and analysis of common practices in the use of Concrete and Structural Steel in Uganda
d) Stakeholder consultations
e) Drafting guidelines on the use of the Steel, Timber and Concrete in Slab systems in Construction

The work of the technical committee was on safety grounds, and is not primarily intended to influence registration of Intellectual Property by proponents of new methods of construction. The work of the technical committee was not restricted to any purported design by any proponent, since variants of the same had already been adopted by the industry.

1.4 Composition of the Technical Committee

The technical committee comprised professionals from various institutions and professions within the built environment and academia as shown in Table 1-1.

2 METHODOLOGY

2.1 Introduction

To answer satisfactorily the set objectives of the investigation into the STC method, the following activities were undertaken;

i. Document Review
ii. Inspection of STC construction sites
iii. Meeting with Industry Players
    a. FSSD
    b. M/s Steel and Tube Industries
    c. M/s Green Resources/Busoga Forestry Company
    d. Uganda Timber Growers Association
iv. Timber Testing
v. Tests on Structural Steel
vi. Assessing the welded steel connections on various sites
vii. Tests on Concrete Practices
viii. Structural analysis of as-built STC constructions
ix. Formulation of guidelines for construction using steel, timber and concrete
x. Stakeholder engagement workshop

2.2 Document Review

The following documents were reviewed:

- STC investigations report, 2022
- Reports from previous investigations
- Standards on the use of timber and steel in construction

2.3 STC Sites Inspection

2.3.1 Findings from site inspections

Five STC sites in Kampala City and Wakiso District were inspected by the technical committee to better understand the as-built constructions.

The main findings included:

a) The majority of the structures were Class B structures[1].

b) The construction of posts/columns involved the use of I-section steel profiles and occasionally steel hollow sections. Proponents of the technology voiced a preference for rectangular hollow sections, expressing concerns about potentially higher costs associated with the former (I-sections).

c) The sub-structure of the steel column was wrapped in a Damp Proof Membrane to prevent direct contact with the ground in some instances.

d) I-sections of varying sizes were predominantly employed in constructing the beams. Although not observed during the site visits, it was reported that C-sections were utilised by some practitioners in STC construction.

e) The steel I-sections used for beam elements had flange widths ranging from 55 to 85mm and the web depths ranging from 100 to 180mm. The predominant section was the IPE140 (with flange width of 140mm and web depth of 73mm).

f) Some sites had concrete infill in the hollow rectangular sections used as columns.

g) The sizes of the steel square hollow sections used for columns ranged between 75x75mm to 150x150mm, with general thickness of 4mm.

h) There was rust observed on most of the columns and beams.

i) There was misalignment of columns from one floor to another, with a connection detail where the upper section was sitting on a beam as

[1] Class B Structures: Residential or commercial buildings; floor area >30m^2; single or multi-storeyed of up to 12m high of simple structural form, boundary wall built of bricks, concrete or other solid material of permanent nature.

opposed to a beam connecting to a single continuous column section.

j) None of the sites had steel bracing elements.

k) In some cases, secondary steel beams were incorporated in the slabs, with the timber joists supported by the bottom flanges, with no fasteners.

l) The timber moisture content tested on site with a moisture meter was found to be way greater than 12%.

m) There was no connection between the timber floor joists and the I-section beams i.e. the timber was placed on top of the steel beams without any fastener.

n) On at least one of the sites, the floor beams had significantly deflected necessitating propping of the steel beams.

o) The timber joists being used were averaging between 75mmx100mm to 100mmx100mm as cross-sectional dimensions.

p) Many timber joints had visible attack by blue stain fungi. Blue stain reduces bending strength of timber and increases decay vulnerability.

q) Cracking and checking of timber joists were also observed in several members.

2.3.2 Site Inspection Photos

The photos in Figures 2-1 to 2-23 depict different aspects of the site observations.

Figure 2-1: Storied Primary school block in STC

Figure 2-2: Proposed Hotel Complex in STC

Figure 2-3: Storage block at a hotel


Figure 2-4: Residential apartment block

Figure 2-5: Soffits of STC slab

Figure 2-6: Propping of steel beams with timber during construction

Figure 2-7: Failed Ceiling panel

Figure 2-8: Masonry infill walls

Figure 2-9: Concrete honey combing

Figure 2-10: MEP installations

Figure 2-11: Metal lathe layer filled and eventually plastered to encase the steel sections

Figure 2-12: Construction stage leakage through slab and dampness at bottom of slab

Figure 2-13: Column loading a beam

Figure 2-14: Offset beam from column position, loading another beam

Figure 2-15: Unconventional connections

Figure 2-16: Connections with angles

Figure 2-17: No contact between beam and its supposed lower column section

Figure 2-18: Gaps left at 'connection' points

Figure 2-19: No contact between beam and its supposed lower column section

Figure 2-20: Cracking in the Concrete Slab

Figure 2-21: Superstructure to substructure connection

(a) Buckled beam web

b) *Crack along the flange line*

Figure 2-22: Early failures

(a)

(b)

Figure 2-23: (a) Joists appearing black due to blue stain, (b) Cracking and checking in timber joists

2.4 Meetings with Industry Players

2.4.1 Meeting with FSSD

The technical committee held consultations with the Forest Sector Support Department (FSSD) of the Ministry of Water and Environment (MWE) on 4th September 2023 in order to obtain insights on the challenges and strides taken in the Forest Sector (see Fig. 2-24).

FSSD is the department in-charge of Forest sector support in the MWE, providing technical backstopping to district local governments as well as hosting the National Forestry Stewardship Council (NFSC). According to FSSD, standards for soft wood and hard wood had been developed. They noted that grading of timber has an additional cost attached to it; the market was reluctant to adapt since the price of timber would escalate.

Figure 2-24: Meeting between Technical Committee and FSSD at the MWE

It was agreed that:

1. It was critical that timber is harvested at the right age, sawn using the right method and properly seasoned and treated if it is to behave satisfactorily for the purposes it is expected to be put to.

2. There is a need for multi-sectoral pressure to ensure that timber has a stamp of origin, so that quality is traceable.

2.4.2 Meeting with Steel and Tube Industries

The Technical Committee visited the Steel and Tube Industries, Kazinga branch on 16th January 2024, to look at their capacity for fabricating engineered buildings, as well as their quality of workmanship and technology especially with regards to welding (see Figs. 25 a-f).

The factory majorly imports steel plates, cuts, welds and assembles elements according to customer specifications. They are in the process of automating the welding operations, but at the moment, the welding is done manually. They currently use submerged arc welding (SMAW) and submerged arc welding / machine welding[2] (mechanized welding). The structural elements are then taken to site for installation / assembly with bolt connections.

The main components of the pre-engineered building are:

- Hot and cold formed built up structural framing members (columns and rafters)
- Cold formed 'Z' and 'C' shaped purlins and girts
- Customised roll formed sheeting profiles (roof and wall cladding)

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[2] Machine welding: Welding with equipment that requires manual adjustment of the equipment controls in response to visual observation of the welding, with the torch, gun or electrode holder held by a mechanized device.

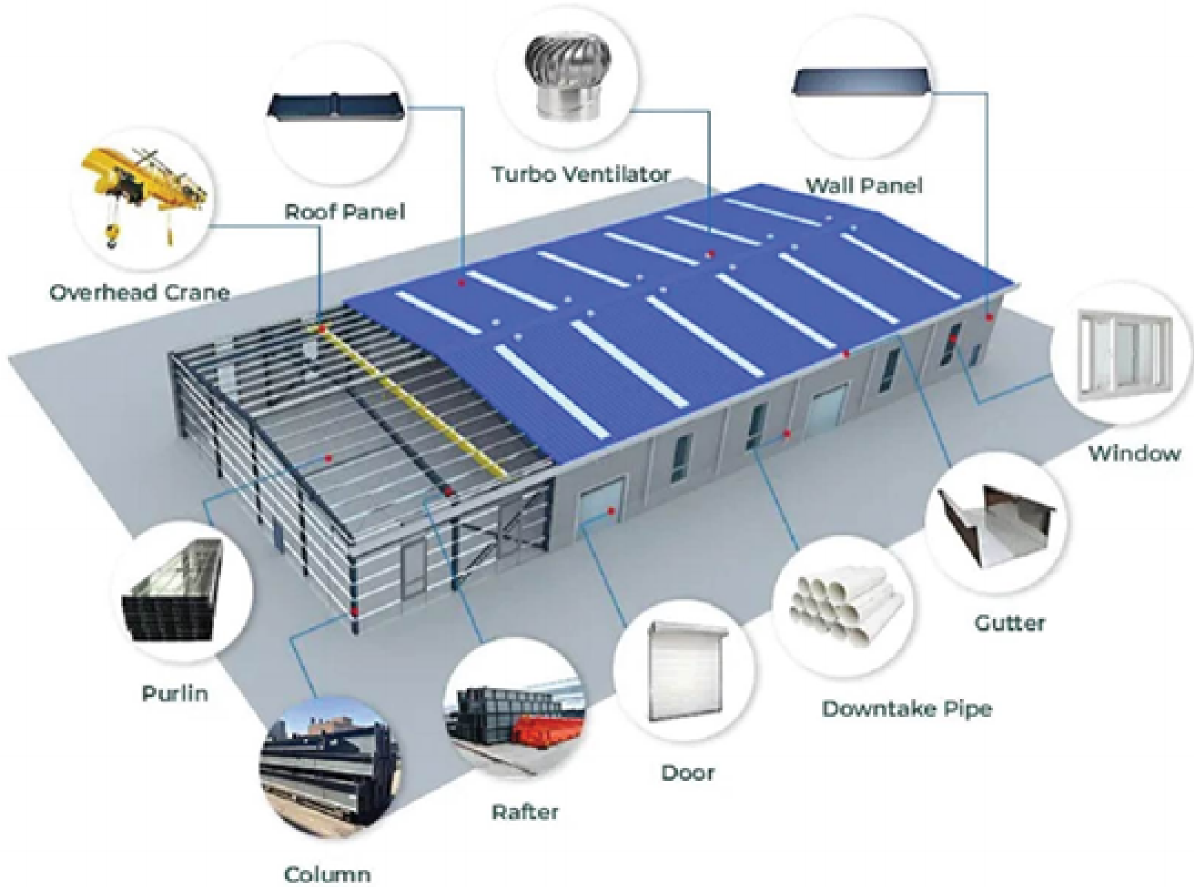
(a) (b)

(c) (d)

(e) (f)

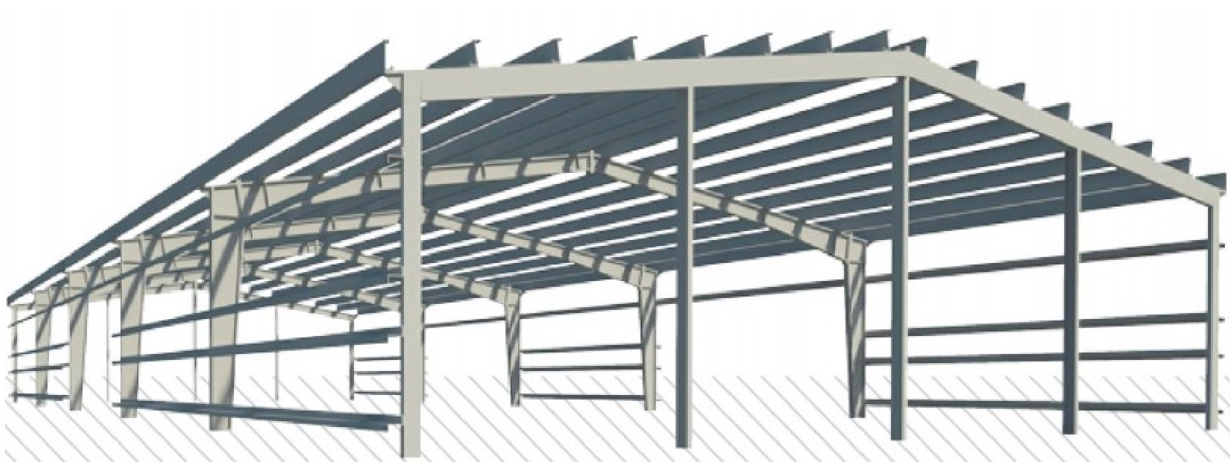
Figure 2-25: Technical Committee visit to Steel and Tube Industries

Fig. 2-26 (a-c) is an illustration of the company's pre-engineered building.

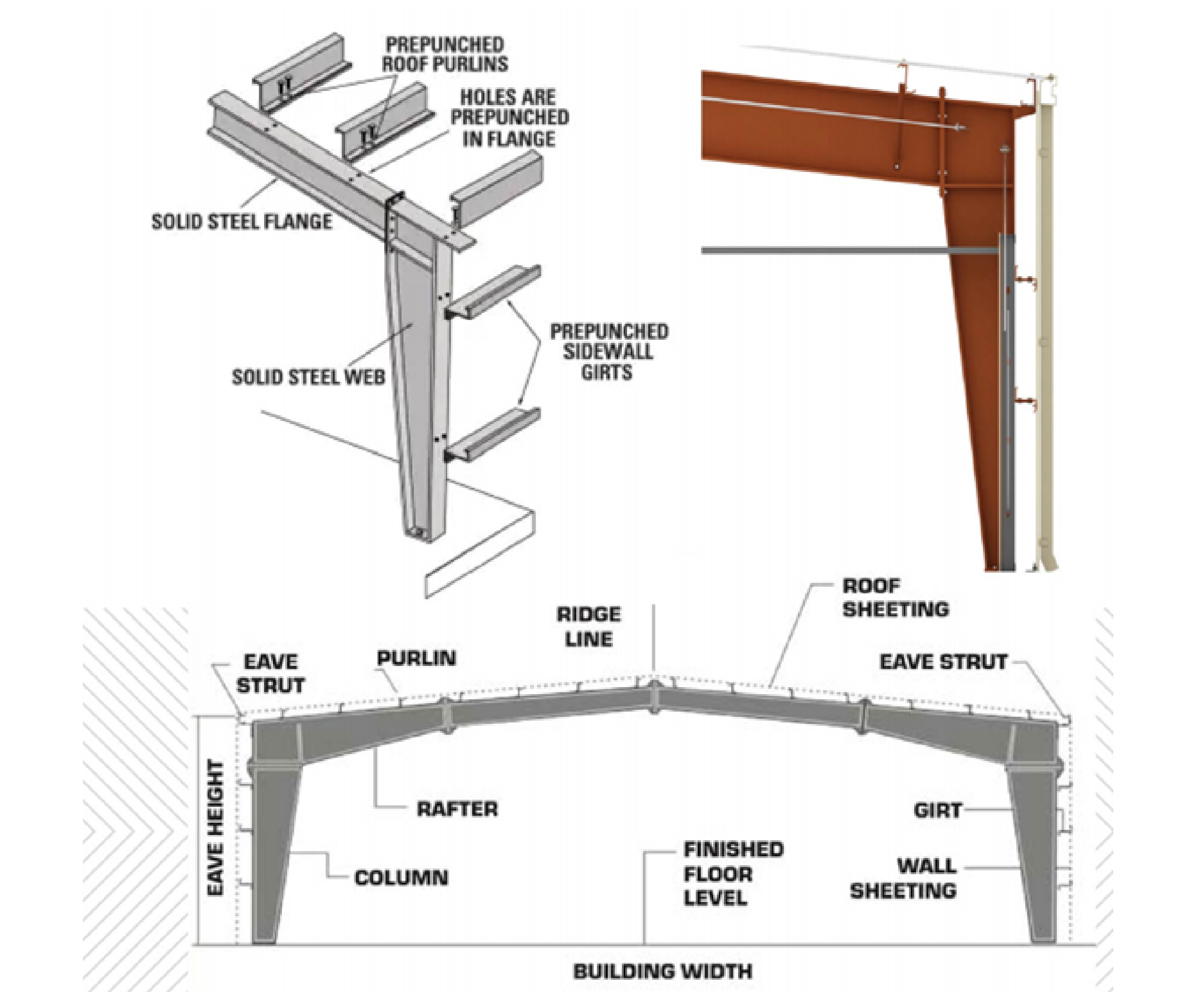


Turbo Ventilator
Wall Panel
Roof Panel
Overhead Crane
Window
Gutter
Purlin
Downtake Pipe
Door
Rafter
Column

(a)



(b)



(c)

Figure 2-26: STIL pre-engineered buildings

2.4.3 Meeting with Green Resources/Busoga Forestry Company (BFC)

The Technical committee visited the Busoga Forestry Company's milling plant in Mayuge on 6th February 2024 and noted the following:

a) BFC grows trees commercially and has 6,500 hectares of plantation in Mayuge District, mainly of eucalyptus, pine and a few of musizi.
b) The timber is kiln dried to a moisture content of 12-15% and for clients who prefer treatment before delivery, the timber is CCA treated.

c) The company deals in both structural and industrial timber (<2 inches or 50mm thick).
d) The company uses Tanzanian standards for their grading and outsources laboratory tests to the Wood science laboratory at CAES, Makerere University.
e) The timber is not marked. The company has two distribution locations i.e. Mukono-Seeta and Jinja. They claimed it was easy to track their timber because of their sawing.
f) They have a treatment plant at Masese, Jinja, majorly for electric transmission poles. This pole treatment plant is certified by UNBS.
g) They are Forest Stewardship Council (FSC) certified. FSC Standards are the world's leading standards for responsible forest management.
h) Their products include: -
    - CCA Treated poles (utility poles for power and telecom distribution, as well as fencing poles)
    - Structural timber (majorly for roofing and formwork)
    - Value added timber (for flooring, ceiling, wall cladding, decking and joinery works)
    - Certified heat-treated timber (for food-grade wooden pallets)
    - Sustainable wood mass (firewood, dry wood chippings, wood chips and sawdust)
    - Tree seedlings
i) They generally harvest their timber at the age of 10 to 20 years
j) There was structured/deliberate quality assurance at the factory as compared to other timber yards.

Fig. 2-27 (a-e) show the Technical Committee visiting various timber processing areas of the factory.

(a)

(b)

(c)

(d)

(e)

Figure 2-27: Technical Committee Visit to BFC

2.4.4 Meeting with Uganda Timber Growers Association (UTGA)

The Technical Committee interacted with Uganda Timber Growers Association representatives on 15th January 2024. UTGA is an association of commercial tree growers whose mandate is to bring together all the tree growers and lobby for the government to support them. The association has 750 members. They mainly plant eucalyptus and pine, especially the commercial species; 75% of the growers' plants are on government (NFA) land.

As an association, UTGA monitors the wood value chain i.e. establishment of the growers, harvesting of the trees, marketing and processing. They as well monitor products of harvested trees i.e. timber and other secondary products like ceiling boards. The two parties discussed the future outlook of use of timber in construction, agreeing that it is preferable to use certified timber that is well seasoned, preserved and therefore is more durable. They noted that there is a need to advocate for kiln drying of timber and at the moment, there are three kilns available-at New Forest Company, Green Resources, and UTGA.

UTGA noted that with the recent curriculum review by Nyabyeya Forestry College and planned acquisition of new machinery, the skill level in the use of timber and training of the next generation would be fast tracked. UTGA has also got a grant to skill the carpenters in the field to get modern skilling and training.

UTGA noted some challenges in the timber industry, including:

a) Designers and the engineers limit their choice to a few genus and species.
b) Timber grading was not being done properly as well as wood drying, wood storage and wood preservation.
c) Quality assurance tests were not being conducted well.
d) The current quality standards are voluntary standards and certification

is optional which is not helping in supporting uptake of timber in construction.

2.5 Timber Testing

2.5.1 Species Selection

For this study, wood samples of five common timber species used for construction were collected from the Bwaise and Ndeeba timber hotspots, which are two of the busiest timber selling points in Kampala. The timber species that were mainly traded in, at the two timber hotspots, for both construction and furniture use and which were tested are: *Eucalyptus* spp (eucalyptus); *Albizia Coriaria* (Mugavu); *Pinus* spp (Pine); *Markhamia lutea* (Musambya); and *Maesopsis eminii* (Musizi)

2.5.2 Determination of Moisture Content

One of the most important variables influencing the performance of wood is its moisture content (MC). Moisture content of timber is water present in the wood divided by the mass of the wood with no moisture in it (W_0). The amount of water present not only influences its strength, stiffness, and mode of failure, but it also affects its dimensions, shrinkage and swelling, its susceptibility to fungal attack, its workability as well as its ability to accept adhesives and coatings.

For wood to perform well, its moisture content must be reduced to a level corresponding to the equilibrium MC of the place where it is expected to be used, about 12% for use indoors. Schedule 16, Part II of the National Building (Structural Design) Code, 2019, specifies that the moisture content of timber in buildings should be in the range12-15% in its permanent position and 15-22% at the time of erection. Prior to mechanical testing, the MC of the wood samples was, therefore, measured following the BS 373 standard protocol. The moisture content (MC) was calculated using the formula in equation (2.1).

$$MC\ (\%) \quad = \frac{W_a - W_o}{W_0} \times 100 \ \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad (2.1)$$

where Wa =mass of timber in its normal state, W_0 = mass of dry timber

Figure 2-28: Moisture content determination using the oven dry method setup

Perhaps the single most important property controlling the mechanical performance of a piece of wood is its density (Dinwoodie & Desch, 1996). Basic Density is the ratio of its oven dry weight to its green volume. Generally, the density of wood is correlated to its mechanical properties in that as density increases, the strength increases. The green volume (G_v) was determined from the samples of size 20 x 20 x 20 mm (BS 373:1957) as shown in Figure 2-29.

The test specimens were then oven-dried at 105±1 °C until constant weight (OD_w). The mean basic density, ρ_d was calculated from equation (2.2) and its characteristic 5-percentile value, $\rho_{k.05}$ obtained using equation (2.3), where *s* is the standard deviation. (EN 384:2004)

$$\rho_d \ = \frac{OD_w}{G_v} \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad (2.2)$$

$$\rho_{k.05} = (\rho_d - 1.65 * s) \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad (2.3)$$

Figure 2-29: Green Volume determination

2.5.3 Preparation of Samples for Mechanical Properties Testing

The specimens were resized according to the test method requirements (BS EN 384: 2004). The details for the sample dimensions (BS 373 (1957)) are shown in Table 2-1. A minimum of 20 test pieces were tested for physical properties and a minimum of 6 test pieces for each species were analysed for strength and stiffness properties.

Table 2-1: Sample dimension for timber strength and density analysis

Timber Properties	**Dimension** (Length x Width x Thickness in mm)
Mechanical Properties (Bending and stiffness)	1000 x 50 x 50
Moisture content	50 x 50 x 50
Basic density	20 x 20 x 20

2.5.4 Mechanical Properties (Bending Strength and Stiffness) Testing

The mechanical properties of wood are crucial in assessing its overall relative strength and its propensity to deform under load. The mechanical property testing was carried out using a T-Olsen Universal Testing Machine (UTM) at the Wood Science Laboratory in the School of Forestry, Environmental and Geographical Sciences of Makerere University. The UTM was calibrated to a maximum loading of 50kN which suits the strength class of wood. The static bending tests were performed in a four-point bending apparatus for stiffness properties analysis as well as three-point bending for strength analysis as shown in Figure 2-30.

Figure 2-30: UTM Ste up for 4-point and 3-point loading test

The strength properties based on Modulus of Rupture (MOR) and Stiffness properties based on Modulus of Elasticity (MOE) of the timber were determined following the guidelines of the British Standard (BS EN 408:2003) at a loading rate of 6.6 mm per minute (Dinwoodie, 1981). The MOR and MOE were measured directly from a computer with specialised software (Q-mat) connected to the UTM.

2.5.5 Modulus of Elasticity (MOE) and Shear Modulus

The mean characteristic value of MOE parallel to grain (Eqn. 2.4) was obtained using sample values of MOE auto generated then exported from Q-mat software connected to the UTM. Other stiffness characteristic values such as the 5th percentile of MOE parallel to grain and mean MOE perpendicular to the grain were derived following the guidelines of British Standard (BS EN 384:2004) and using equations (2.5) to (2.6).

$$\boldsymbol{E}_{0.\ \text{mean}} = \frac{\sum E_j * nj}{\sum nj} \text{ .... } (2.4)$$

Where: E_j is the mean value of modulus of elasticity for sample j expressed in Newtons per square millimetre (N/mm^2);

n_j is the number of specimens in sample j;

$$\boldsymbol{E}_{0.05} = 0.67 * E_{0.\ \text{mean}} \text{ (softwood)................ } (2.5)$$

$$\boldsymbol{E}_{90,\ \text{mean}} = 0.84 * E_{0.\ \text{mean}} \text{ (hardwood)........... } (2.6)$$

$$\boldsymbol{E}_{90,\ \text{mean}} = E_{0.\ \text{mean}} / 30 \text{ (softwood).......... } (2.7)$$

$$\boldsymbol{E}_{90,\ \text{mean}} = E_{0.\ \text{mean}} / 15 \text{ (hardwood)......... } (2.8)$$

The mean shear modulus G_{mean} was derived from the equation (2.9).

$$\mathbf{G}_{\text{mean}} = E_{0.\ \text{mean}} / 16 \text{ .... } (2.9)$$

2.5.6 Modulus of Rupture (MOR) and Shear Strength

The characteristic value of bending strength, f_k (equation 2.10) was obtained using the adjusted 5th percentile ($f_{0.5}$) values of MOR auto generated then exported from Q-mat software connected to the UTM. (EN 338, 2009 quoted by Idris & Muhammad, 2013)

$$f_k = \frac{\sum f_{0.5} * n}{n} \text{ .... } (2.10)$$

2.5.7 Tension, Compression and Shear Strength

The characteristic values of tensile strength parallel to grain, $f_{t.0.k}$; compressive strength parallel to grain, $f_{c.0.\ k}$, and shear strength, $f_{v.k.}$ for softwood species, were calculated using equations 2.11, 2.12 and 2.13, respectively.

$$f_{t.0.k} = 0{,}6 f_k \text{ .... } (2.11)$$

$f_{c.0.k} = 5(f_k)^{0,45}$.. (2.12)

$f_{v.k} = min.\{3.8, 0.2(f_k)^{0.8}\}$ .... (2.13)

The characteristic values of both tensile strength perpendicular to grain, $f_{t.90.k}$, and the compressive strength perpendicular to grain, $f_{c.90.k}$ were calculated using the adjusted density values as shown in Equations (2.14), (2.15) and (2.16).

$f_{t.90.k}, = min.\{0.6\ , 0.0015\rho_k\}$ .... (2.14)

$f_{c.90.k} = 0{,}007\rho_k$ (softwoods) .... (2.15)

$f_{c.90.k} = 0{,}015\rho_k$ (hardwoods) .... (2.16)

2.6 Tests on Concrete and Assessment of Common Practices

2.6.1 Objective and Scope

The Committee investigated the effects of different material sources/material types on the strength properties of concrete. Six different locations around Uganda were selected and sampled. The objective of this investigation was to conduct a quick survey, establishing the effects of varying materials types and sources on the strength of concrete, and investigating common behavioural practices around concrete application.

The scope of investigation on concrete practices was limited to concrete testing of mixes that utilised locally available materials in the given selected regions of Uganda.

2.6.2 Constraints and Assumptions

The following assumptions were made and constraints noted during the execution of this assignment:

i) The samples were taken across the country, and fairly represented the spatial disparity of aggregate rock.

ii) Cement type CEM IV was mostly used.

iii) Project focus was mainly on the attainment of concrete strength and not on the quality of the constituent materials of sand, aggregate and water.

iv) Only concrete grade of C20/ 25 was investigated, which is the commonest and minimum grade for structural applications.

v) A Water-Cement ratio of 0.5 was used for the different mixes.

2.6.3 Work Approach

The detailed methodology of the tasks is summarised in Table 2-2.

Table 2-2: Summary of work approach for concrete subcommittee

ACTIVITY	**METHODOLOGY**
1. REPORT REVIEWS	
a) Review	• NBRB shared reports on collapsed buildings around Uganda • Subcommittee reviewed reports with an emphasis on the concerns on concrete highlighted in the report • Subcommittee reviewed 2022 STC Report from NBRB
b) Site Visits	Conducted Site Visits to sample STC buildings
2. CONCRETE MIX ASSESSMENTS	
a) Defining Study area	Research on the materials' prevalence in the regions of Uganda was done and zones created according to the material prevalence for the research study. Six sites were identified and investigated.
b) Sourcing for aggregates	Borrow pits/Material sources were identified in the defined regions and their location pin, location name, photos of samples captured.
c) Identification & collection of samples (sand & aggregates)	Samples were collected from the identified borrow pits and organised for transportation to the laboratory for testing.
d) Preparation of trial mixes using CEM IV & CEM II	Concrete trial mixes were conducted for design mixes and concrete cubes prepared. Some ratios were adopted from already existing sites as a quality check whereas others were designed by the committee, all aimed at Concrete grade C20/25.

ACTIVITY	METHODOLOGY
e) Tests on Materials	• The sand and aggregate tests were conducted simply to give an indication of the quality of the materials and not necessarily as a quality control measure. • The verification of the quality of cement used relied on the UNBS Q mark given to the manufacturer.
f) Concrete Tests	Cube Samples were prepared and cured on site as per the BS 1881 • After 7 and 28 days (14 days for some), respectively, the cured samples were taken to the laboratory for compressive strength testing. • Slump tests were also conducted for some sites
3. CONCRETE PRACTICES RESEARCH	
Assessment of common concrete practices	Research & Assessment of field practices on the application of Concrete aided by a questionnaire and observations

2.6.4 Concrete Mix Assessments

Selection of sites

The selection of the sites was purposively done based on the team's access to active sites in these regions. Construction sites selected were located in the following selected regions around the country:

- Yumbe (North West)
- Luweero (Central)
- Lira (North)
- Fort Portal (West)
- Tororo (East)
- Mengo-Kampala (Central)

Materials Sourcing

The sites sampled were using materials (coarse aggregates, sand, water, cement) sourced from various locations within the region as summarised in Table 2-3.

Table 2-3: Site and construction material selection

S/N	LOCATION	REGION	SITE COORDINATES	WATER SOURCE	CEMENT TYPE	FINE AGGREGATE (LAKE SAND) SOURCE	COARSE AGGREGATE SOURCE	BUILDING FUNCTION
1	Luweero	Central	0.937509, 32.473870	NWSC	CEM II	Katosi	Zirobwe	Vocational Institution
2	Mengo, Kampala	Central	0.308022, 32.554521	NWSC	CEM IV	Lwera	Muyenga Quarry	Shopping Mall
3	Fort Portal	West	0.656466 30.279678	NWSC	CEM IV	Kibuku	Kasese	Health facility
4	Lira	North	2.253682, 32.902926	NWSC	CEM II	AFRICAN CONMAT	Lira Quarry	Health Facility
5	Yumbe	North West	3.248208, 31.311211	Surface Water	CEM IV	River Ntegero	Imvempi Camp	Treatment Plant
6	Tororo	East	0.693822, 34.179050	NWSC	CEM IV	Kuluku Village	Logululе Quarry	Storage Warehouse

Sample Collection & Classification

Figures 2-31 and 2-32 show the materials used for the tests and their classification.

a. Luweero; Well graded

b. Mengo-Kampala; Well graded

c. Fortportal; Well graded

d. Lira; Well graded

e. Yumbe; Well graded

f. Tororo; Well graded

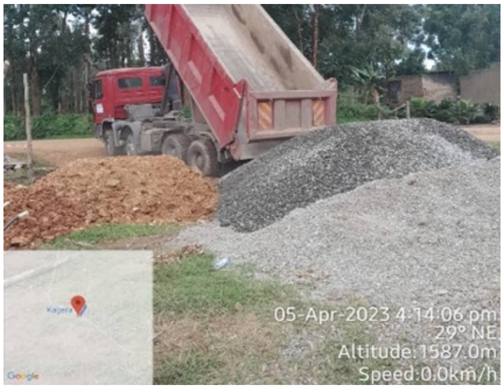
Figure 2-31: Sand types used and their gradation for concrete experiments

a) Luweero: *Machine crushed, Well graded aggregates from Zirobwe*

b) Kampala (Mengo): *Machine crushed, Well graded aggregates from Muyenga*

c) Fort Portal: *Machine crushed, Well graded aggregates from Kasese*

d) Lira: *Machine crushed Well graded, aggregates from AFRICAN CONMAT*



e) Yumbe: *Machine crushed, Well graded aggregates from Imvepi Refugee Camp*

f) Tororo: Machine *crushed, Well graded aggregates from Logulule Quarry*

Figure 2-32: Coarse aggregate type used and their gradation for concrete experiments

Concrete Mix Designs Used

Trial mixes for Concrete C20/25 Mix Designs were done. Machine mixing was used for all the mixes. The Concrete mixes were batched by volume. Table 2-4 summarises the concrete mix designs that were tested as converted to equivalent batch box mix ratios.

Table 2-4: Mix design ratios for concrete experiments

	LOCATION	MIX DESIGN RATIOS USED (BY VOLUME)	EQUIVALENT MIX RATIOS	CEMENT TYPE USED	SIZE OF BATCH BOX USED	AMOUNT OF WATER USED
1.	LUWEERO	1 bag of cement: 3:4	1:2.3:3.1	CEM II	300x300x300	20-22 Ltrs of water
2.	KAMPALA	1 bag of cement: 2:4	1:1.5:3.1	CEM IV	300x300x300	20 Ltrs of water
3.	FORTPORTAL	1:1.5:3	1:1.5:3	CEM IV	300x300x400	30 Ltrs of water
4.	LIRA	1:2:4	1:2:4	CEM IV	300x300x400	Approximately 30 Ltrs
5.	YUMBE	1 bag of cement: 3:4	1:2.3:3.1	CEM IV	300x300x300	20-25 litres
6.	TORORO	1:1.5:3	1:1.5:3	CEM IV	300x300x400	25 -30 Ltrs

It was noted that the concrete users tend to prefer to calibrate their mix design ratios against a bag of cement, for easier handling of the cement. For such sites, the committee converted them to the equivalent mix ratios that use the same size of batching box. Two sizes of batch boxes of 0.027m^3 and 0.035m^3

were predominantly used i.e.300mm by 300mm by 300mm and 300mm by 300mm by 400mm. It should be noted that one bag of cement is equivalent to of 0.035m^3.

Compressive Strength Testing

A minimum of three Cube samples were prepared and tested per trial mix. The procedure of testing concrete was guided by BS1881. Samples were prepared and cured on site and later taken to designated laboratories for testing at a minimum of 7 days and 28 days. The laboratories in Table 2-5 were used for testing.

Table 2-5: Laboratories used for concrete experiments

Test site	**Laboratory used**
1. Luweero	Teclab Limited
2. Mengo	Teclab Limited
3. Fort Portal	MoWT Fort Portal Regional Materials Laboratory
4. Lira	MoWT Gulu Regional Materials Laboratory
5. Yumbe	MoWT Arua Regional Materials Laboratory
6. Tororo	MoWT Mbale Regional Materials Laboratory

Slump Testing:

Slump testing was done for a few select sites

Figure 2-33: Slump test for concrete experiments

It was observed that sites whose slumps were within acceptable limits (50-100mm)[3] consistently exhibited compressive strengths that achieved the desired target strength. This was attributed to the acceptable water–cement ratios in these scenarios.

2.6.5 Survey on Common Practices in Concrete

To investigate common concrete practices, a questionnaire was designed to guide the interviews of the different select sites. A total of 15 sites were chosen at random. The sites were active multi storey construction sites ranging from 2-storeys to 5-storeys.

2.7 Tests on Structural Steel

Tensile and chemical tests were conducted on selected samples to check on their compliance to standards and also provide the required data for modelling of the STC components.

The assumption was made that the steel products commonly used in construction are typically purchased randomly from hardware stores and sites in and around Kampala. To capture a diverse range of samples, offcuts of steel sections were acquired from hardware stores and sites around Kampala. The selection aimed to include products from various manufacturers and suppliers but focusing on sections with a high prevalence in STC construction, as additionally reported by the proponents.

The tests conducted on the steel sections only considered the cheapest sections available on the market, with disregard for the design specifications, if any. The Subcommittee considered that the fact that the overriding factor in selection of material in Uganda is the cost. The sampled steel products underwent testing for both mechanical and chemical properties. These tests were conducted at the materials laboratory of the Mechanical Engineering

[3] BS EN 206-1 2006 and BS 8500: Part 1:2002, Table A.7

Department at Makerere University and at the Uganda National Bureau of Standards Laboratory.

Each sample was measured to verify its dimensions, a full list of which is shown in Table 2-6. Subsequently, a test-piece was cut from each sample; specifically, from the web in case of an I-member. The coupons were then subjected to testing using a computerised Testometric Universal Testing Machine.

Table 2-6: Steel testing sample IDs

S/N	Sample designation	Testing Laboratory
1	SHS 75x75x3 mm	Makerere/CEDAT/Mechanical
2	SHS 75x75x4 mm	Makerere/CEDAT/Mechanical
3	SHS 75x75x4 mm	Makerere/CEDAT/Mechanical
4	SHS 75x75x4 mm	Makerere/CEDAT/Mechanical
5	SHS 75x75x3 mm	Makerere/CEDAT/Mechanical
6	SHS 75x75x4 mm	Makerere/CEDAT/Mechanical
7	SHS 100x100x3 mm	Makerere/CEDAT/Mechanical
8	SHS 100x100x3 mm	Makerere/CEDAT/Mechanical
9	SHS 100x100x3 mm	Makerere/CEDAT/Mechanical
10	SHS 100x100x4 mm	Makerere/CEDAT/Mechanical
11	SHS 100x100x4 mm	Makerere/CEDAT/Mechanical
12	SHS 100x100x4 mm	Makerere/CEDAT/Mechanical
13	I-beam 160x85x4 mm	Makerere/CEDAT/Mechanical
14	I-beam 140x86x4 mm	Makerere/CEDAT/Mechanical
15	I-beam 120x65x3 mm	Makerere/CEDAT/Mechanical
16	I-beam 140x75x4 mm	Makerere/CEDAT/Mechanical
17	I-beam 100x75x4 mm	Makerere/CEDAT/Mechanical
18	I-beam 120x65x4 mm	Makerere/CEDAT/Mechanical
19	I-beam 120x65x4 mm	Makerere/CEDAT/Mechanical
20	CHS 100x3 mm	Makerere/CEDAT/Mechanical
21	CHS 100x4 mm	Makerere/CEDAT/Mechanical
22	CHS 75x3 mm	Makerere/CEDAT/Mechanical
23	CHS 76x5 mm	Makerere/CEDAT/Mechanical
24	CHS 100x4 mm	Makerere/CEDAT/Mechanical
25	U-section 100x50x4 mm	Makerere/CEDAT/Mechanical
26	I-beam 87.5x56x4 mm	Makerere/CEDAT/Mechanical
27	I-beam 87.5x56x4 mm	Makerere/CEDAT/Mechanical
28	I-beam 87.5x56x4 mm	Makerere/CEDAT/Mechanical

S/N	Sample designation	Testing Laboratory
29	CHS 100x3 mm	Uganda National Bureau of Standards
30	SHS 100x100x4 mm	Uganda National Bureau of Standards
31	SHS 75x75x3 mm	Uganda National Bureau of Standards
32	Channel	Uganda National Bureau of Standards
33	I-beam 102x60x4 mm	Uganda National Bureau of Standards
34	I-beam 118x68x4 mm	Uganda National Bureau of Standards

2.8 Weld Connections Tests

The inspection was done in accordance with AWS D1.1 2020 – Structural Welding Code - Steel, ASME Section V 2007 – Non-Destructive Examination. Probity Engineering and Investment Limited was engaged to carry out Non-Destructive Test Inspections (NDT) that included Visual Inspection of welds, Liquid Penetrant Tests (DPT) and assessment for suitability for Ultrasonic flow detection of welds (UTI).

The standards referenced in the welding tests are listed below:-

- AWS D1.1 2020 – Structural Welding Code – Steel
- ASTM E165 – Test Method for Liquid Examination
- ASME V – Article 6 – Liquid Penetrant Examination
- ASME V – Article 4 – Ultrasonic Examination Methods for Welds
- ASME V – Article 9 – Visual Examination

3 STC METHOD REVIEW

3.1 Review of the NBRB Study Report of 2022

The technical committee reviewed the NBRB Study report of 2022; observed that the report was well written, very methodical, and the purpose for which the investigation was set-up was justified.

Following the review, the committee noted that:

a) The observed defects and the lack of professionalism in the construction, for example the detail of a column supported by a beam rather than the column section underneath are sufficient to have the buildings condemned as unsafe and illegal.

b) Steel Beam to timber connections: The timber joists did not have anchors to the steel beams, it was evident that what the proponents specify verbally is completely different from what is observed from the field.

c) The timber joists are made in the same way as for common plastered ceilings. However, these timber ceilings do not carry loads on a daily basis, apart from self-weight. Just nailing secondary to primary joists may not be sufficient to support a floor slab carrying gravity load every day; bolting and use of gusset plates may be needed.

d) The BRC is placed at a uniform depth, however in design, it should be closer to the bottom of the slab in the spans and closer to the top at the supports.

e) The depth of the concrete topping and BRC to use should be based on design for the moment and checks for shear and deflection of the slab panels.

f) Although proponents/practitioners construct what they claim to be composite steel-timber-concrete slab, there is hardly any composite action, due to absence of shear connectors and the method should not be referred to as such, since it breeds confusion within the engineering fraternity.

g) Theoretically, there are two alternatives to designing the STC system:

- Where there is assumed composite action between the timber & concrete, and steel & concrete (where steel directly supports the concrete slab). In this case, shear connectors would be needed.
- Each material acts independently. In this case, the engineer must answer the questions: *Can the concrete slab panel support the loads applied to it? If these loads are transferred to the timber, can the timber system support them? Can the steel beams support the loads applied on them? Can the column support the loads from the steel beams framing into it?*

h) A number of misrepresentations were made by the technology proponents. Any proponent should be aware that structural inadequacies cannot be hidden and gotten away with because the forces of nature do not respect impressive presentations.

i) The purported inclusion of Polythene between the concrete slab and the timber framing seems reactive and ad hoc rather than based on systematic research and tests, hence casting doubt on the reliability of the STC construction method.

j) The method does not address Durability concerns; issues to do with Timber strength stability overtime (rotting), fungal and insect attack; Slab protection against moisture, did not seem to have been given thought by the proponents. It is impossible to check the state of the timber after covering it up. It could suffer from fungal and insect attack without anyone knowing and continue to support loads up to the time of the catastrophic collapse.

k) MEP design integration is a challenge; electrical and plumbing installations, especially given the timber framework in use as part of the slab.

l) The STC structural system doesn't appear to be adequately engineered. It is unlikely that any designs against earthquake loads were ever considered for the STC constructions. The absence of an explicit lateral load resisting system is clearly a major omission. It is conceivable that the presence of numerous walls within the structure undertakes this crucial function unintentionally, even though they might not have been explicitly designed for such a purpose.

m) The strength of timber needs to be known before design. For this system to work, timber sellers should be required to attach a certificate on their timber consignment certifying the following:
   - The species of the timber, and the age
   - Certifying that proper seasoning has been carried out
   - The strength class of the timber

n) One area among others that needs research is the performance of the STC system during and after a fire outbreak.

3.2 Review of Steel-Timber-Concrete Composite Constructions World Wide

Composite constructions are not a novel idea. For hundreds of years, combinations of materials have been used in construction. In Africa, the commonest composite constructions are Steel-Concrete. In other parts of the world, concrete-timber composite constructions have existed for centuries.

According to Cuerrier-Auclair (2020), Timber-Concrete Composites (TCCs) have been in use since 1910's. What is essential for optimal performance of these systems is effective connection between the two materials and the indispensable need for shear connectors. Given that in composite constructions, the mechanical properties of the two or more materials are used efficiently, these composites can prove to be cost effective and efficient functionally. For example, the TCCs are expected to have a better acoustic performance compared to timber-only floors because of the additional mass the concrete affords to the set up.

Steel-Timber composite constructions have also been used in the world, with the commonest being the use of steel sections as the main framing elements and the timber as the flooring with shear connectors joining the two materials.

The American Institute of Steel Construction (AISC) published a report in 2017 as part of their Steel & Timber Research for High-Rise residential buildings for a structural system that consisted of structural steel framing and composite cross laminated timber (CLT) floor planks. In the system tested:

i. The CLT planks are typically 8 inches thick and are connected to each other to form a mass-timber floor. *This is unlike the STC method where discrete joists are placed typically 600mm from each other orthogonally, with the space in between filled with metal lathe and pricking up mortar at the bottom and at the top, and the topping is BRC mesh-reinforced concrete slab.*
ii. The CLT planks are topped with 2.5 inches thick concrete topping slab to enhance durability, acoustics and fire resistance of the floor framing system.
iii. Composite action between the topping slab and the mass-timber floor planks is achieved using structural screws designed to provide composite behaviour. *There are hardly any shear connectors in the STC method which makes composite action very unlikely.*
iv. The topping slab is reinforced to make the composite floor system continuous over steel beam supports. *In the STC method, the BRC reinforcement also enables continuity over the timber joists but is only located at one position, near the bottom and therefore hogging moments are not accounted for. This is a critical omission given that for STC, given that the concrete slab is a structural element subject to flexural stresses.*

v. The steel beams are asymmetrical in order to simplify the placement of the timber decks on the bottom flanges. The mass-timber floor planks are notched at the bearing location to create a flat soffit condition with the bottom flange of the steel beams. *In the STC method, the beams are symmetrical making it a concern on whether there is adequate bearing length for configurations where the joists are seated on the bottom flanges.*

Figure 3-1: Steel-CLT timber system soffit (Credit: AISC, 2017)

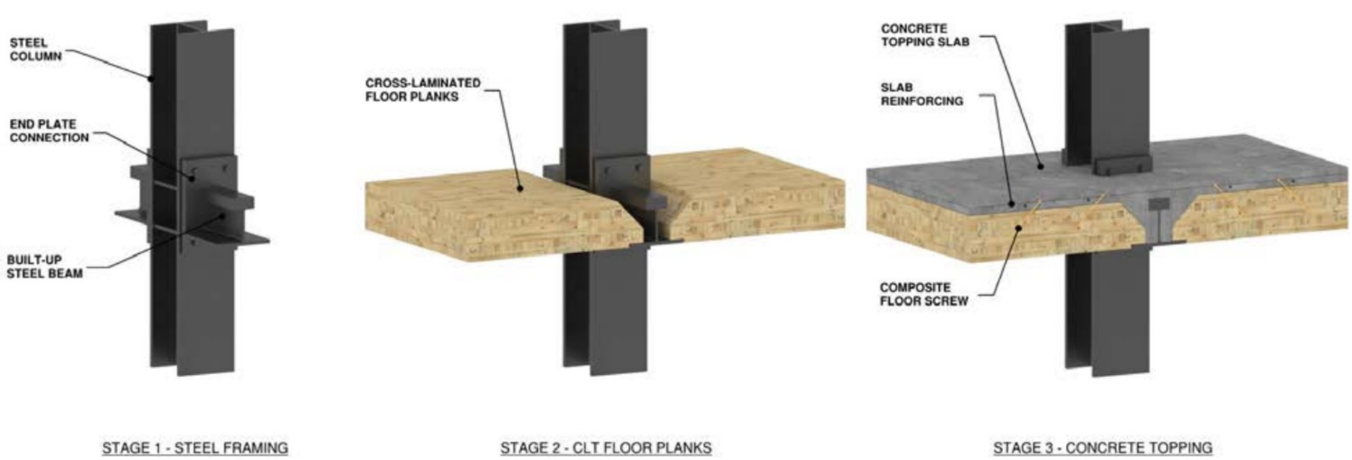


Figure 3-2: AISC Steel-Timber system construction stages (Credit: AISC, 2017)

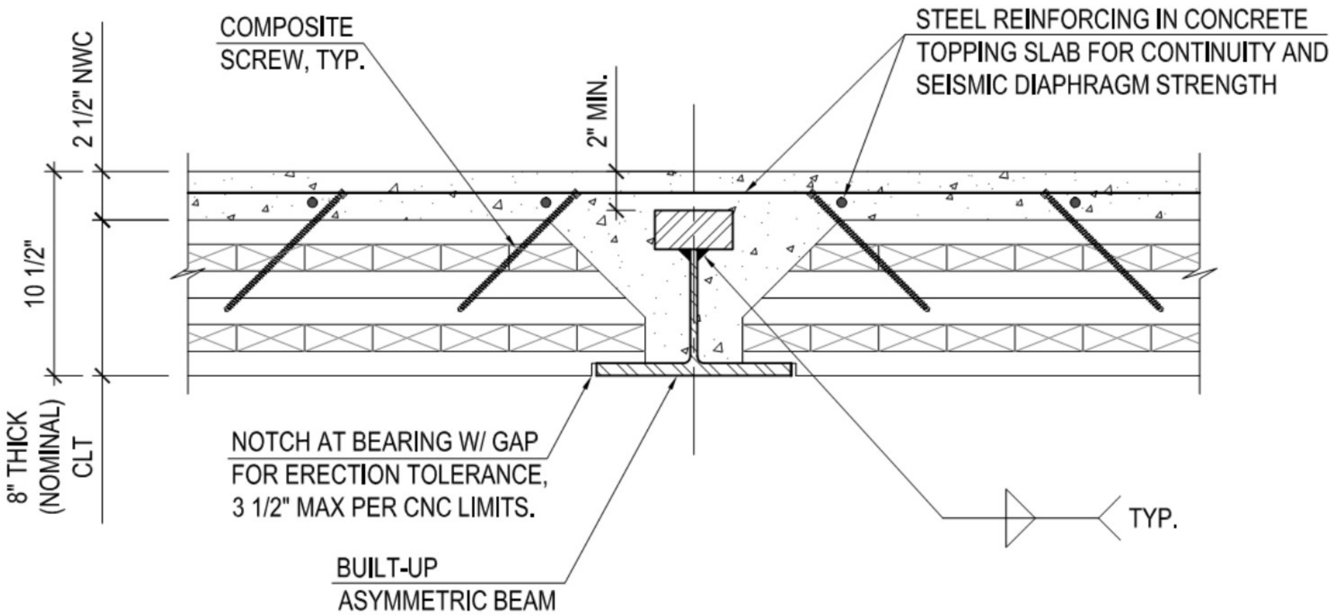


Figure 3-3: Typical composite timber deck and steel beam detail (Credit: AISC,2017)

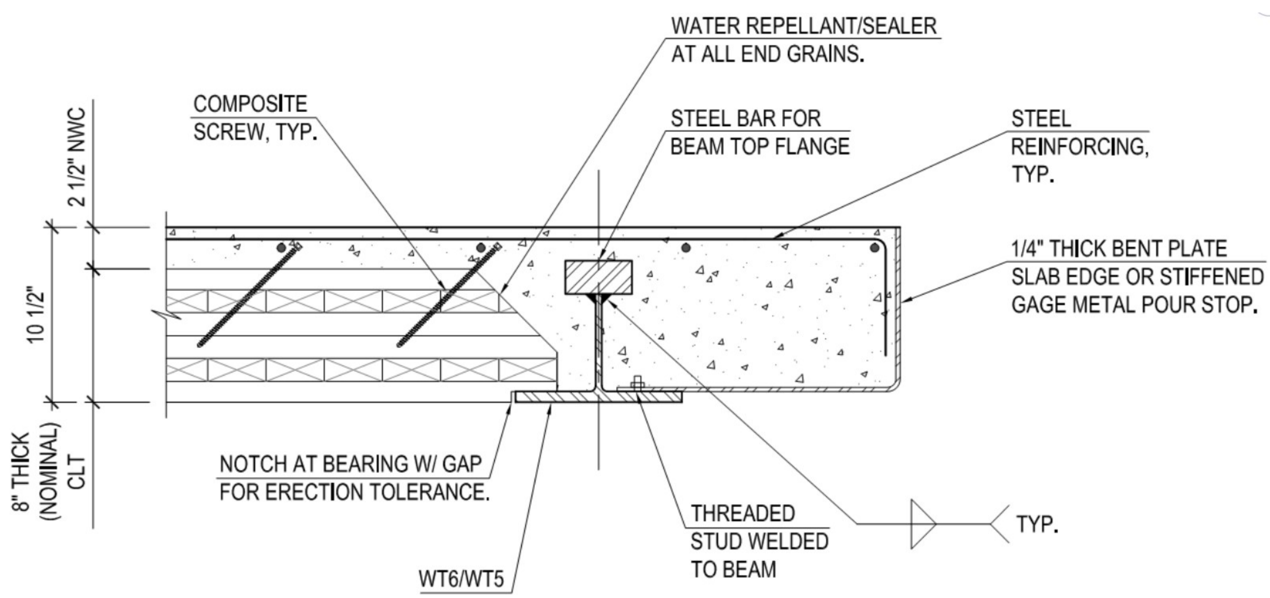


Figure 3-4: Typical Floor Connection detail at the edges (Credit: AISC, 2017)

vi. The beams are designed as composite with the concrete topping; composite action is provided by shear connectors to avoid testing but if testing is available to confirm composite behaviour, this can be achieved by concrete bonding to the steel beam. *There are no shear connectors in the STC constructions inspected.*

vii. The lateral load resisting system comprises braced frames. *In most of the STC constructions, braces are not provided for, with the only possible bracing obtainable from the masonry infills, which are hardly anchored into the main framing elements.*

viii. The concrete topping slab has a top surface treatment to avoid exposing the timber to moisture long term. *In STC, there is no moisture protection, which exposes the timber to water through the pricking up mortar, leakages in plumbing, etc.*

ix. The steel and mass-timber building elements are prefabricated and can be constructed faster than a concrete framed building. This leads to cost saving associated with reduced project time. *In STC construction, the majority of connections are welds on site and fabrication is largely done at site.*

x. The weight of the system is approximately 65% of a comparable concrete framed building (AISC, 2017). This has an effect on reduced foundation costs, seismic loading, and construction time, which are all advantageous in construction.

xi. According to the report, pricing information suggested that the cost for the system would be comparable to a concrete framed structure (within 10%).

3.3 Forms of the STC Method in Uganda

It was clear from the inspections that what the main proponent of the STC method (Nana Corps Ltd) claimed to be the method was at variance in the majority of cases with what was actually implemented in the field.

There were two prevalent configurations of the STC on the market, specifically with regard to Steel beam to Timber connections.

a) The timber joist sat atop the steel beams, in which case the concrete slab sat on timber joist. In this case the load transfer was clear; slab to timber to steel beams and to the columns.

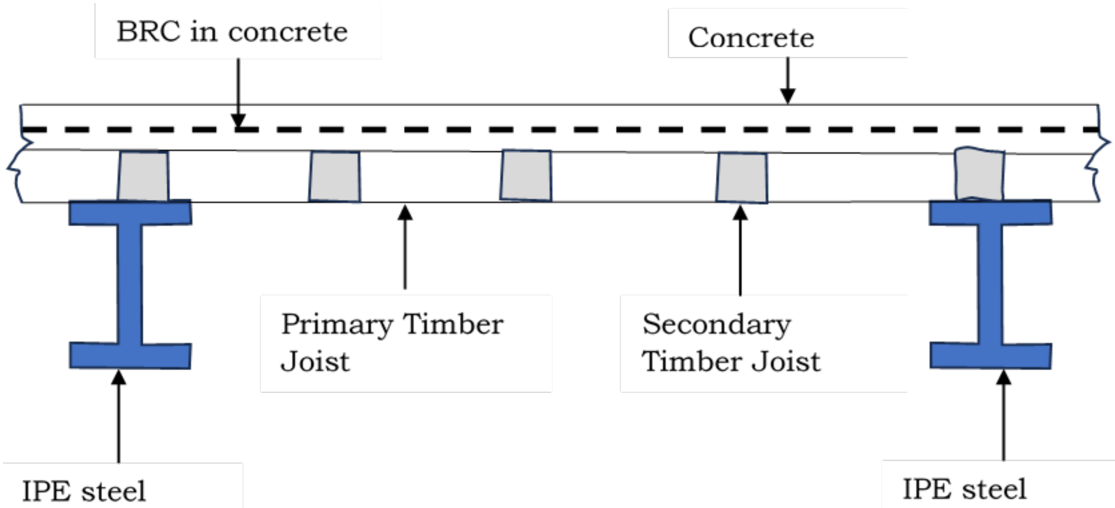


Figure 3-5: STC Configuration 1 (As designed)

b) Where the timber joists were supported on the bottom flange of the steel beams and therefore the concrete slab sat on both the timber joists and the steel beams at different sections.

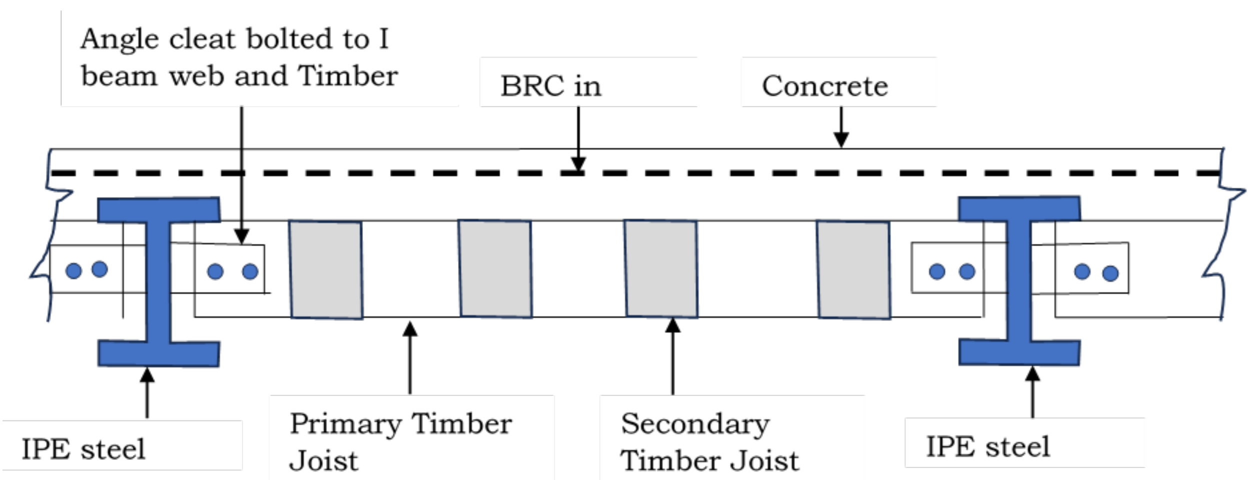


Figure 3-6: STC Configuration 2 (as designed)

From the inspections however for Configuration 2, the majority of the construction appear as shown in Figure 3-7.

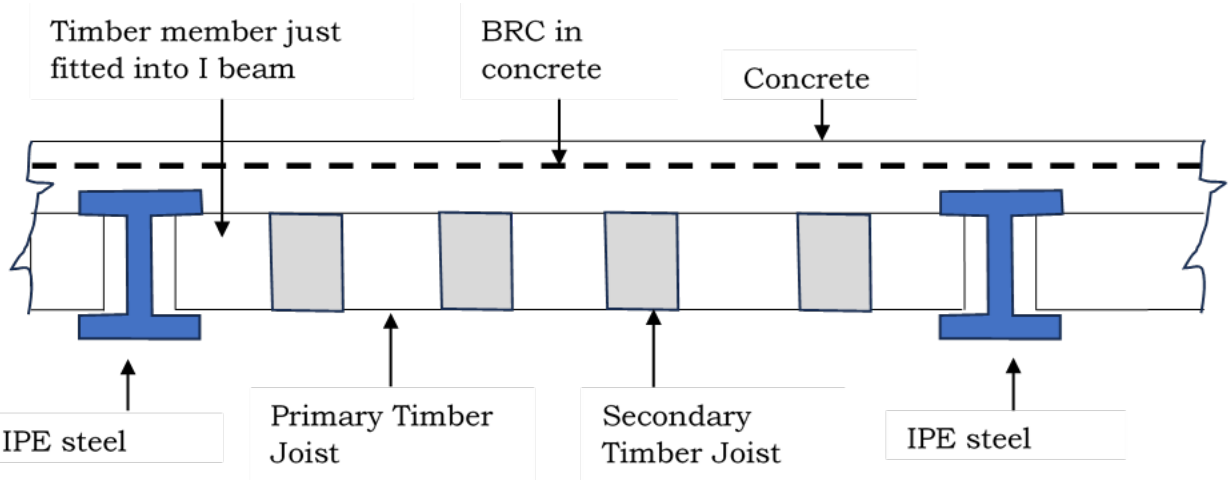


Figure 3-7: STC Configuration 2 (As Constructed)

3.4 Existence or Absence of Composite Action

In structural engineering, composite construction exists when two **different materials are bound together so strongly that they act together as a single unit from a structural point of view.** When this occurs, it is called **composite action.** Such arrangements seek to combine their different properties to provide a more effective overall solution. For example, if the beam is not connected firmly to the slab, then the slab transfers all of its weight to the beam and the slab contributes nothing to the load carrying capability of the beam. However, if the slab is connected positively to the beam with studs, then a portion of the slab can be assumed to act compositely with the beam. The two share the load, not just one transferring to another.

The conventional Steel-Concrete composite floors are partially covered by the NBC (SD 2019) Par.62 and Par.63. For composite action, shear connectors would have been needed. In all sites inspected, there was no evidence of shear connectors being used. Even without shear connectors, some composite action could have been generated due to friction between concrete and timber members, albeit accidental. However, any chance of composite action is eliminated by the polythene placed on top of timber to prevent ingress of moisture from concrete onto the timber.

The Timber-Steel connections are also largely pin connections, with no fastener/connector in between the two materials.

Figure 3-8: Timber framework simply placed on top of the steel beams (Credit: Enock Kirabo)

3.5 Weld Connections

The weld connections were very poor visually, and in the majority of cases, tack welding was used, which is not acceptable for structural purposes.

The weld testing report for the sample sites noted that the majority of welds were unacceptable. The welds were undersized, some had pin holes, no capping, lack of fusion at some points, poor weld profile, others were not fully welded, and others not welded at all (see Figs. 3-9 and 3-10).

Figure 3-9: Poorly welded joints with inconsistency in weld profile

Figure 3-10: No fusion between the base metal and the angle bar

The poor weld profiles, undersized welds, craters and other joints not being welded fully or at all is an indication that welders lack training and pre-qualification.

Of the 132 welds tested, 85% were rejected as unacceptable and should be redone.

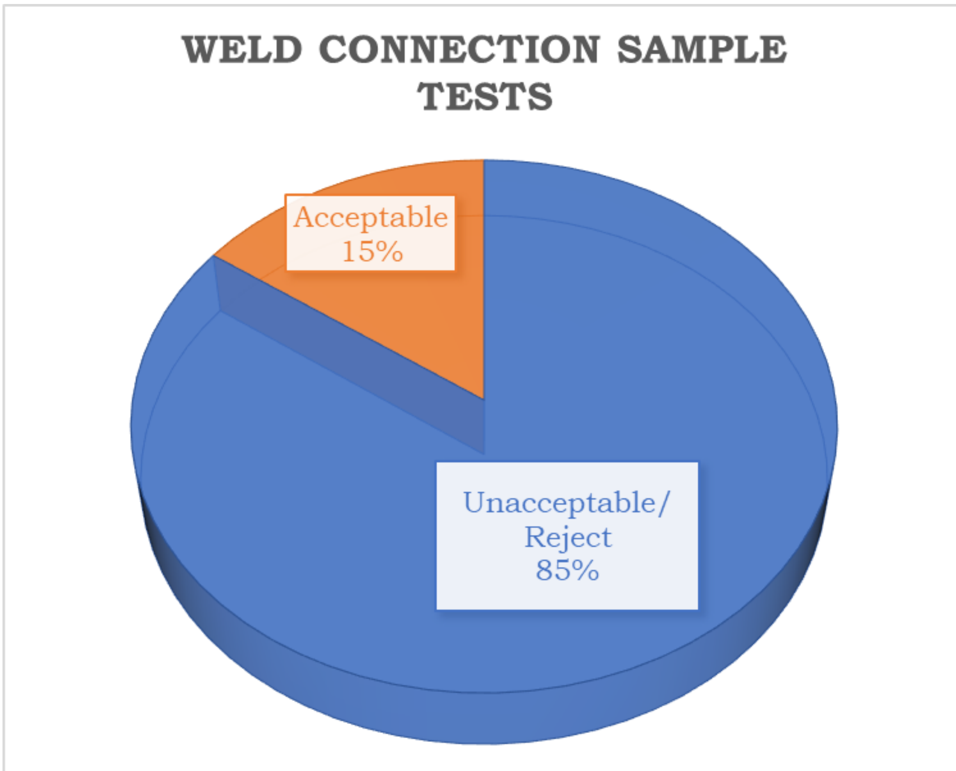


Figure 3-11: Welds that were acceptable vs rejects

3.6 Assessment of the Viability of the STC method as Marketed in Uganda

3.6.1 Desk Technical Review

The commonest configuration for STC slabs in Uganda is as illustrated in Figure 3-12.

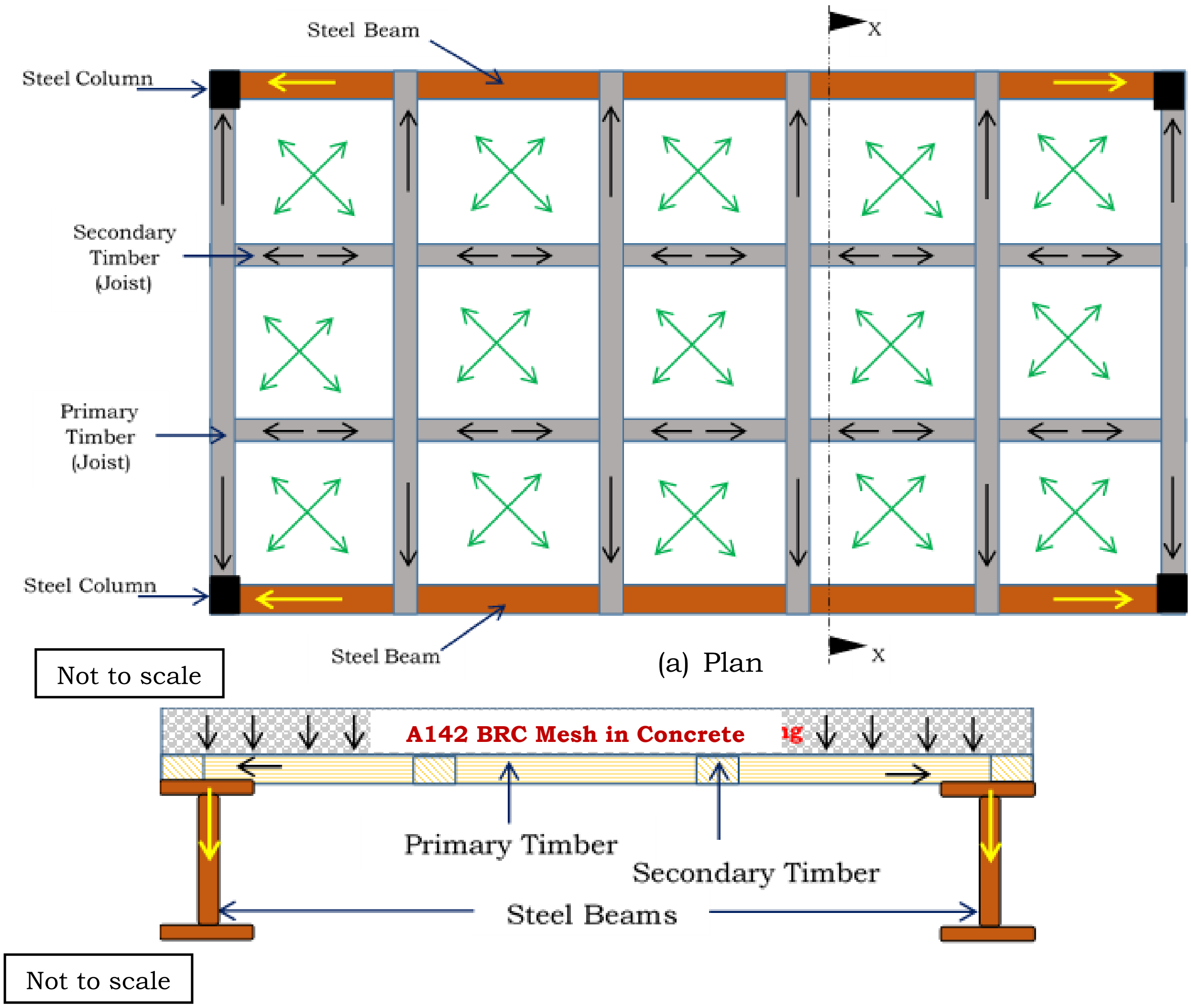


Figure 3-12:Idealised layout for STC slab (Configuration 1)

3.6.2 Checking the Performance of Timber Beams in STC

A check on the performance of timber beams in one of the structures using the STC building system was carried out. The check was based on EN 1995. The shortest timber beam was checked for its adequacy in Bending, shear, Bearing, and Deflection. Pine (strength class C14) was adopted. **It was found to be inadequate in bending, shear and deflection.**

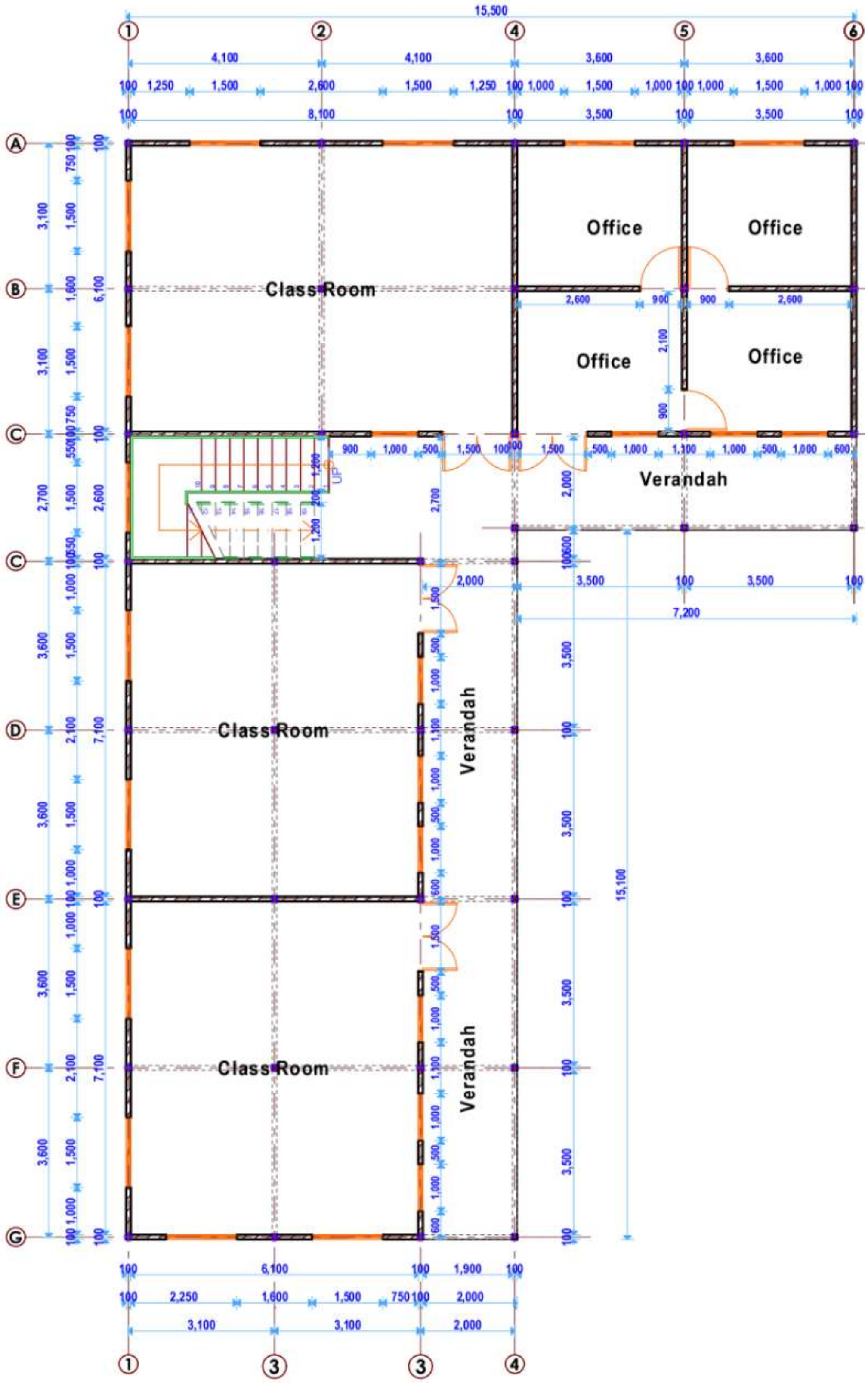


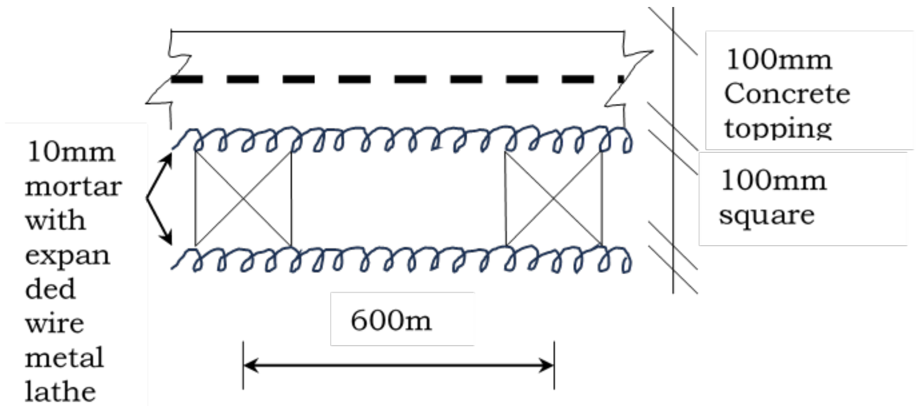
Figure 3-13: Layout of STC floor for a primary school

(a) Floor panel width b=3.5m, design span = 1.9m

(b) Joists are spaced at 600mm

(c) The joists are 100mm thick x100mm deep

(d) The steel beams were 140mm deep x 75mm wide and the flange thickness $t_f = 4.0mm$. For Configuration 1, it may be assumed to be full width of steel member, i.e. 75mm



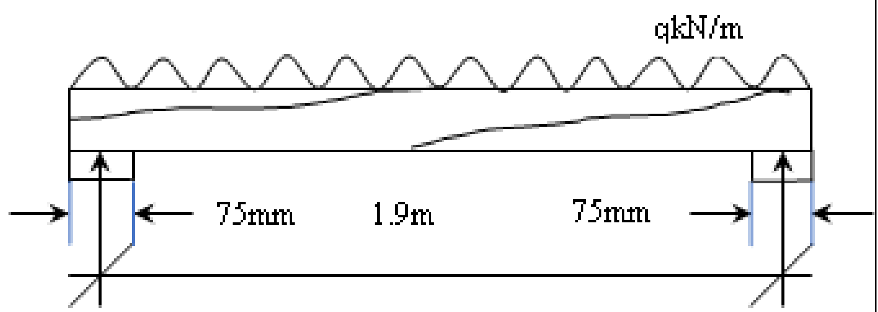
Typical X section

Permanent Actions

100mm concrete topping (assuming unit weight of concreter is $25kN/m^3$ =0.1x1x25 = $2.5kN/m^2$

Assuming Pinus (as tested); Density =$495kg/m^3$

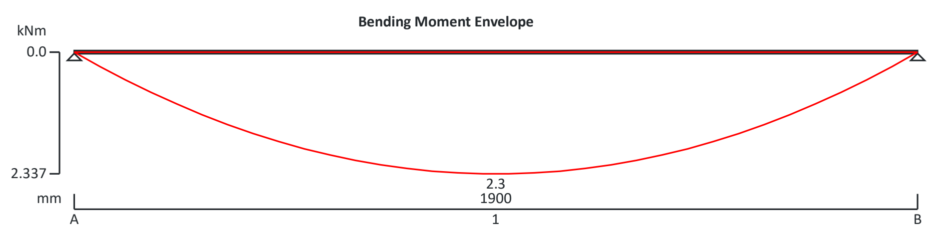
REF	DISCUSSION / CALCULATIONS	OUTPUT
	• Weight of joists = $2 * \left(\frac{Density\ \ x\ \ Area}{spacing\ x\ 1000}\right) x9.81$ $= 2 * \left(\frac{495x(0.1x0.1)}{0.6x1000}\right) x9.81 = 0.16kN/m^2$ • Pricking up mortar under the joists (assumed to be 10mm thick) with a unit weight of $20kN/m^3$ = 0.01x2=$0.2kN/m^2$ • Underside plaster ((assumed to be 10mm thick) with a unit weight of $20kN/m^3$= 0.01 x 20=$0.2kN/m^2$	

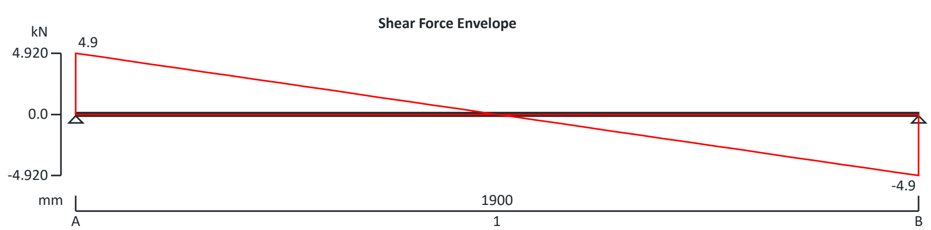
Table 6.2; EN1991-1	**Permanent Action** (G_k) **= 2.5+ 0.2+0.16+0.2=**$3.06kN/m^2$ **Variable action** Q_k **for Schools (Category C1)** $Q_k = 3.0kN/m^2$ **Design Action q** $q = 1.35G_k + 1.5Q_k$ $q = 1.35x3.06 + 1.5x3 = 8.6kN/m^2$ Action supported by each joist $q = 0.6x8.6 = 5.2kN/m$	G_k**=** $3.06kN/m^2$ Q_k=3 kN/m^2 Design Action $q = 5.2kN/m$
	 	

TIMBER BEAM ANALYSIS & DESIGN TO EN1995-1-1:2004

In accordance with EN1995-1-1:2004 + A2:2014 and Corrigendum No.1 and the UK National Annex incorporating National Amendment No.1

Tedds calculation version 1.7.04



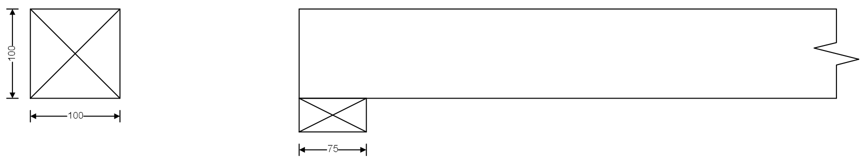


Applied loading

Beam loads

Permanent full UDL 1.836 kN/m

Variable full UDL 1.800 kN/m



Timber section details

Breadth of timber sections; $b = \mathbf{100}$ mm

Depth of timber sections; $h = \mathbf{100}$ mm

Number of timber sections in member; $N = \mathbf{1}$

Overall breadth of timber member; $b_b = N \times b = \mathbf{100}$ mm

User defined timber properties

Characteristic bending strength; $f_{m.k} = \mathbf{15.7}$ N/mm^2

Characteristic tensile strength parallel; $f_{t.0.k} = \mathbf{9.4}$ N/mm^2

Characteristic tensile strength perpendicular; $f_{t.90.k} = \mathbf{0.4}$ N/mm^2

Characteristic compressive strength parallel; $f_{c.0.k} = \mathbf{17.3}$ N/mm^2

Characteristic compressive strength perpendicular; $f_{c.90.k} = \mathbf{1.9}$ N/mm^2

Characteristic shear strength; $f_{v.k} = \mathbf{1.8}$ N/mm^2

Mean modulus of elasticity parallel; $E_{0.mean} = \mathbf{7100}$ N/mm^2

5% modulus of elasticity parallel; $E_{0.05} = \mathbf{4800}$ N/mm^2

Mean modulus of elasticity perpendicular; $E_{90.mean} = \mathbf{240}$ N/mm^2

Mean shear modulus; $G_{mean} = \mathbf{440}$ N/mm^2

Characteristic density; $\rho_k = \mathbf{277}$ kg/m^3

Mean density; $\rho_{mean} = \mathbf{353}$ kg/m^3

Member details

Load duration - cl.2.3.1.2; **Medium-term**

Service class of timber - cl.2.3.1.3; **3**

Length of span; $L_{s1} = \mathbf{1900}$ mm

Length of bearing; $L_b = \mathbf{75}$ mm

In accordance with cl.6.6 the member is one of several similar and equally spaced members laterally connected by a continuous load distribution system capable of transferring loads from one member to the neighboring members.

Section properties

Cross sectional area of member; $A = N \times b \times h = \mathbf{10000}$ mm^2

Section modulus; $W_y = N \times b \times h^2 / 6 = \mathbf{166667}$ mm^3

$W_z = h \times (N \times b)^2 / 6 = \mathbf{166667}$ mm^3

Second moment of area; $I_y = N \times b \times h^3 / 12 = \mathbf{8333333}$ mm^4

$I_z = h \times (N \times b)^3 / 12 = \mathbf{8333333}$ mm^4

Radius of gyration; $r_y = \sqrt{(I_y / A)} = \mathbf{28.9}$ mm

$r_z = \sqrt{(I_z / A)} = \mathbf{28.9}$ mm

Partial factor for material properties and resistances

Partial factor for material properties - Table 2.3; γ_M = **1.300**

Modification factors

Modification factor for load duration and moisture content - Table 3.1 k_{mod} = **0.650**

Deformation factor for service classes - Table 3.2; k_{def} = **2.000**

Depth factor for bending - exp.3.1; $k_{h.m} = \min((150\text{ mm} / h)^{0.2}, 1.3)$ = **1.084**

Depth factor for tension - exp.3.1; $k_{h.t} = \min((150\text{ mm} / \max(b, h))^{0.2}, 1.3)$ = **1.084**

Bending stress re-distribution factor - cl.6.1.6(2); k_m = **0.700**

Crack factor for shear resistance - cl.6.1.7(2); k_{cr} = **0.670**

Load configuration factor - exp.6.4; $k_{c.90}$ = **1.000**

System strength factor - cl.6.6; k_{sys} = **1.100**

Lateral buckling factor - cl.6.3.3(5); k_{crit} = **1.000**

Compression perpendicular to the grain - cl.6.1.5

Design compressive stress; $\sigma_{c.90.d} = R_{A\_max} / (N \times b \times L_b)$ = **0.656** N/mm^2

Design compressive strength; $f_{c.90.d} = k_{mod} \times k_{sys} \times k_{c.90} \times f_{c.90.k} / \gamma_M$ = **1.045** N/mm^2

$\sigma_{c.90.d} / f_{c.90.d}$ = **0.628**

PASS - Design compressive strength exceeds design compressive stress at bearing

Bending - cl 6.1.6

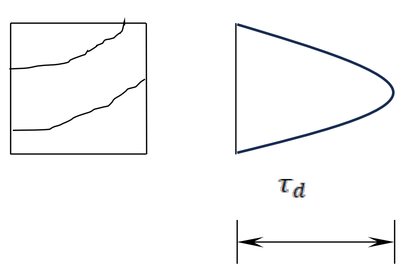
Design bending stress; $\sigma_{m.d} = M / W_y$ = **14.021** N/mm^2

Design bending strength; $f_{m.d} = k_{h.m} \times k_{mod} \times k_{sys} \times k_{crit} \times f_{m.k} / \gamma_M$ = **9.364** N/mm^2

$\sigma_{m.d} / f_{m.d}$ = **1.497**

FAIL - Design bending stress exceeds design bending strength

Shear - cl.6.1.7



Stress Distribution across a Rectangular Timber Section

Applied shear stress; $\tau_d = 3 \times F / (2 \times k_{cr} \times A)$ = **1.101** N/mm^2

Permissible shear stress; $f_{v.d} = k_{mod} \times k_{sys} \times f_{v.k} / \gamma_M$ = **0.990** N/mm^2

$\tau_d / f_{v.d}$ = **1.113**

FAIL - Design shear stress exceeds design shear strength

Deflection - cl.7.2

Deflection limit; $\delta_{lim} = \min(14\text{ mm}, 0.004 \times L_{s1})$ = **7.600** mm

Instantaneous deflection due to permanent load; δ_{instG} = **5.492** mm

Final deflection due to permanent load; $\delta_{finG} = \delta_{instG} \times (1 + k_{def})$ = **16.475** mm

Instantaneous deflection due to variable load; δ_{instQ} = **5.384** mm

Factor for quasi-permanent variable action; ψ_2 = **0.6**

Final deflection due to variable load; $\delta_{finQ} = \delta_{instQ} \times (1 + \psi_2 \times k_{def})$ = **11.845** mm

Total final deflection; $\delta_{fin} = \delta_{finG} + \delta_{finQ} =$ **28.319** mm

$\delta_{fin} / \delta_{lim} =$ **3.726**

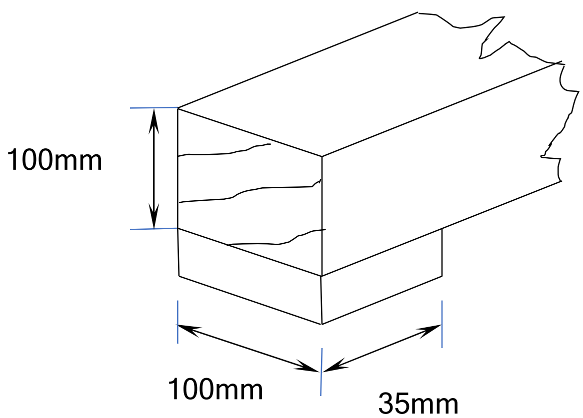
FAIL - Total final deflection exceeds the deflection limit

The effect of excessive deflection was seen during the site visits to this school, hoop iron was attached and stretched across the floor as remedial measure to prevent excessive deflection.

With the 1.9m span failing under the loading, it is obvious that the longer spans would fail as well.

DISCUSSION / CALCULATIONS
Checking for Adequacy to resist bearing stresses if the beam is supported on the bottom flange

Design bearing force = Design Shear Force



Compression perpendicular to the grain - cl.6.1.5 (BS EN 1995-1-1)

Design compressive stress; $\sigma_{c.90.d} = R_{A\_max} / (N \times b \times L_b) =$ **1.380** N/mm^2

Design compressive strength; $f_{c.90.d} = k_{mod} \times k_{sys} \times k_{c.90} \times f_{c.90.k} / \gamma_M =$ **1.045** N/mm^2

$\sigma_{c.90.d} / f_{c.90.d} =$ **1.320**

Compression perpendicular to the grain - cl.6.1.5

Design compressive stress;$\sigma_{c.90.d} = R_{A\_max} / (N \times b \times L_b) =$ **1.406** N/mm^2

Design compressive strength;$f_{c.90.d} = k_{mod} \times k_{sys} \times k_{c.90} \times f_{c.90.k} / \gamma_M =$ **1.045** N/mm^2

$\sigma_{c.90.d} / f_{c.90.d} =$ **1.345**

FAIL - Design compressive stress exceeds design compressive strength at bearing

One would need a wider bottom flange for this configuration to avoid failure in bending

Lateral-Torsional Buckling

There would be no need to check lateral buckling of beams if:

(a) Twisting of members at the supports was prevented. But in the case where the beams are just placed on the bottom flanges of the steel section, there is a need to check for lateral torsional buckling.

(b) Lateral displacement of the compression edge is prevented over the whole length of the beam. Shear connectors such as nails or screws with half their depth in timber and the other half in concrete would serve to prevent lateral displacement of the compression edge. However, in this case these are not provided, hence the need to check for lateral buckling.

Since the beam fails in bending, it follows automatically, that it would fail in lateral torsional buckling.

3.6.3 Stability-Lateral Resistance

The absence of an explicit lateral load resisting system is evident in the STC method as employed in Uganda. It is conceivable that the presence of numerous walls within the structure undertakes this crucial function unintentionally, even though they might not have been explicitly designed for such a purpose.

3.6.4 Fire Resistance of Timber in the STC Floor System

Construction works must be designed and built in such a way, that in the event of an outbreak of fire:

a) the load bearing resistance of the construction can be assumed for a specified period of time;
b) the generation and spread of fire and smoke within the works are limited;
c) the spread of fire to neighbouring infrastructure is limited;
d) the occupants can leave the works or can be rescued by other means;

e) the safety of rescue teams is taken into consideration.

The STC Floor System and Fire Exposure

The STC floor system has wood as the major component supporting the floor, yet timber is material that is susceptible to failure under fire. However, it is to be noted that the timber sizes (100mmx100mm) used in the STC system, may take some time before finally being weakened by fire to the point of collapse.

In an investigation of the STC slab performance under action of fire at Kyambogo University (courtesy of Dr Michael Kyakula), an STC slab was constructed as shown in Figure 3-14.

Figure 3-14: Underside of the STC Floor system investigated for fire resistance

The slab was subjected to a natural fire using wood as fuel. The experiment lasted 280 minutes. The fire exhibited a three-phase fire behaviour that is to say ignition–smouldering, flash over point leading to heating phase and lastly cooling phase. The average maximum temperature recorded was 734^0C , nearly equal to the maximum temperature obtained from Eurocode parametric curve of 728^0C. It is to be noted that the structure did not collapse after 3 hours of heating or 2 hours of cooling. Thus, the STC structure may be safe for 2 hours allowing for evacuation. The charred remains of the STC slab underside are shown in Figure 3-15.

Figure 3-15: Charred Timber of STC floor system after the fire

3.6.5 Beam to Column Connections

Apart from poorly welded joints, the deviation from best practice of a continuous column (with beams connected as shown in Figure 3-16 (A)), Figure 3-16 (B) which is existent in the majority of constructions clearly weakens the connection and may result in failure of the beam (by web buckling) at the support due to loads from the column.

With regard to steel-beam to timber beam connections, in one of the common configurations, especially those that incorporate secondary steel beams, the timber is just fitted inside the I-section in actual construction, while in the configuration where timber runs over the steel beams, it is not clear how the timber is connected to the I Section (if any) in actual construction. In either case this lack of proper anchoring of the timber will lead to failure under lateral loads.

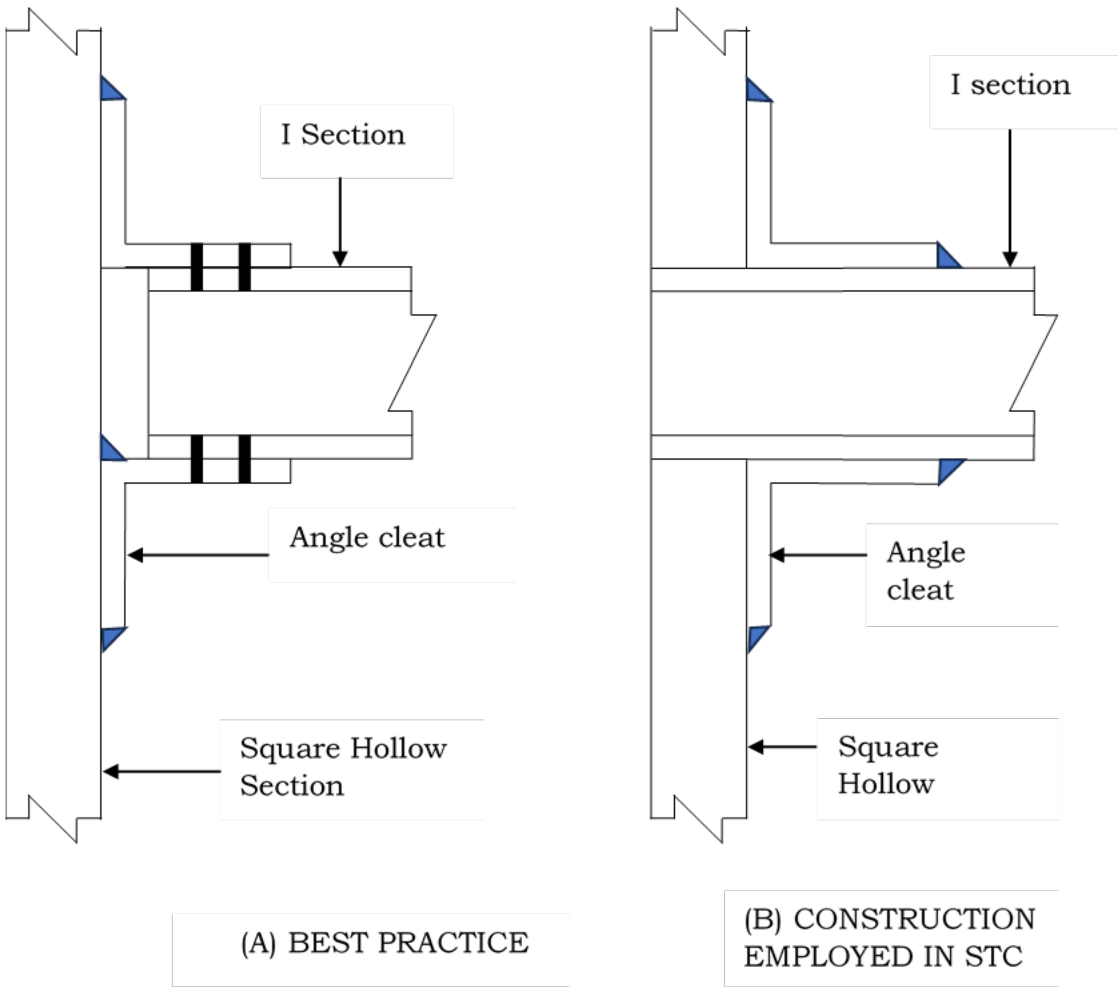


Figure 3-16: Beam to column connection: Best Practice Vs Construction Employed in STC

3.6.6 Reinforcement of the Overlay Concrete Slab

In the STC method as marketed, the BRC reinforcement enables continuity over the timber joists, and is only located in a single layer at one position, near the bottom (sometimes at the bottom with no spacers to allow concrete cover to the mesh) and therefore hogging moments are not accounted for. This is a critical omission given that for STC, given that the concrete slab is a structural element subject to flexural stresses over the joists where the top part is subject to tensile stresses. Effectively, in regions around the joists, the slab is plain concrete (unreinforced).

3.6.7 Computer Modelling

3.6.7.1 Objective

The objective was to assess the effects of standard loading on two common STC construction configurations, using a case study of a 4-level classroom block located Nabweru parish, Nansana Municipality, Wakiso District.

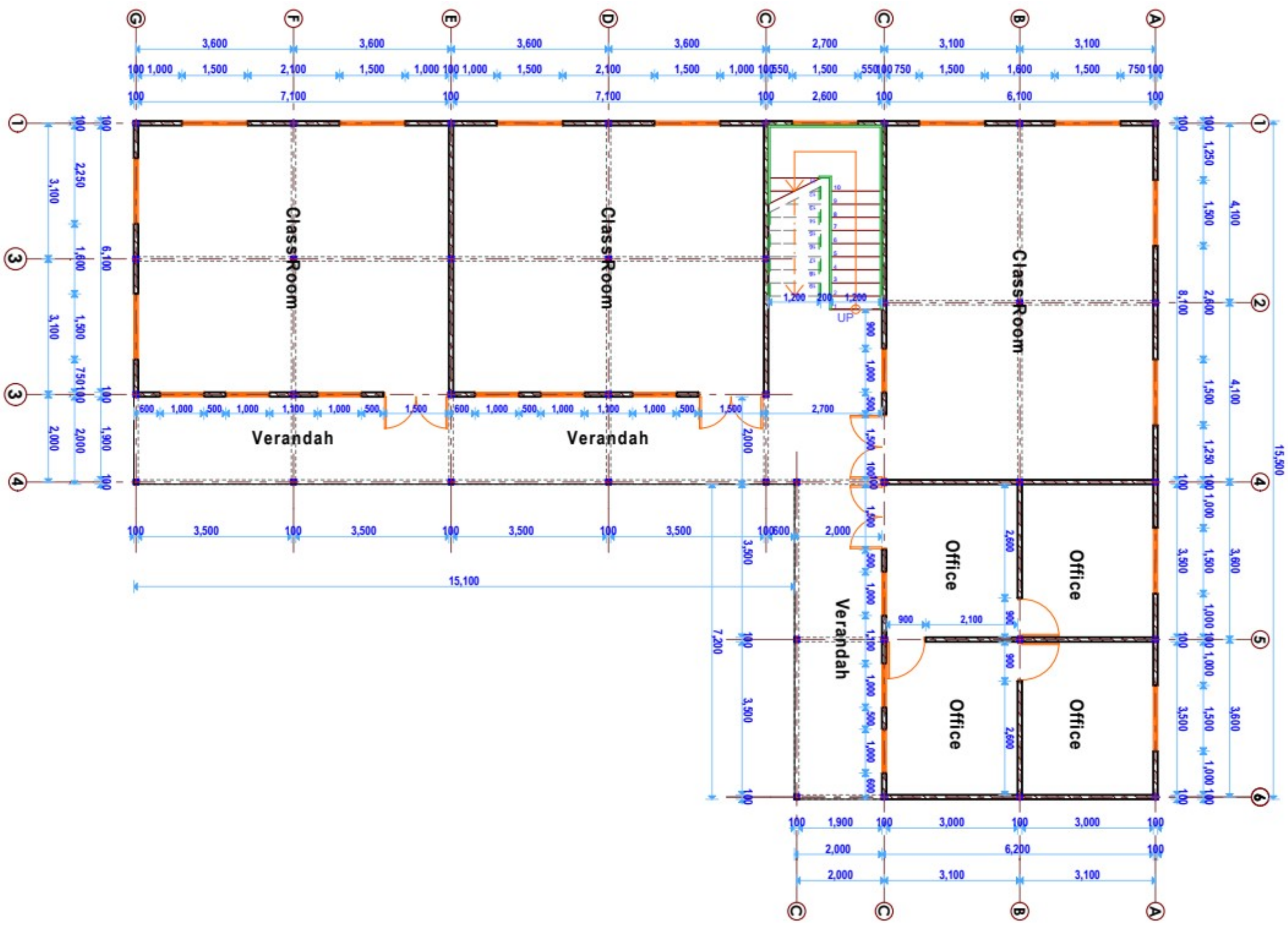


Figure 3-17: Ground Floor plan of the case study

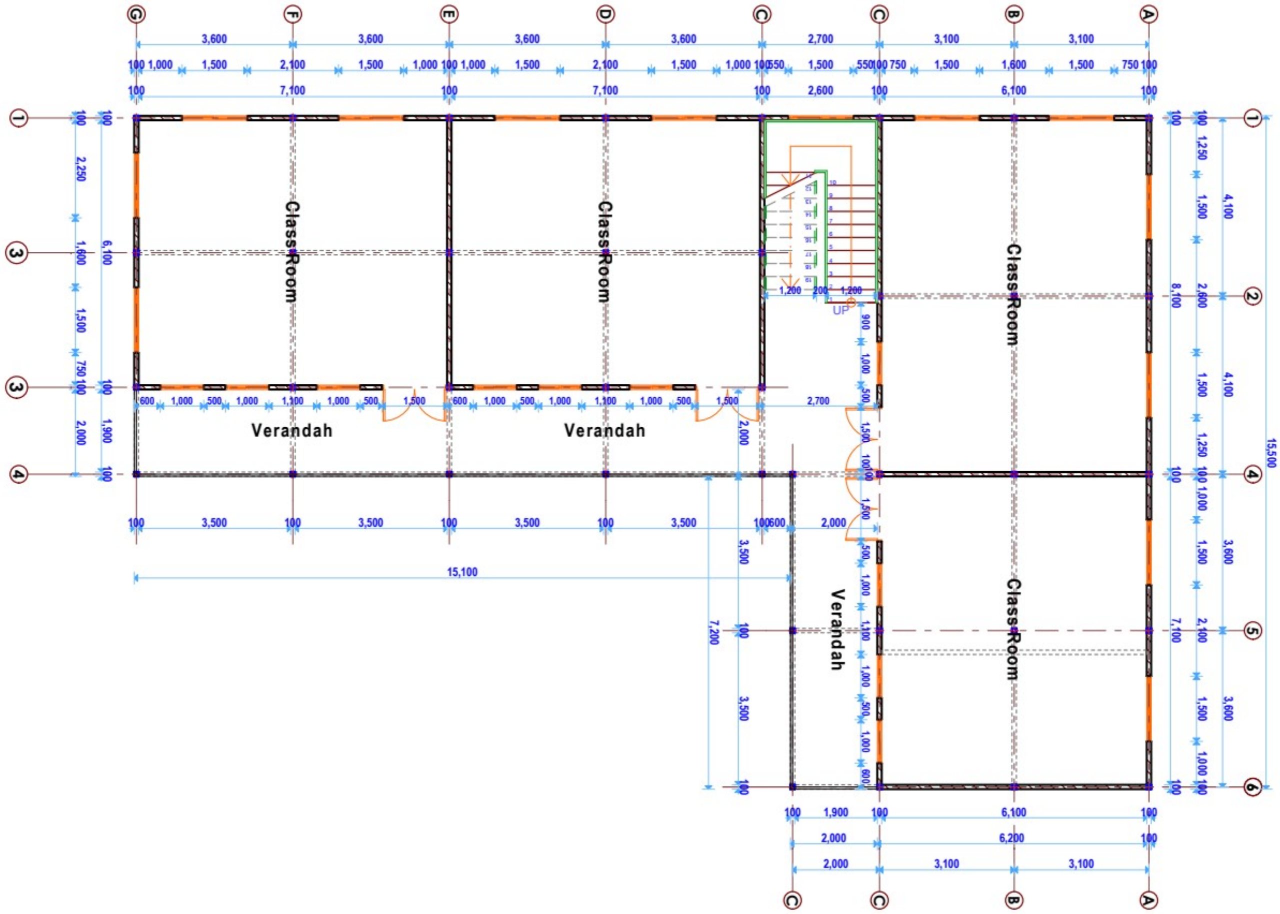
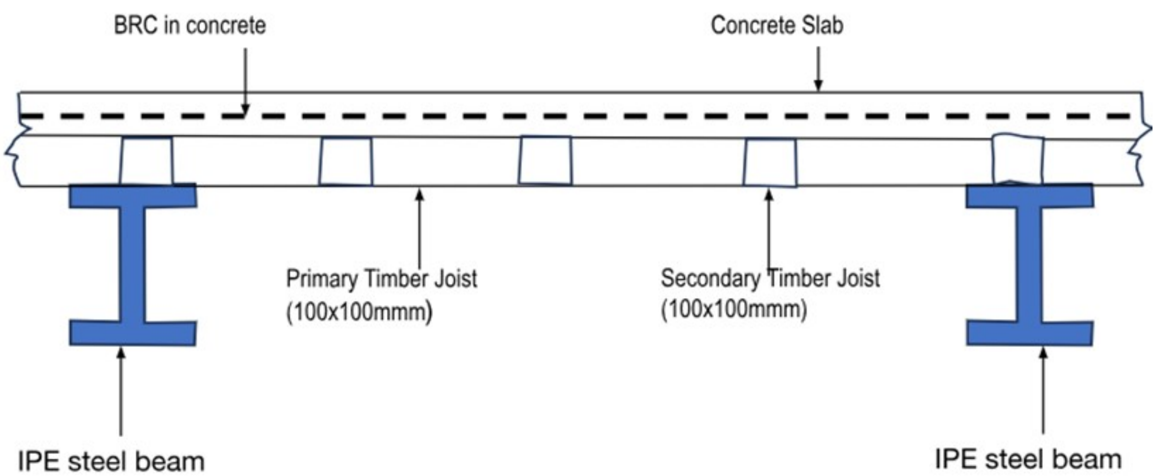


Figure 3-18: 1st, 2nd and 3rd Floor Plan of the case study

3.6.7.2 Model Cases-Idealisation

1. Form A- Current practice Type 1



A. ***Concrete Slab:*** Modelled as a series of 600x600 plate elements with stiffness such that they would be two-way spanning between the timber joists.

B. ***Secondary timber Joists:*** Modelled as continuous 600mm beam elements supported by the Primary timber joists.

C. ***Primary timber Joists:*** Modelled as beam elements spanning between the steel beams. In this form the timber is supported by resting on the top flange of the steel beam.

D. ***Steel Beams:*** Modelled as beam elements

Connection configuration: In this form the steel beam is welded to the columns. Although the degree of fixity of the welded joints is contingent on the weld workmanship, quality and specification, full restraints for the Forces and rotation in all 6 degrees of freedom were applied.

E. ***Steel Columns:*** modelled as beam elements

The General layout of the Form 1 slab is presented in the Figure 3-19:

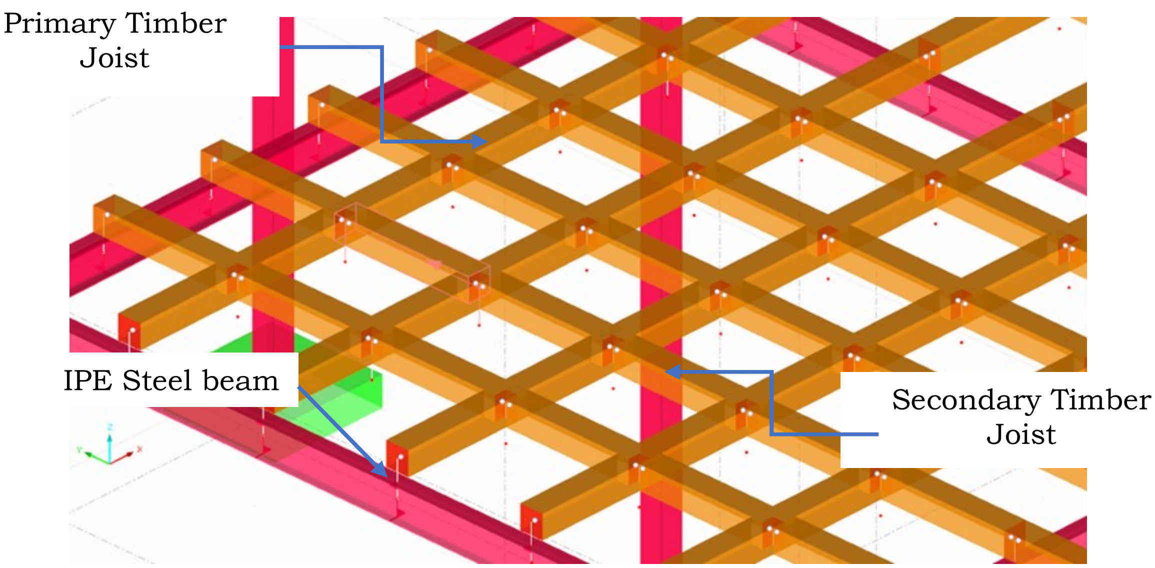
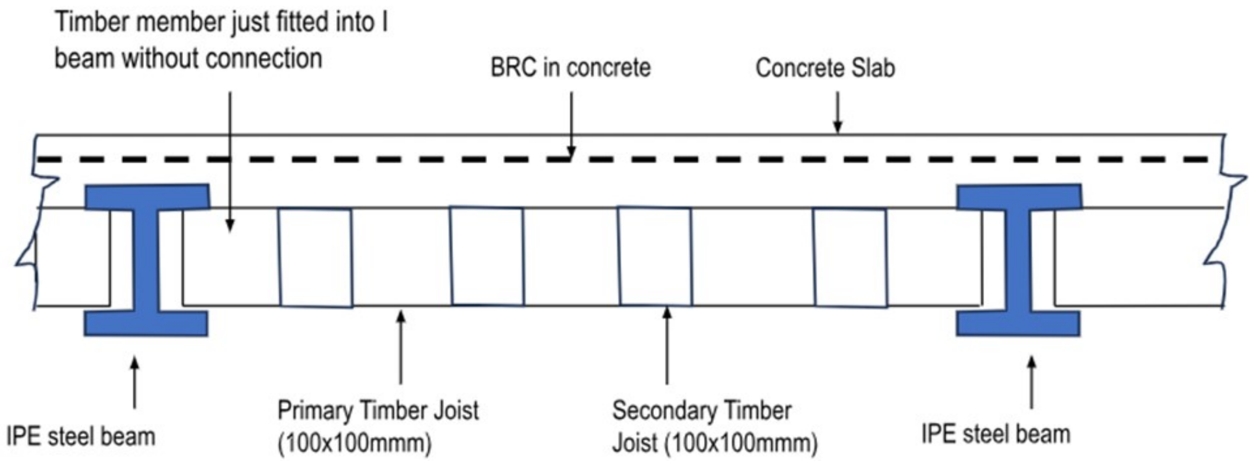


Figure 3-19: STC Current practice Form 1 Slab 3D Model

2. Form B- Current practice Type 2



Form B varied from form A in the placement of the timber joists. In this form, the primary timber beams are supported by bearing on the bottom flange of the steel beam. The secondary timber joists were modelled as continuous 600mm beam elements supported by the Primary timber joists.

The General layout of the Form 2 slab is presented in the Figure 3-20:

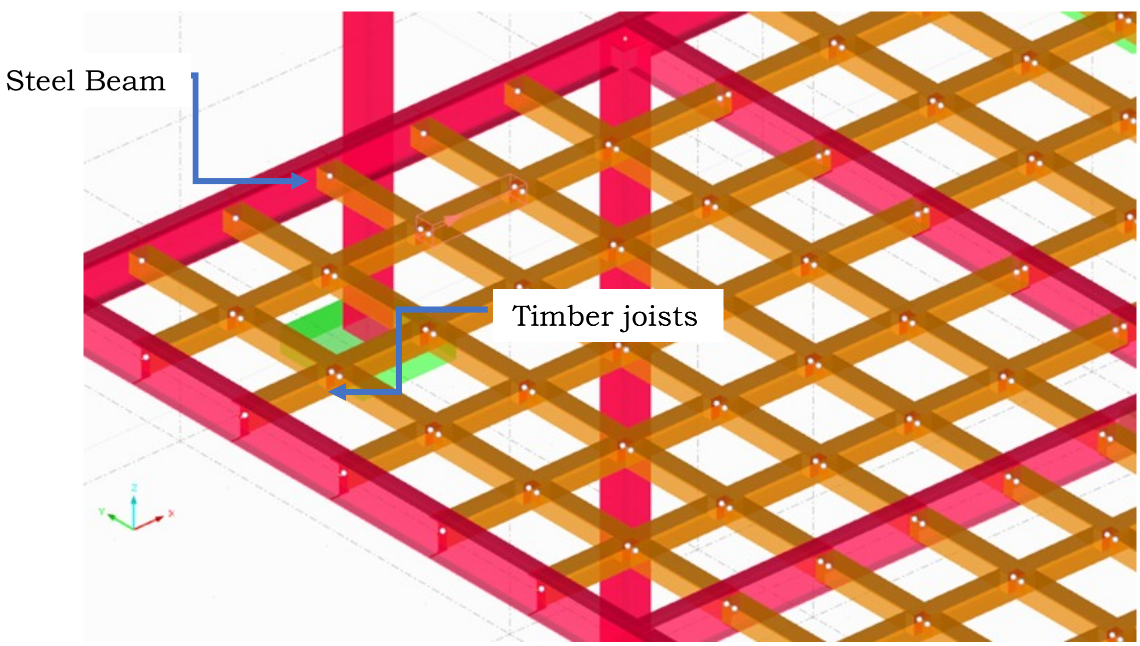


Figure 3-20: STC Current practice Form 2 Slab 3D Model

3.6.7.3 Constraints and assumptions made

The following assumptions and constraints were made and noted during the execution of this assignment:

1. The case study was chosen because it is one of the very few which had original drawings stamped by a registered engineer.
2. The as-built element sizes were what was modelled and not necessarily the designed.
3. Connection detail:
   a) Secondary to primary beam connection was a nailed connection in practice which was idealised as incapable of mobilising reliable moment transfer. As a result, the restraints on Moments on the My and Mz were released; effectively making the members pin-connected.
   b) The Primary timber beam to steel connection was idealised as capable of providing adequate restraint against forces in the z direction (Fz), though its effectiveness in restraining forces in the x and y directions is questionable. The presence of friction from dead loads is the only factor enabling some level of restraint in these directions. Therefore, partial restraints were applied to Fx and Fy, while restraints on moments for My, Mx, Mz were released.
   c) Beam to column connection: In this form the steel beam is welded to the columns. Although the degree of fixity of the welded joints is contingent on the weld workmanship, quality and specification, full restraints for the Forces and rotation in all 6 degrees of freedom were applied.
   d) Column configuration: The steel columns at each level extend from the connecting steel beams through welding. While this deviates from the typical practice for column continuity, this anomaly was considered a construction defect. For modelling purposes, the columns were assumed to be continuous, and full restraints for

forces and rotation in all six degrees of freedom at the beam joints were applied.

4. The National Building (Structural Design) Code, 2019 was used for the capacity checks and where the gaps were realised, applicable Eurocodes were adopted.

3.6.7.4 Methodology

a) ***Analysis method***

3D building finite element analysis (FEA) was the computational technique used to analyse and simulate the behaviour of the STC structures. This methodology involved creating a digital model of the building using RFEM 5 software.

b) ***Global Analysis***

- Gravity Loading***:*** A global analysis was conducted to evaluate the structural response of the STC system under gravity loading conditions. This analysis considered the combined effects of dead loads and live loads.
- Lateral Loading: A separate global analysis was performed to assess the structural behaviour of the STC system under lateral loading, such as wind and/or seismic loads. This analysis aimed to evaluate the lateral stability and overall structural performance of the system.
- Element Assessment***:*** Based on the results of the global analyses, specific elements of the STC system were selected for detailed local element analysis. This analysis focused on critical elements such as beams, columns, and slabs to evaluate their performance under various loading scenarios.

3.6.7.5 Basis of Assessment

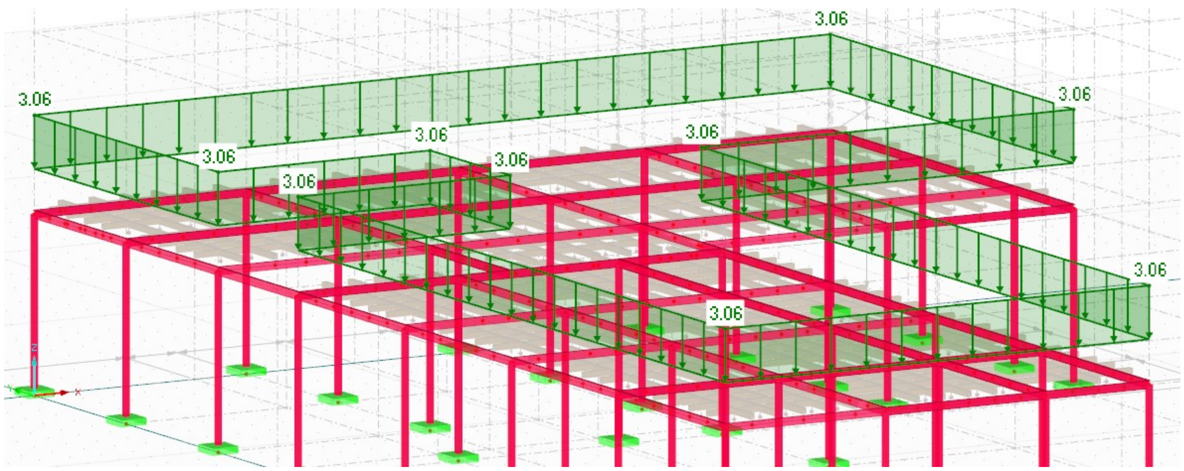
Both Ultimate Limit State (related to the safety of the structure-persistent and seismic situations) and Serviceability Limit State (relating to the functioning of the structure and the comfort of the humans using it-deflection) design situations were considered.

3.6.7.6 Assessment Actions

The Structure was subjected to the following actions:

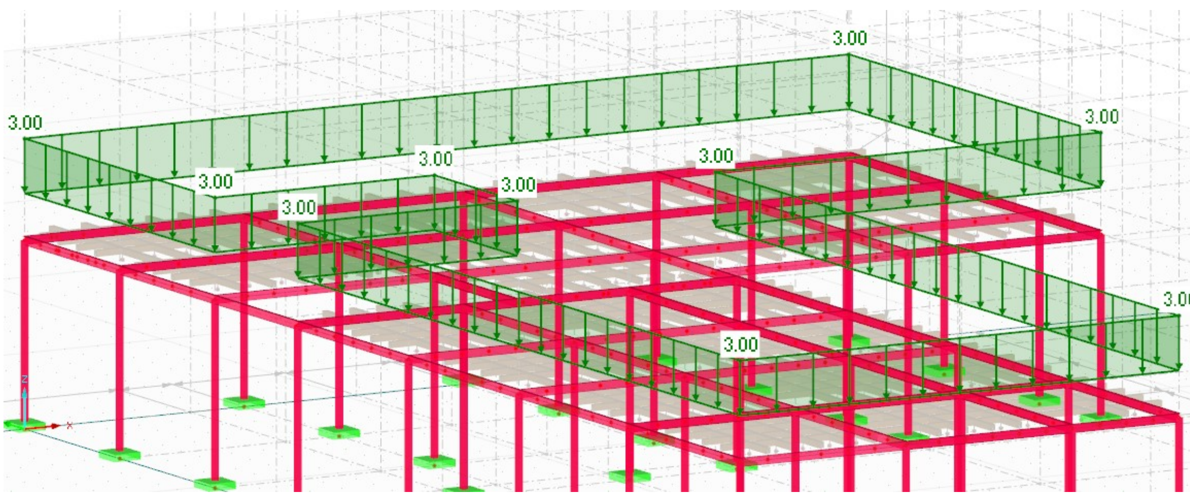
a) Persistent Situation:

i. *Permanent Actions:* self-weight of the materials (Steel, Concrete, Timber and pricking up mortar)

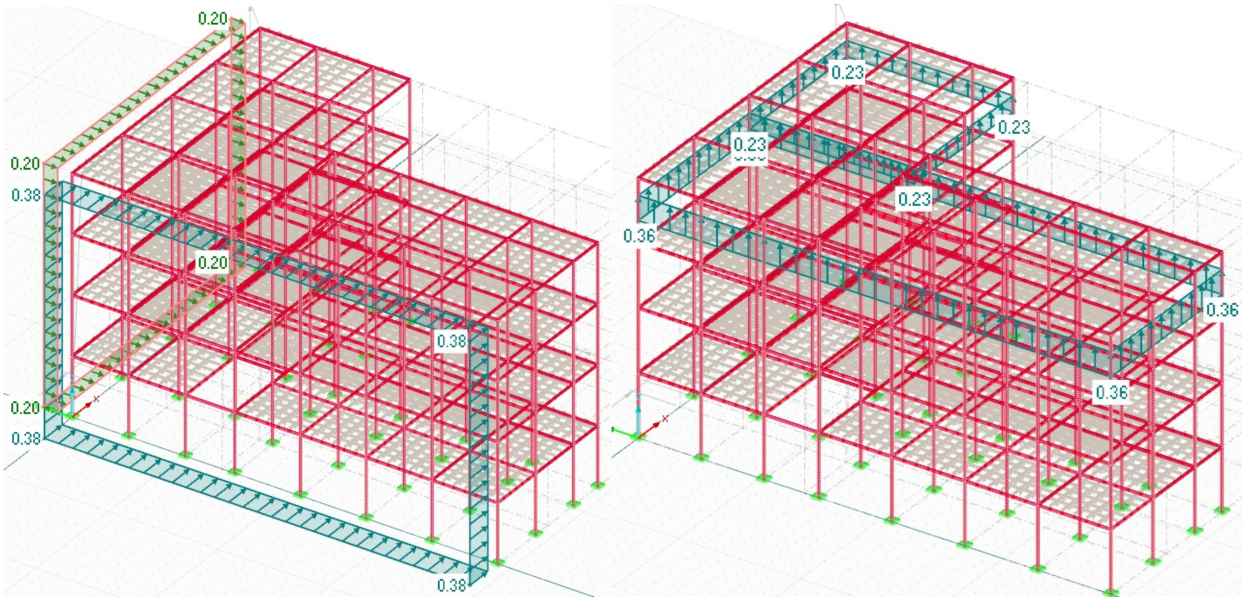


Total permanent actions applied per floor

ii. *Variable Actions:* Imposed load of 3 kN/m^2 for a school (Building category C)



The total variable action applied per floor *iii. Wind Action*



Wind pressures calculated and applied in all the three directions

b) Seismic Situation

Table 3-1: Seismic design parameters for case study

Aspect	Selected Parameter	Ref
Seismic Zone	Wakiso district -Zone 3	US 319:2003
Importance factor	School class III-1.5	US 319:2003
Peak Ground Acceleration	Wakiso District -0.08g	Global earthquake model, Uganda
Analysis Method	For building with irregular configurations-Modal Response Spectrum Method	US 319:2003
Structural type	For Welded Steel Frame-Moment resisting frame combined with infills	BS EN 1998-1:2010
Ductility Level	For Low dissipative structural behaviour approach DCM (Ductility Class Low) Behaviour factor, q = 1	BS EN 1998-1:2010
Spectrum Type	Expected surface-wave magnitude, MS < 5.5 Type 2	BS EN 1998-1:2010
Damping ratio	For Steel structures-0.03	BS EN 1998-1:2010
Ground Type	Medium Dense Gravel Presumed- D	BS EN 1998-1:2010

Accidental torsion effects: *BS EN 1998-1* accounts for accidental torsional effects assuming that the mass is displaced from the calculated centre of mass, in each direction, a distance equal to 5% percent of the building dimension at that level perpendicular to the direction of the force considered. These were considered in the modelling.

3.6.7.7 Structural Analysis, Results and Interpretation

Attempts were made to model the structure to represent the practically worst-case scenario. This scenario considered that since the timber joists "just sit" on the steel beams (No connection), the joint would be free to translate in the x and y directions. In the absence of sufficient joint action, the structure behaves as a mechanism, and lacks lateral resistance capabilities.

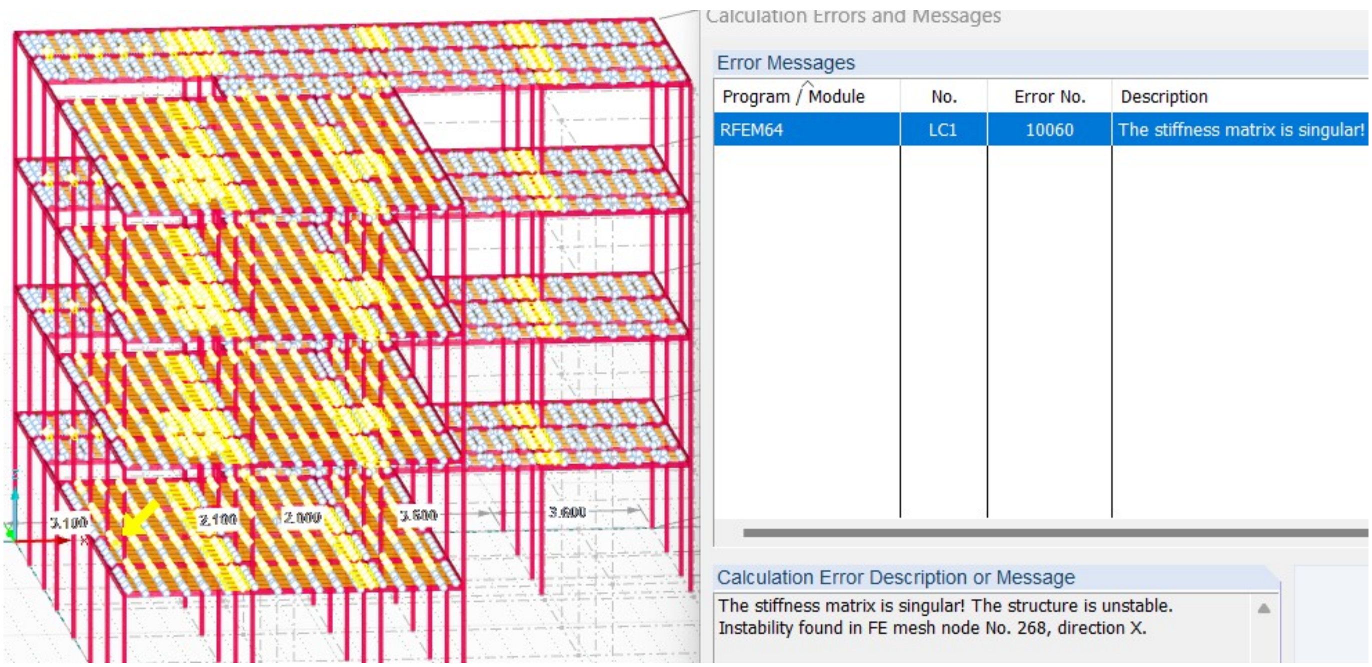


Figure 3-21: Unstable output from model due to worst case scenario 1

This instability which could occur in STC structures, would render the structure unanalysable. To enable analysis, a generous scenario of fixity was assumed. The models consequently assume a level of connectivity not guaranteed in the actual structure.

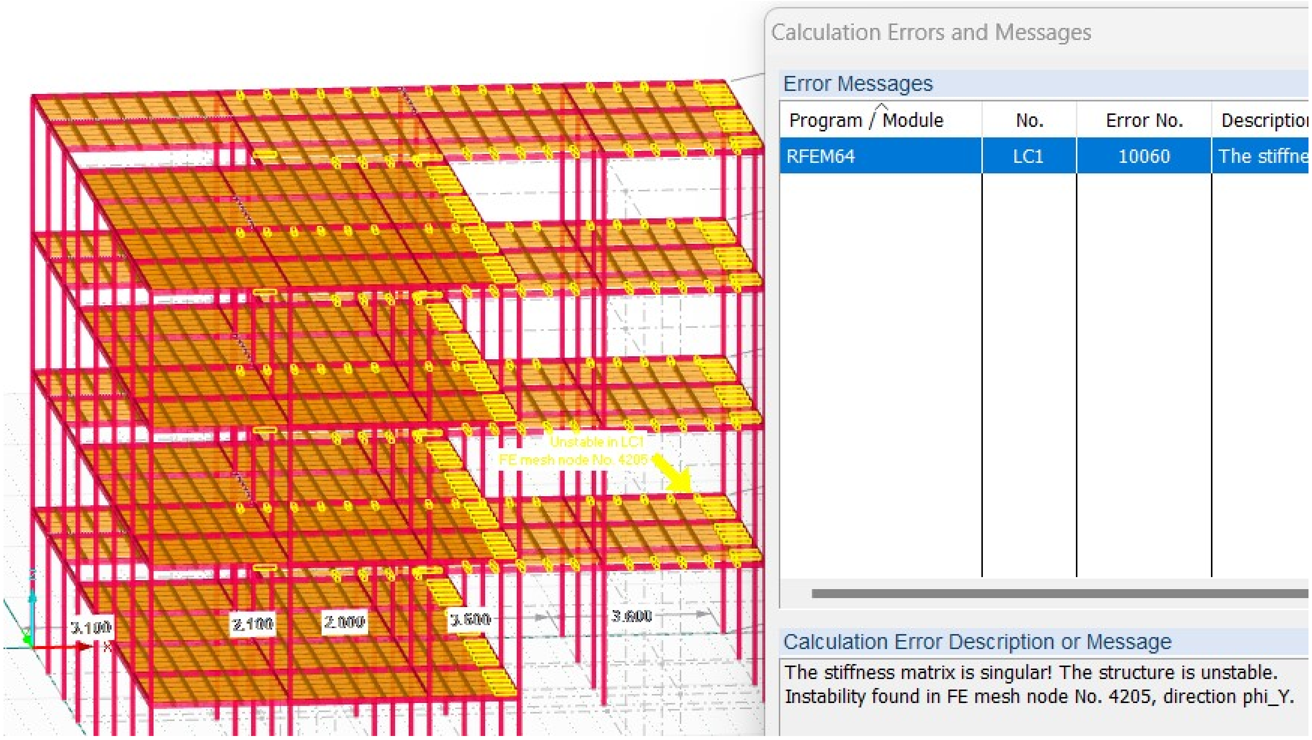
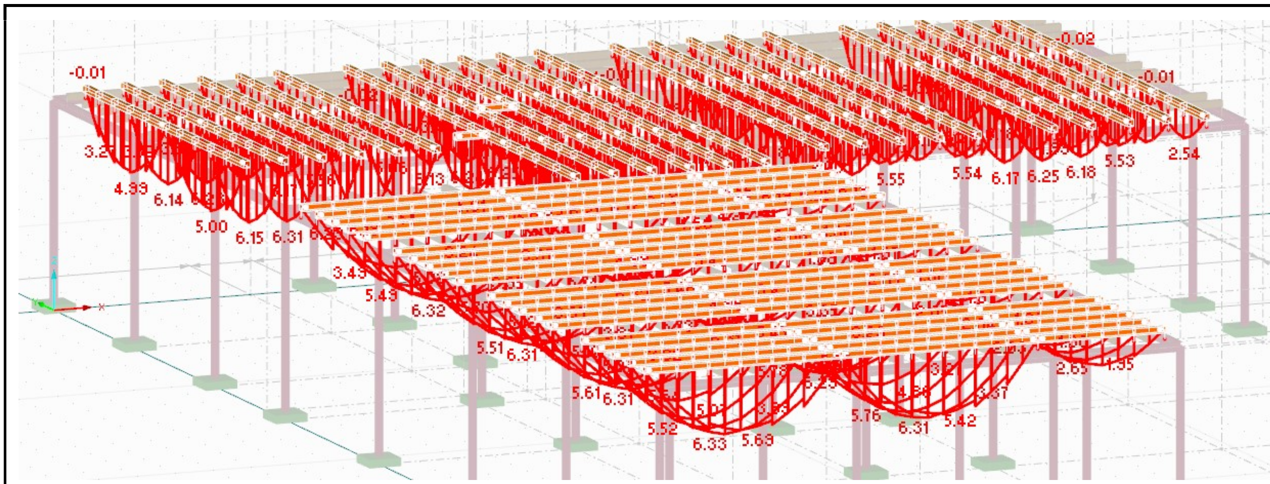


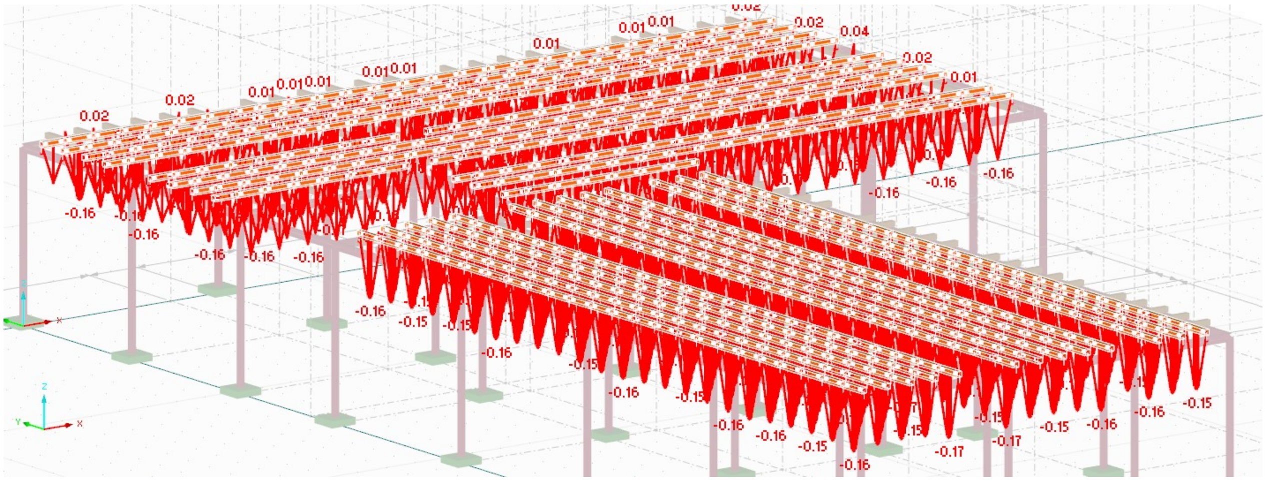
Figure 3-22: Unstable output from model due to worst case scenario 2

1. **Form 1**

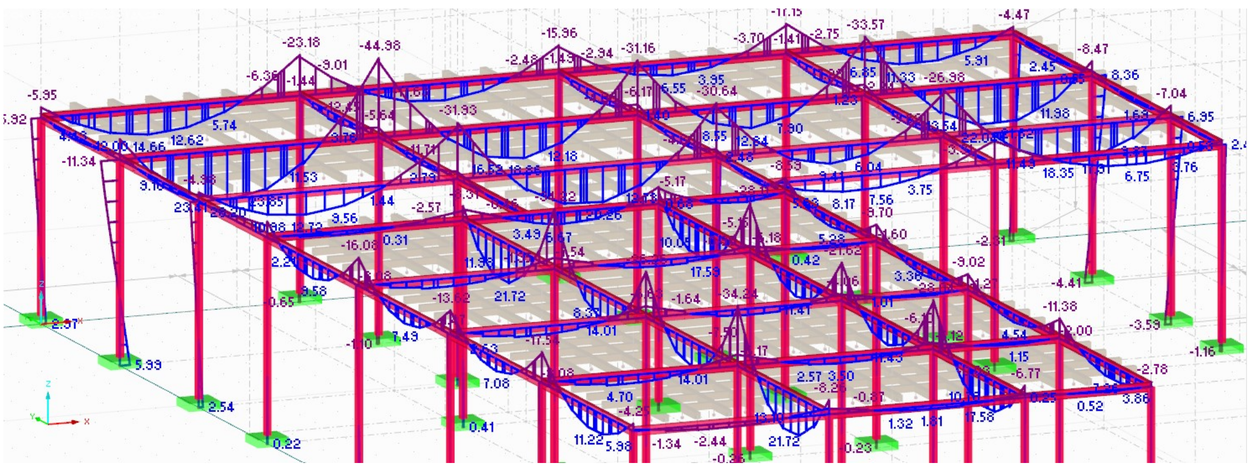
A. Design Situation 1- Persistent: The analysis of the global structure subjected to gravity loads yielded internal effects in the form of moments, shear and axial forces.



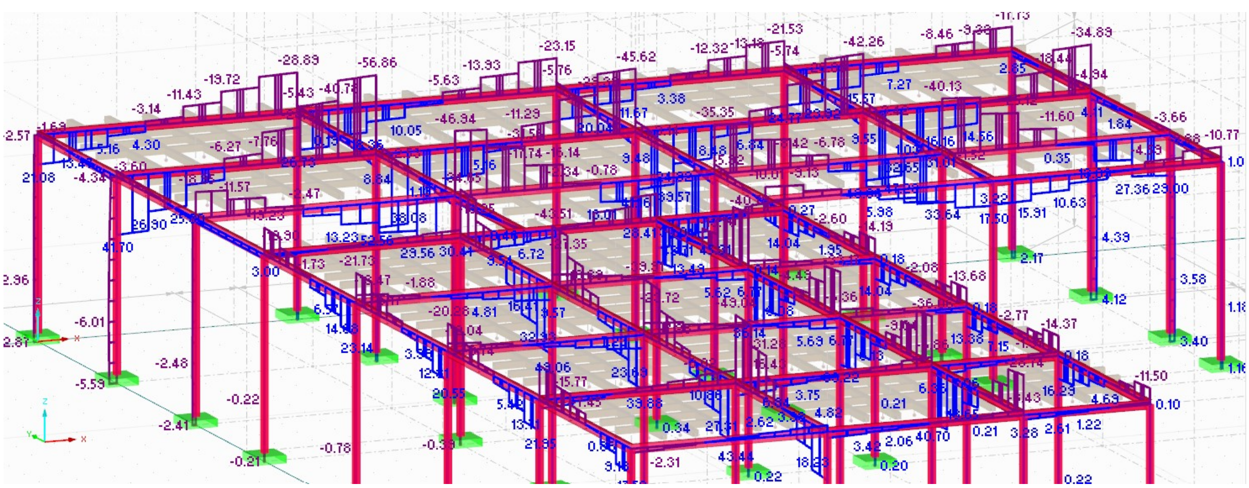
Critical Moment Diagrams (kNm)-Primary timber joists



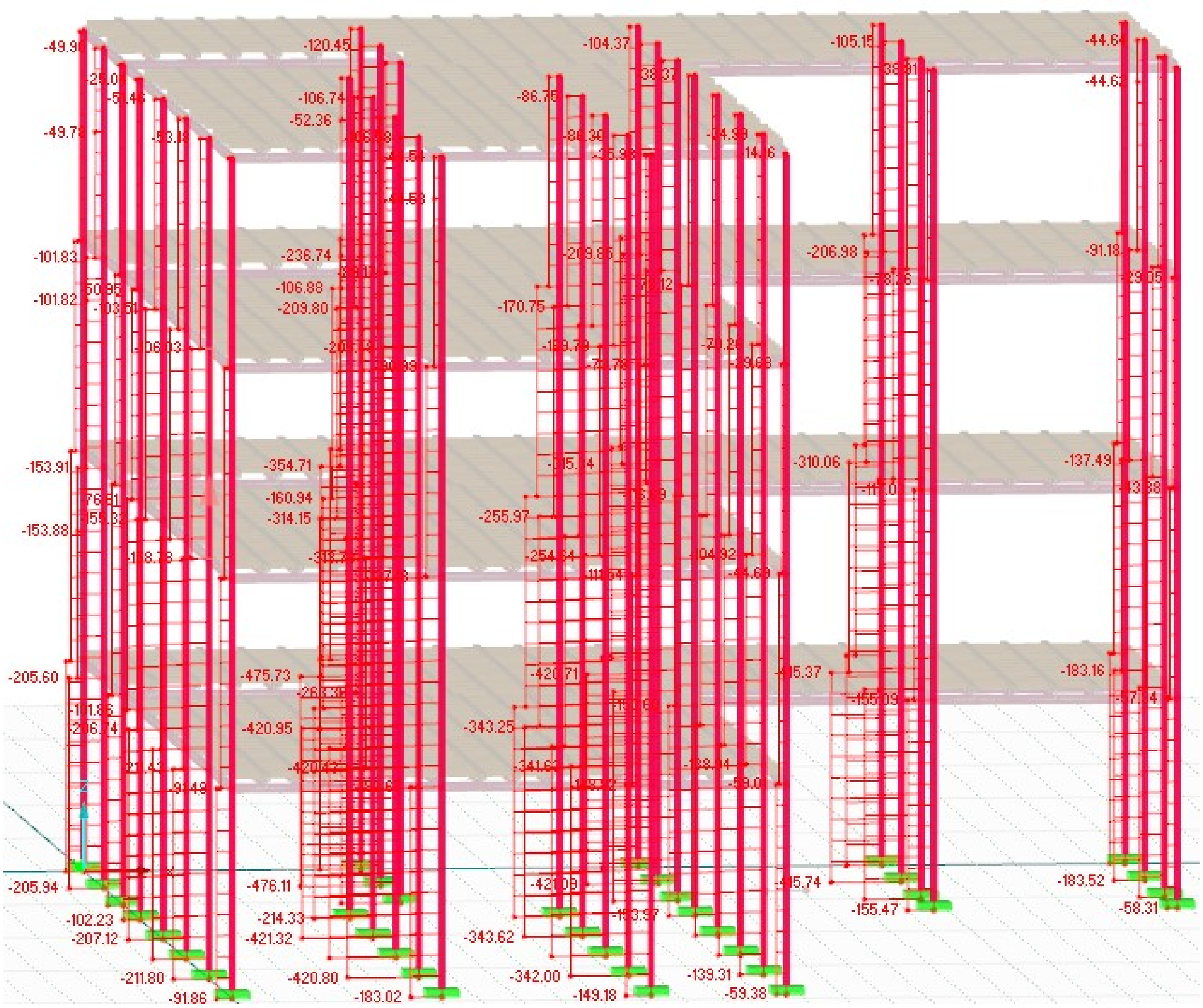
Critical Moment Diagrams (kNm)-Secondary timber joists



Critical Moment Diagrams (kNm)-Steel Beams



Critical Shear Force Diagrams (kN)-Steel Beams



Critical Axial Loads (kN)- Steel Columns

Section Verification-Form 1: Persistent situation

The Ultimate Limit State (ULS) and Serviceability Limit States (SLS) verified by checks for Axial, Flexural and Shear, and deflection respectively revealed the Form-1 floor system primary timber joists, main steel beams and Columns ***FAIL*** to meet the limit state requirements. The concrete slab passes the ULS and SLS checks.

The results are summarised as tabulated in Table 3-2:

Table 3-2: Results for Form 1 Model Persistent Design Situation

FORM-1 PERSISTENT DESIGN SITUATION							
Element	**Section properties**		**Critical Design Actions**		**Section Capacity**	**Utilization Ratio**	**Verification**
	Section Size	**Material Grade**	**Type**	**Value**			
Concrete Slab	100mm Thick	C20/25 Reinforced with A142 BRC	V_{ED} (kN/m)	3.00	32.70	0.092	Section ok
			M_{ED} (kNm/m)	0.14	0.46	0.307	Section ok
			Deflection (ratio)	Actual-8.96		0.276	*Actual <32.5* Section ok
			Slab Overall				***Section Pass***
Primary Timber joist	Square Solid 100x100 *Span 3.1m*	C14 Softwood	Shear stress (N/mm^2)	1.68	1.38	1.214	Section Fails
			Bending stress (N/mm^2)	38.00	7.00	5.429	Section Fails
			Axial Stress (N/mm^2)	38.20	5.54	6.897	Section Fails
			Deflection (mm)	*Actual* 94.80	*Allowable* 12.40	7.645	Section Fails
			Primary Timber joist-Overall				***Section Fails***
Secondary Timber joist	Square Solid 100x100 *Span 0.6m*	C14 Softwood	Shear stress (N/mm^2)	0.21	1.38	0.151	Section ok
			Bending stress (N/mm^2)	1.00	8.60	0.116	Section ok
			Axial Stress (N/mm^2)	-4.20	-12.31	0.341	Section ok
			Secondary Timber joist-Overall				***Section Pass***
Primary Steel-Beam	IPE 140	S275	V_{ED} (kN)	56.86	120.93	0.470	Section ok
			M_{ED} (kNm)	44.98	24.31	1.850	Section Fails
			N_{ED} (kN)	-7.83	-386.01	0.020	Section ok
			Deflection (mm)	*Actual* 23.80	*Allowable* 12.40	1.919	Section Fails
			Primary Steel-Beam-Overall				***Section Fails***
Steel-Column	100x100 x3 SHS *Height 3.1m*	S275	N_{ED} (kN)	-476.11	-186.85	2.548	Section Fails
			M_{ED} (kNm)	14.78	13.60	1.087	Section Fails
			V_{ED} (kN)	9.38	109.10	0.086	Section ok
			Steel-Column-Overall				***Section Fails***

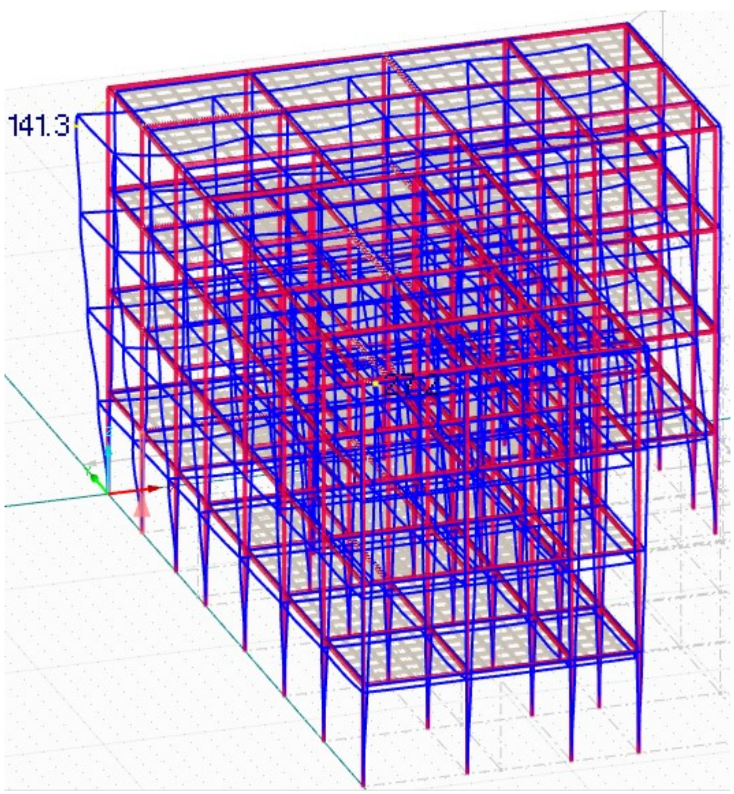
B. Situation 2- Seismic

Periods, effective masses and modal shapes: *The two fundamental periods of vibration of the building amount used were 2.68 and 2.63 seconds for x and y translation directions respectively.*

In the modal response spectrum analysis, 20 modes of vibration were considered (the sum of the effective modal masses amounts to 99% of the total mass of the structure). Five modes would be sufficient to satisfy the requirements in *BS EN 1998-1/4.3.3.3(3)* (the sum of the effective modal masses amounts to at least 90% of the total mass).

Mode	Period	Modal Mass	Effective Modal Mass Factor	
No.	Seconds	M_i [kg]	f_{meX} [-]	f_{meY} [-]
1	2.681	102362.97	0.820	0.006
2	2.633	23710.38	0.008	0.853
3	2.348	5087.02	0.034	0.007
4	1.33	344.26	0.000	0.001
5	0.99	319.86	0.000	0.000

Displacements: A displacement of 141.3mm corresponding to the seismic design situation was obtained as presented below.



<u>Serviceability Limit State: Damage Limitation check</u>

The performance requirement associated with this Limit State requires the structure to support a relatively frequent earthquake without significant damage or loss of operationality. The damage limitation requirement was therefore verified in terms of the interstorey drift (d_r) (*BS EN 1998-1/4.4.3.2*) using the equation $d_r \cdot v \leq \alpha \cdot h \Rightarrow \frac{d_r}{h} \leq \frac{\alpha}{v}$

Storey drift d_r is evaluated as the difference of the average lateral displacements, d_s in cm at the top and bottom of the storey as per *BS EN 1998-1/4.4.2.2(2)*; *h* is the storey height; *v* is the reduction factor which considers the lower return period of the seismic action associated with the damage limitation requirement. It depends on the importance class of the building. The Test building is classified as importance class III (BS EN 1998-1/Table 4.3) and the corresponding reduction factor *v* amounts to 0.4 *(BS EN 1998-1/4.4.3.2(2))*; **α** is factor which considers the type of the non-structural elements and their arrangements into the structure. It amounts to 0.005, 0.0075 and 0.01 *(BS EN 1998-1, equations 4.31, 4.32 and 4.33).*

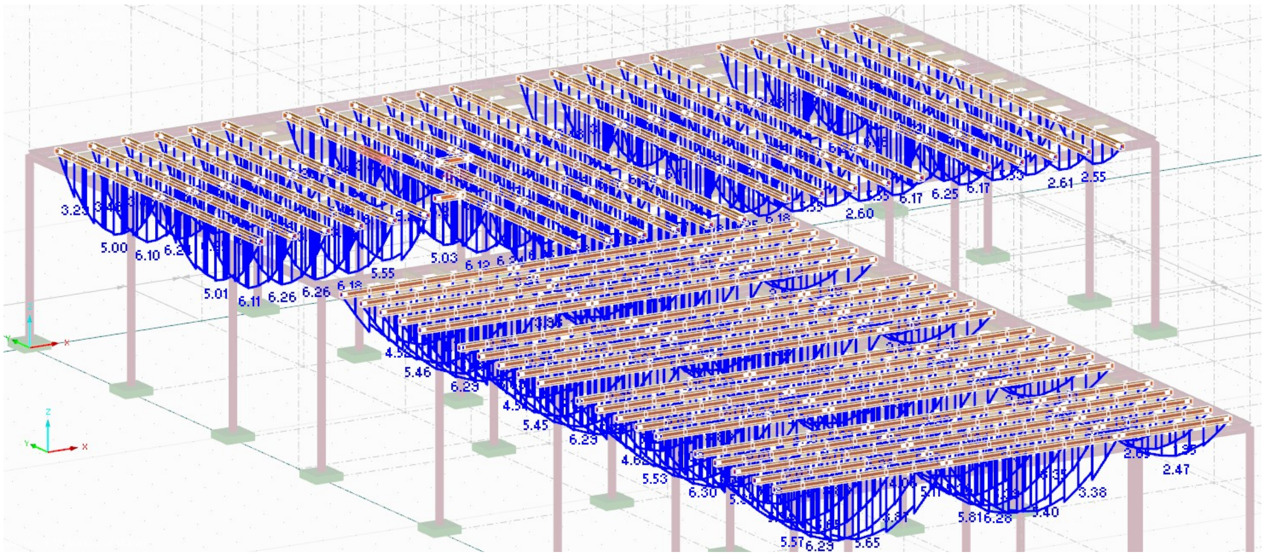
The results in show that Form 1 ***fails*** the SLS requirements for seismic situation

Calculation of Form 1 Seismic SLS check

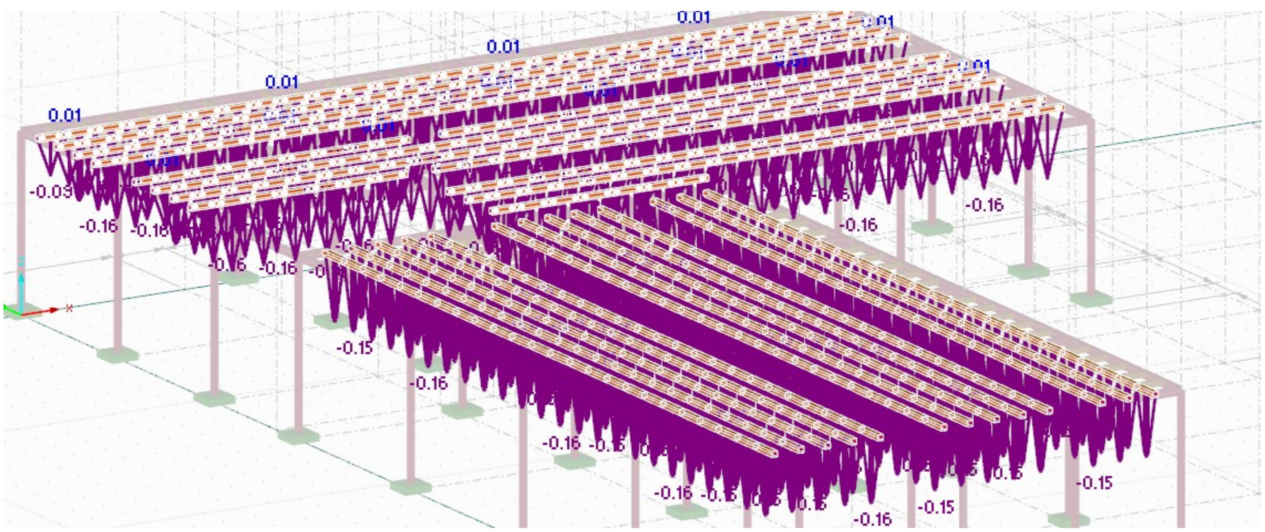
Storey	**Height (m)**	**Displacement (cm)**	**dr (cm)**	**v**	**v*dr/h**	**Verification**
1	3.1	**4.68**	4.68	0.4	0.60	v*dr/h>0.01 ***Fail***
2	3.1	**9.40**	4.72	0.4	0.61	v*dr/h>0.01 ***Fail***
3	3.1	**12.59**	3.19	0.4	0.41	v*dr/h>0.01 ***Fail***
Roof	3.1	**14.13**	1.54	0.4	0.20	v*dr/h>0.01 ***Fail***

2. Form 2

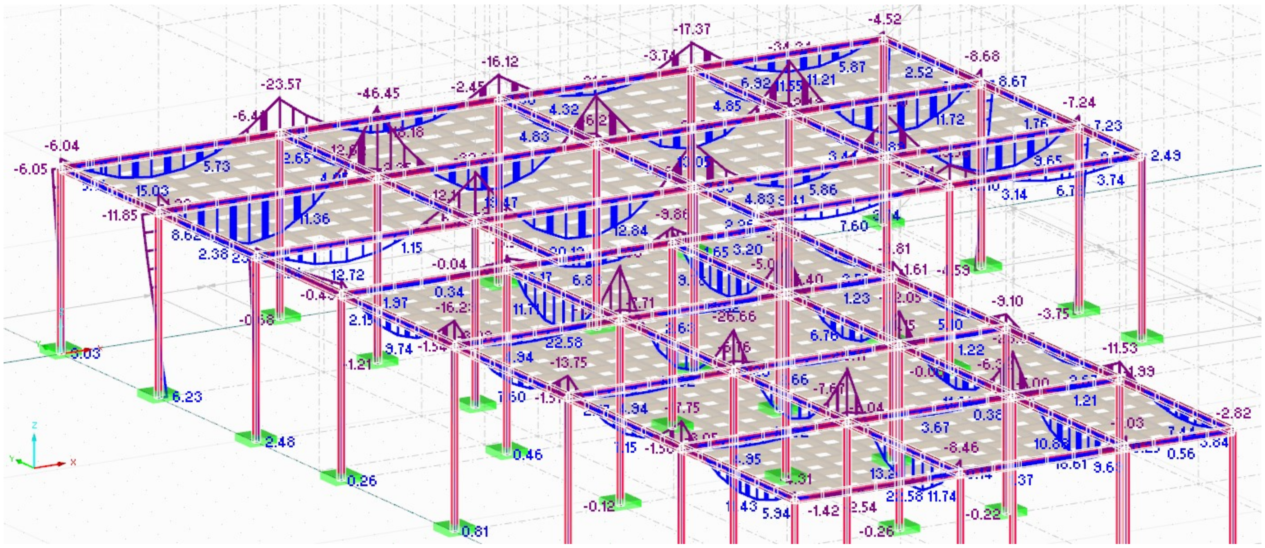
A. Design Situation 1- Persistent



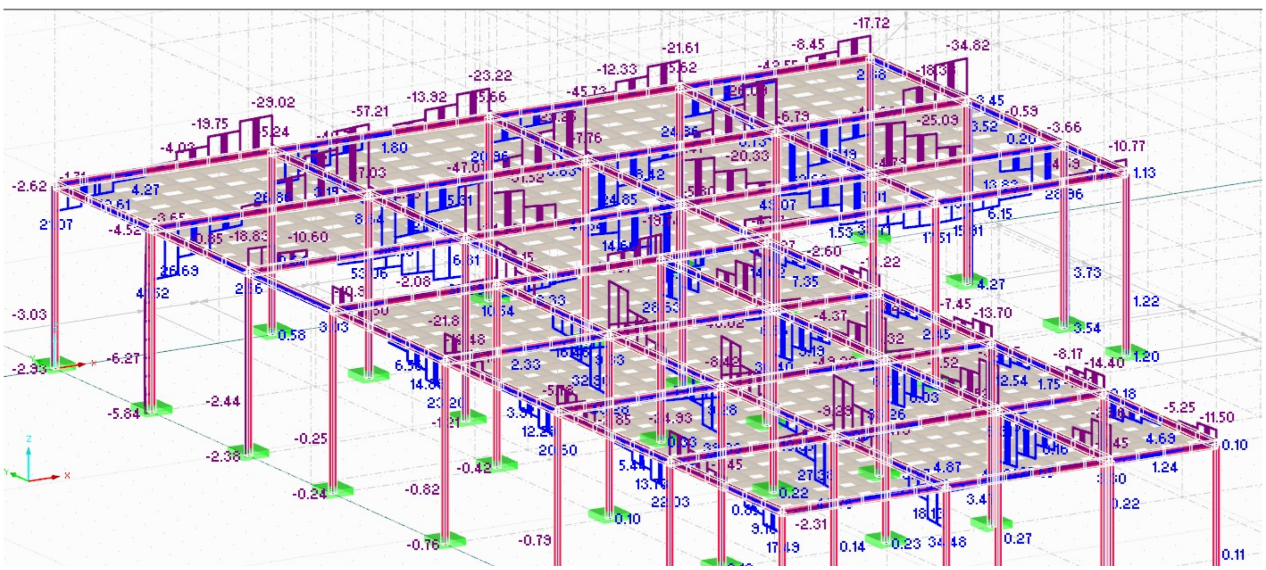
Critical Moment Diagrams (kNm)-Primary timber joists



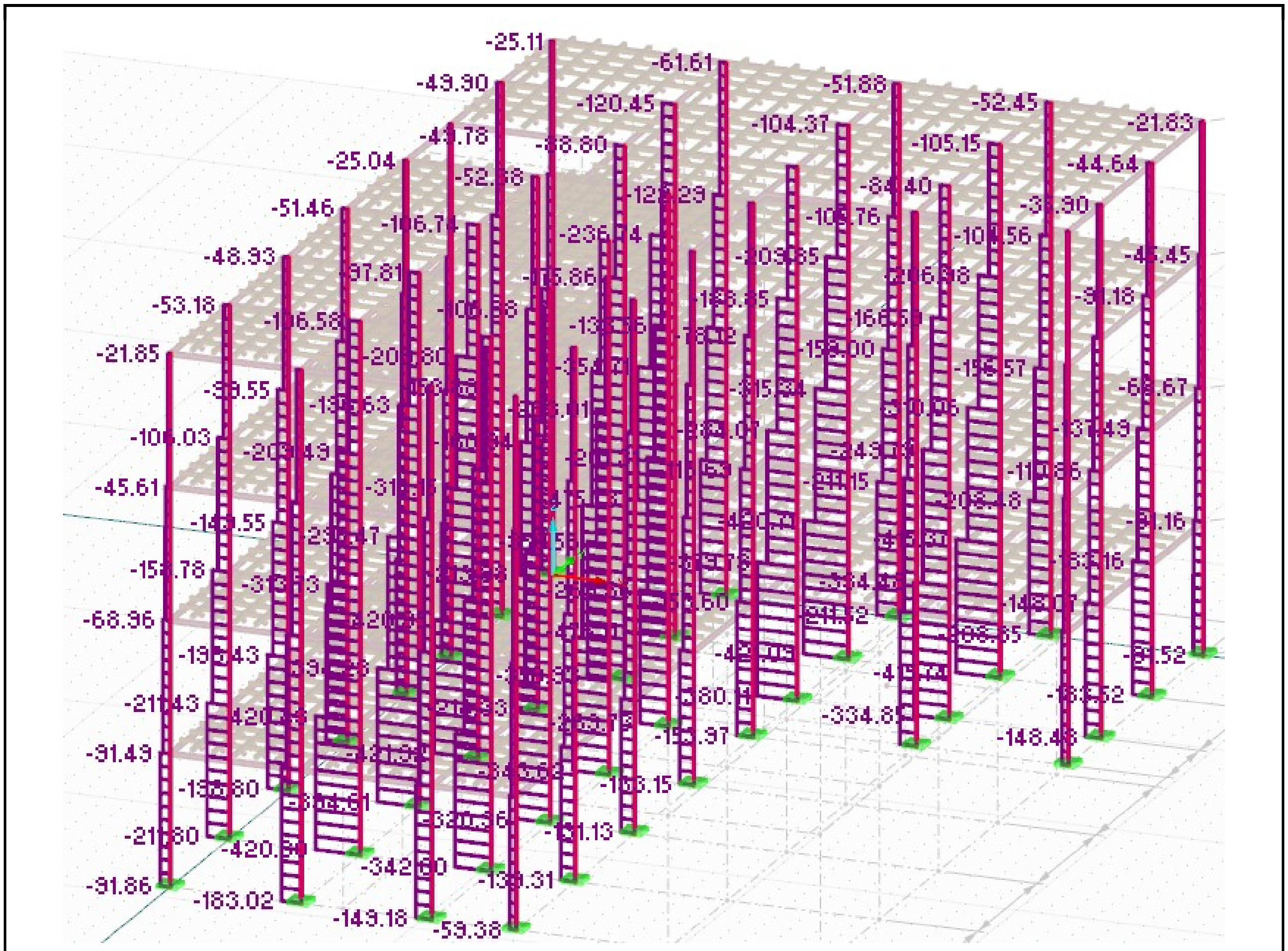
Critical Moment Diagrams (kNm)-Secondary timber joists



Critical Moment Diagrams (kNm)-Steel Beams



Critical Shear Force Diagrams (kN)-Steel Beams



Critical Axial Loads (kN)- Steel Columns

<u>Section Verification-Form 2: Persistent situation</u>

The Ultimate Limit State (ULS) and Serviceability Limit States (SLS) verified by checks for Axial, Flexural and Shear, and deflection respectively revealed the Form-1 floor system primary timber joists, main steel beams and Columns ***<u>FAIL.</u>*** The concrete slab passes the ULS and SLS checks.

Table 3-3: Results for Form 2 Model Persistent Design Situation

<table>
<tr><th colspan="8">FORM-2 PERSISTENT DESIGN SITUATION</th></tr>
<tr><th rowspan="3">Element</th><th colspan="2">Section properties</th><th colspan="2">Critical Design Actions</th><th rowspan="3">Section Capacity</th><th rowspan="3">Utilization Ratio</th><th rowspan="3">Verification</th></tr>
<tr><th rowspan="2">Section Size</th><th rowspan="2">Material Grade</th><th colspan="2"></th></tr>
<tr><th>Type</th><th>Value</th></tr>
<tr><td rowspan="3">Concrete Slab</td><td rowspan="3">100mm Thick</td><td rowspan="3">C20/25 Reinforced with A142 BRC</td><td>V_{ED} (kN/m)</td><td>3.00</td><td>32.70</td><td>0.092</td><td>Section ok</td></tr>
<tr><td>M_{ED} (kNm/m)</td><td>0.14</td><td>0.46</td><td>0.307</td><td>Section ok</td></tr>
<tr><td colspan="4">Slab Overall</td><td>Section Pass</td></tr>
<tr><td rowspan="5">Primary Timber joist</td><td rowspan="5">Square Solid 100x100 Span 3.1m</td><td rowspan="5">C14 Softwood</td><td>Shear stress (N/mm^2)</td><td>1.69</td><td>1.38</td><td>1.219</td><td>Section Fails</td></tr>
<tr><td>Bending stress (N/mm^2)</td><td>38.00</td><td>8.60</td><td>4.419</td><td>Section Fails</td></tr>
<tr><td>Axial Stress (N/mm^2)</td><td>-37.90</td><td>-12.31</td><td>3.079</td><td>Section Fails</td></tr>
<tr><td>Deflection (mm)</td><td>Actual 95.40</td><td>Allowable 12.40</td><td>7.694</td><td>Section Fails</td></tr>
<tr><td colspan="4">Primary Timber joist-Overall</td><td>Section Fails</td></tr>
<tr><td rowspan="4">Secondary Timber joist</td><td rowspan="4">Square Solid 100x100 Span 0.6m</td><td rowspan="4">C14 Softwood</td><td>Shear stress (N/mm^2)</td><td>0.18</td><td>1.38</td><td>0.128</td><td>Section ok</td></tr>
<tr><td>Bending stress (N/mm^2)</td><td>1.00</td><td>8.60</td><td>0.116</td><td>Section ok</td></tr>
<tr><td>Axial Stress (N/mm^2)</td><td>1.60</td><td>5.54</td><td>0.289</td><td>Section ok</td></tr>
<tr><td colspan="4">Secondary Timber joist Overall</td><td>Section Pass</td></tr>
<tr><td rowspan="5">Primary Steel-Beam</td><td rowspan="5">IPE 140</td><td rowspan="5">S275</td><td>V_{ED} (kN)</td><td>57.21</td><td>120.93</td><td>0.473</td><td>Section ok</td></tr>
<tr><td>M_{ED} (kNm)</td><td>46.45</td><td>24.31</td><td>1.911</td><td>Section Fails</td></tr>
<tr><td>N_{ED} (kN)</td><td>-4.67</td><td>-386.01</td><td>0.012</td><td>Section ok</td></tr>
<tr><td>Deflection (mm)</td><td>Actual 25.40</td><td>Allowable 12.40</td><td>2.048</td><td>Section Fails</td></tr>
<tr><td colspan="4">Primary Steel-Beam-Overall</td><td>Section Fails</td></tr>
<tr><td rowspan="4">Steel-Column</td><td rowspan="4">100x100 x3 SHS Height 3.1m</td><td rowspan="4">S275</td><td>N_{ED} (kN)</td><td>-476.11</td><td>-186.85</td><td>2.548</td><td>Section Fails</td></tr>
<tr><td>M_{ED} (kNm)</td><td>14.78</td><td>13.60</td><td>1.087</td><td>Section Fails</td></tr>
<tr><td>V_{ED} (kN)</td><td>10.96</td><td>109.10</td><td>0.100</td><td>Section ok</td></tr>
<tr><td colspan="4">Steel-Column-Overall</td><td>Section Fails</td></tr>
</table>

B. Situation 2-Seismic

Periods, effective masses and modal shapes

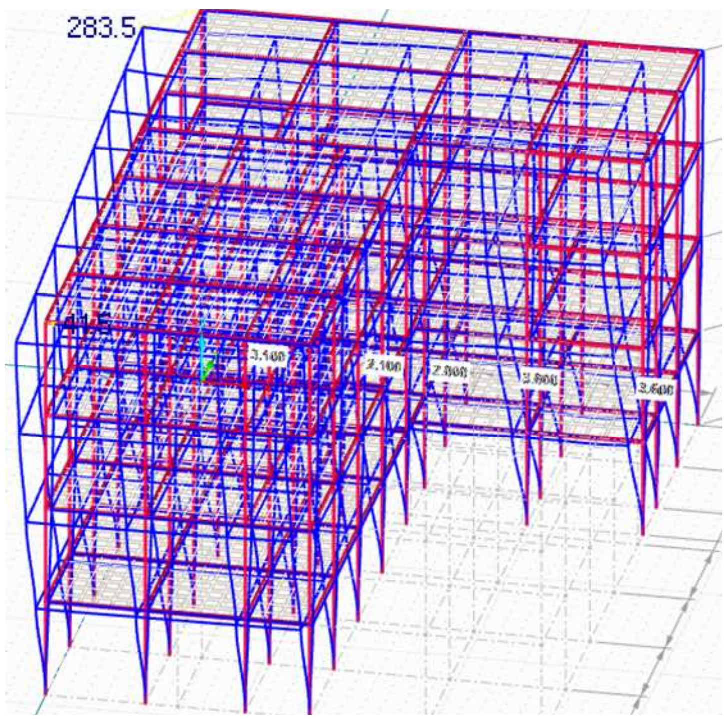
The basic modal properties of the building are summarised in the table below.

Mode	Period	Modal Mass	Effective Modal Mass Factor	
No.	Sec	M_i [kg]	f_{meX} [-]	f_{meY} [-]
1	2.669	106683.29	0.784	0.025
2	2.620	171670.10	0.032	0.839
3	2.342	51604.24	0.057	0.010

Mode	Period	Modal Mass	Effective Modal Mass Factor	
4	0.906	131553.40	0.085	0.001
5	0.895	197096.06	0.002	0.090

The two fundamental periods of vibration of the building amount to 2.669 and 2.62 Seconds for x and y translation directions respectively. In the modal response spectrum analysis all 20 modes of vibration were considered (the sum of the effective modal masses amounts to 99% of the total mass of the structure). Five modes would be sufficient to satisfy the requirements in *BS EN 1998-1/4.3.3.3(3)* (the sum of the effective modal masses amounts to at least 90% of the total mass).

Displacements: A displacement of 283.5 mm corresponding to the Seismic design situation was obtained as presented below.



<u>Serviceability Limit State: Damage Limitation check</u>

The performance requirement associated with this Limit State requires the structure to support a relatively frequent earthquake without significant damage or loss of operationality. The damage limitation requirement was therefore verified in terms of the interstorey drift (*dr*) (*BS EN 1998-1/4.4.3.2*) using the equation below.

$$d_r \cdot v \leq \alpha \cdot h \quad \Rightarrow \quad \frac{d_r}{h} \leq \frac{\alpha}{v}$$

The results in the table reveal that Form 2 ***fails*** the SLS requirements for seismic situation

Calculation of Form 1 Seismic SLS check

Storey	**Height(m)**	**Displacement (cm)**	**dr (cm)**	**v**	**v*dr/h**	**Verification**
1	3.1	1.36	1.36	0.4	0.17	v*dr/h>0.01 ***Fail***
2	3.1	14.18	7.77	0.4	1.00	v*dr/h>0.01 ***Fail***
3	3.1	4.07	9.12	0.4	1.18	v*dr/h>0.01 ***Fail***
Roof	3.1	28.35	16.21	0.4	2.09	v*dr/h>0.01 ***Fail***

3.6.7.8 Conclusions

The analysis of both forms reveals the following key findings:

A. **Persistent Design Situation:** The main frame elements, including primary timber joists, steel beams, and steel columns, fail to meet the ultimate limit state (ULS) and serviceability limit state (SLS) requirements.
B. **Seismic Design Situation**: For both forms, the diaphragm behavior of the slab is compromised by timber floor joists "just" resting on steel beams. Additionally, despite relying on frame action, the structures lack adequate lateral bracing to resist seismic forces effectively.

Significant revisions are required in the structural design of the forms to meet both Ultimate Limit State (ULS) and Serviceability Limit State (SLS) requirements.

3.6.8 Technical Committee' Opinion on the Viability of the STC Method

Given the lack of adequate engineering basis, lack of composite action, lack of consideration for lateral resistance, lack of hogging reinforcement, poor welding quality, lack of specification of timber grades and properties for use; the poor timber on the open market; the lack of engineered connections; the method as marketed by the proponents and deployed in industry with its variants is generally unsafe and not viable from a structural point of view. The poor quality of connections is particularly a huge risk since a structure is only as good as the connections bringing the different components together to work as a unit.

Minimum guidelines are necessary for industry in case players would like to deploy the use of the three materials-Steel, Concrete and Timber in combination or otherwise in slab systems.

4 STRUCTURAL TIMBER

4.1 Literature Review on Gradation/Characterization of Timber

4.1.1 Uganda Standards

The main purpose of grading timber is to establish and maintain an acceptable uniformity in the products of different suppliers, so that a given grade represents the same quality and can be used for the same purpose, regardless of the tree from which it is cut and mill by which it is produced (Tanzania Bureau of Standards, in TZS 387).

Table 4-1: Review of Ugandan Timber Standards

Reference Document	**Provision**	**Comment**
The National Building Code (Structural design), 2019, Part VII (General provisions) and Schedule 16	• Four timber strength classes, SG4, SG8, SG12 and SG16 based on allowable bending stress • SG8 and SG12 recommended for building construction • SG4 includes: Nkago, Pine, Musizi, Red Nongo, Nkoba, Mugavu • SG8 includes Mukusu, Kalitunsi (Eucalyptus), Mahogany, Nkuzanyana, Enkalati • SG12 includes: Nsambya, Mpewere, White Nongo and Namagulu • SG16 includes: Lufugo and Mukooge • Only Modulus of rupture and modulus of elasticity are provided in Schedule 16	1. Provisions in NBC (2019) are based on permissible (allowable) stress design philosophy which is now obsolete. Standards using this method were generally withdrawn in 2010. Hardly any designer nowadays uses Allowable Stress Design (ASD) philosophy. The code should be updated to align with the current limit state design (Load and resistance factor design) which uses characteristic strengths values. 2. Properties of timber vary with age, defects levels, seasoning and handling. It is not advisable to allocate a strength grade to a type/species of timber because of these variables. Suppliers should grade their timber irrespective of species and mark them. 3. Designers need more parameters (axial strength, shear strength, density, etc.) than just MoR and MoE for design purposes.

Reference Document	Provision	Comment
US 833-1:2020 Sawn Softwood Timber Grading- Part 1:	Sawn softwood shall be derived from trees of the genus Pinus.	1. What about the other timber species? Is pine the only softwood timber that is expected to be grown in Uganda? 2. There is nothing on stress grades and design values at all. The Standard is not helpful for structural design.
US 833-2 Sawn softwood timber grading — Part 2: Stress-graded structural timber and timber for frame wall construction — Specification	● 3 grades specified (5,7 and 10 ● Stress grades given are dependent on density of visually graded timber S5: 360-425 kg/m^3 S7: 425-475 kg/m^3 S10: >475 kg/m^3	1. It is not clear what properties (tensile, compression, flexural) an engineer can use for each of these grades since the only quantified parameter is density. 2. There is no guidance for visual stress grading on how a timber piece can be placed in any grade
	Timber should be marked with: ➢ the manufacturer's name ➢ the appropriate stress-grade identification ➢ the date or the batch number of the stress grading	Timber on the market is not marked. This should be enforced.
US 833-4: Sawn softwood timber grading — Part 4: Brandering and battens — Specification	Density should be > 345 kg/m^3	Other properties other than density are needed for engineering/structural design
	Should be marked with: ➢ the manufacturer's name ➢ the appropriate stress-grade identification ➢ the date or the batch number of the stress grading	1. There is no guidance for visual stress grading on how a timber piece can be placed in any grade. 2. Marking is commendable and should be enforced.

Reference Document	Provision	Comment
US 2248 Sawn hardwood timber — Grading:	Covers three basic grades *(clear grade, semi-clear grade and knotty grade)* of rough-sawn hardwood timber and timber derived from trees intended for use in the manufacture of furniture	Not applicable for structural/engineering purposes
US 1540, Mechanical stress grading of softwood timber (Flexural method) — Code of practice	Guides on how mechanical stress grading by determination of stiffness in bending of solid timber derived from pine trees should be done.	Restricted to the gradation of softwood, particularly pine
	Gives three stress grades 5,7 and 10, depending on young's modulus of elasticity.	Standard not clear on how to use the TRU grade lower limit and Computermatic lower limit to place a timber piece in a particular grade
	The timber should be marked with • The stress grader's name, trade name or trade mark • The stress grade • The date of stress grading or the batch number	Marking is commendable and should be enforced

4.1.2 European Standards (EN)

BS EN 338:2016 *Structural Timber-Strength classes,* provides for a number of strength classes, each designated by a number indicating the value of the bending in N/mm^2. For softwood species based on bending tests, the characteristic values of strength, stiffness and density for the strength classes Cxx, where xx refers to the 5-percentile characteristic bending strength value as shown in Table 4-2.

Table 4-2 : Strength classes for softwoods based on bending tests (BS EN 338:2016)

	Class	C14	C16	C18	C20	C22	C24	C27	C30	C35	C40	C45	C50
Strength properties in N/mm²													
Bending	$f_{m,k}$	14	16	18	20	22	24	27	30	35	40	45	50
Tension parallel	$f_{t,0,k}$	7,2	8,5	10	11,5	13	14,5	16,5	19	22,5	26	30	33,5
Tension perpendicular	$f_{t,90,k}$	0,4	0,4	0,4	0,4	0,4	0,4	0,4	0,4	0,4	0,4	0,4	0,4
Compression parallel	$f_{c,0,k}$	16	17	18	19	20	21	22	24	25	27	29	30
Compression perpendicular	$f_{c,90,k}$	2,0	2,2	2,2	2,3	2,4	2,5	2,5	2,7	2,7	2,8	2,9	3,0
Shear	$f_{v,k}$	3,0	3,2	3,4	3,6	3,8	4,0	4,0	4,0	4,0	4,0	4,0	4,0
Stiffness properties in kN/mm²													
Mean modulus of elasticity parallel bending	$E_{m,0,mean}$	7,0	8,0	9,0	9,5	10,0	11,0	11,5	12,0	13,0	14,0	15,0	16,0
5 percentile modulus of elasticity parallel bending	$E_{m,0,k}$	4,7	5,4	6,0	6,4	6,7	7,4	7,7	8,0	8,7	9,4	10,1	10,7
Mean modulus of elasticity perpendicular	$E_{m,90,mean}$	0,23	0,27	0,30	0,32	0,33	0,37	0,38	0,40	0,43	0,47	0,50	0,53
Mean shear modulus	G_{mean}	0,44	0,50	0,56	0,59	0,63	0,69	0,72	0,75	0,81	0,88	0,94	1,00
Density in kg/m³													
5 percentile density	ρ_k	290	310	320	330	340	350	360	380	390	400	410	430
Mean density	ρ_{mean}	350	370	380	400	410	420	430	460	470	480	490	520

NOTE 1 Values given above for tension strength, compression strength, shear strength, char. modulus of elasticity in bending, mean modulus of elasticity perpendicular to grain and mean shear modulus have been calculated using the equations given in EN 384.

NOTE 2 The tension strength values are conservatively estimated since grading is done for bending strength.

NOTE 3 The tabulated properties are compatible with timber at moisture content consistent with a temperature of 20 °C and a relative humidity of 65 %, which corresponds to a moisture content of 12 % for most species.

NOTE 4 Characteristic values for shear strength are given for timber without fissures, according to EN 408.

NOTE 5 These classes may also be used for hardwoods with similar strength and density profiles such as e.g. poplar or chestnut.

NOTE 6 The edgewise bending strength may also be used in the case of flatwise bending.

Table 4-3: Strength classes for hardwoods based on bending tests (BS EN 338:2016)

	Class	D18	D24	D27	D30	D35	D40	D45	D50	D55	D60	D65	D70	D75	D80
Strength properties in N/mm²															
Bending	$f_{m,k}$	18	24	27	30	35	40	45	50	55	60	65	70	75	80
Tension parallel	$f_{t,0,k}$	11	14	16	18	21	24	27	30	33	36	39	42	45	48
Tension perpendicular	$f_{t,90,k}$	0,6	0,6	0,6	0,6	0,6	0,6	0,6	0,6	0,6	0,6	0,6	0,6	0,6	0,6
Compression parallel	$f_{c,0,k}$	18	21	22	24	25	27	29	30	32	33	35	36	37	38
Compression perpendicular	$f_{c,90,k}$	4,8	4,9	5,1	5,3	5,4	5,5	5,8	6,2	6,6	10,5	11,3	12,0	12,8	13,5
Shear	$f_{v,k}$	3,5	3,7	3,8	3,9	4,1	4,2	4,4	4,5	4,7	4,8	5,0	5,0	5,0	5,0
Stiffness properties in kN/mm²															
Mean modulus of elasticity parallel bending	$E_{m,0,mean}$	9,5	10,0	10,5	11,0	12,0	13,0	13,5	14,0	15,5	17,0	18,5	20,0	22,0	24,0
5 percentile modulus of elasticity parallel bending	$E_{m,0,k}$	8,0	8,4	8,8	9,2	10,1	10,9	11,3	11,8	13,0	14,3	15,5	16,8	18,5	20,2
Mean modulus of elasticity perpendicular	$E_{m,90,mean}$	0,63	0,67	0,70	0,73	0,80	0,87	0,90	0,93	1,03	1,13	1,23	1,33	1,47	1,60
Mean shear modulus	G_{mean}	0,59	0,63	0,66	0,69	0,75	0,81	0,84	0,88	0,97	1,06	1,16	1,25	1,38	1,50
Density in kg/m³															
5 percentile density	ρ_k	475	485	510	530	540	550	580	620	660	700	750	800	850	900
Mean density	ρ_{mean}	570	580	610	640	650	660	700	740	790	840	900	960	1020	1080

NOTE 1 Values given above for tension strength, compression strength, shear strength, char. modulus of elasticity in bending, mean modulus of elasticity perpendicular to grain and mean shear modulus, have been calculated using the equations given in EN 384.

NOTE 2 The tabulated properties are compatible with timber at moisture content consistent with a temperature of 20 °C and a relative humidity of 65 %, which corresponds to a moisture content of 12 % for most species.

NOTE 3 Characteristic values for shear strength are given for timber without fissures, according to EN 408.

NOTE 4 The edgewise bending strength may also be used in the case of flatwise bending.

For hardwood species based on bending tests, the characteristic values of strength, stiffness and density for the strength classes Dxx, where xx refers to the 5-percentile characteristic bending strength value, are given in Table 4-3.

According to BS EN 338:2016, a timber population may be assigned to a strength class if its characteristic values of strength, mean modulus of elasticity and density equal or exceed the values for that strength class given in Table 4.2 or Table 4.3.

4.1.3 East Africa- Tanzania Standards

The Tanzanian standard, TZS 387:2020 Timber-Strength Grading of coniferous sawn timber (cypress and pine) for structural use, specifies requirements for five stress grades of structural timber derived from trees of coniferous sawn timber (cypress and pine) seasoned at moisture content less or equal to 22%.

The standard gives limiting strength values for the five strength grades; T16, T18, T24, T30 and T40, as shown in the extract in Table 4-4.

Table 4-4: Characteristic/nominal values for sawn timber (TZS 387:2020)

S/No.	Strength Grade	Bending, Mpa	Tension parallel to grain, Mpa	Compression parallel to grain, Mpa	Modulus of Elasticity, Mpa
i)	T16	16	7.5	16	10,500
ii)	T18	18.8	8.7	19	12,600
iii)	T24	22.2	8.9	20.2	13,747
iv)	T30	23.8	9.6	21.8	17,911
v)	T40	33.2	10.7	23.8	19,314

The standard gives characteristic values of 4 strength parameters-Bending, tension parallel to the grain, Compression parallel to the grain and Modulus of Elasticity.

This is a variance with the nomenclature in other standards where the figure in the naming is usually an indication of the bending strength, apart from T16; an engineer would need to refer to the table for flexural design even if a certain timber piece was marked as such.

Other important design parameters such as density, shear strength, axial strength perpendicular to the grain are not given in that table and would have to be computed by the design engineer.

Clause 6 of TZ 387:2020 requires that every piece of graded timber shall have the following information clearly and indelibly marked on at least one face

(i) A mark identifying the grader or the company responsible for grading
(ii) The grade of the piece
(iii) The certification body logo or mark

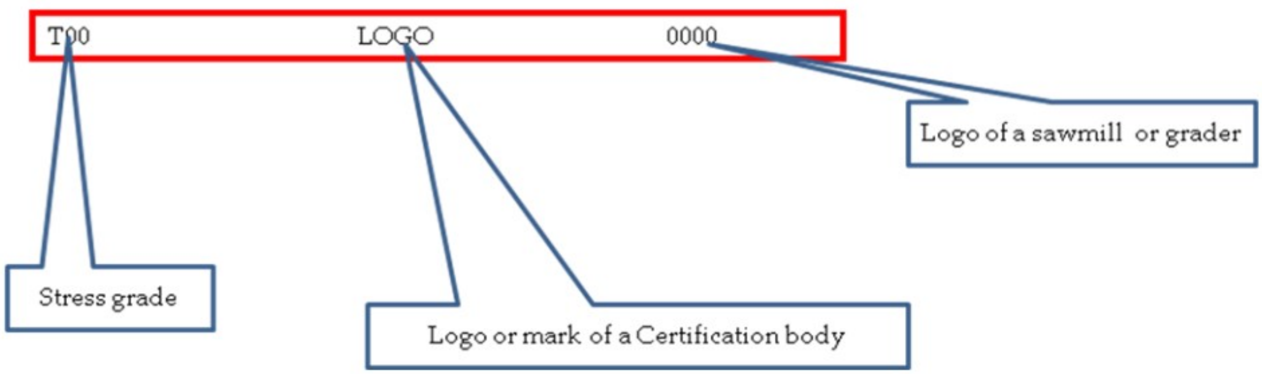


Figure 4-1: Example of a mark on visually strength graded softwood (TZS 387:2020)

4.1.4 Makerere University

Zziwa et al (2010)[4] developed a strength class system, consisting of SG4, SG8, SG12 and SG16; named following the allowable modulus of rupture. In the same publication, the authors lumped tree species in different classes. This characterization was adopted by the National Building (Structural Design) Code, 2019 for timber strength classes and properties.

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[4] 27. Ahamada Zziwa, Yasin Naku Ziraba, Jackson A. Mwakali, Strength Characterization of Timbers for Building Construction in Uganda, Second International Conference on Advances in Engineering and Technology, 2010

They recommended that various tree species on Uganda's market be allocated to the four timber classes for quality assurance in timber trade and utilization. The work was part of a PhD thesis by Ahamada Zziwa, on the topic 'Strength Characterization of timbers for building construction in Uganda', successfully defended in 2012. In his Thesis, Zziwa contends that there are at least 48 used species in Uganda's timber market. *The task force noted that the published data had very high standard deviations. This is expected given that there is no control of the age at harvesting.*

4.2 Properties of Commonly Used Timber in Uganda

4.2.1 Strength and Stiffness Properties of Common Timber

As a biological material, timber is complex due to its heterogeneity in properties which are dependent on on-site characteristics, age, and species, among others. Wood testing is, therefore, important to ascertain its properties to guide decision making on material selection for constructing timber structures and other applications.

The strength and stiffness characteristic values for the selected construction timbers for this study, drawn randomly from Bwaise and Ndeeba are shown in Table 4-5.

The results show that even for timber of the same species, the results could be markedly different as can be seen from Pine (Bwaise) with a bending strength of 26 N/mm^2 against a value of 16 N/mm^2 for the same species sourced from Ndeeba. Eucalyptus from Ndeeba returned a bending strength of 43 N/mm^2 while that from Bwaise returned a bending strength of 35 N/mm^2. It is therefore not wise to allocate a strength class to a species. Factors such as age, growth conditions, growth defects, seasoning and post-harvesting handling, all of which vary even within the same species, affect the strength of the timber.

Table 4-5: Strength and stiffness values of common local timber species

		Pinus caribaea (Pine) (Bwaise)	***Eucalypt hybrid GC 540 (Eucalyptus) (Bwaise)***	***Pinus caribaea (Pine) (Ndeeba)***	***Markhamia lutea (Musambya) (Ndeeba)***	***Albizia coriaria (Mugavu) (Ndeeba)***	***Maesopsis eminii (Musizi) (Ndeeba)***	***Eucalyptus grandis (Ndeeba)***
Strength Properties (N/mm²)	*Characteristic Bending*	*26.3*	*34.7*	*15.7*	*35.9*	*22*	*23.6*	*43*
	Characteristic Tension parallel to the grain	*15.8*	*20.8*	*9.4*	*21.6*	*13.2*	*14.2*	*25.9*
	Characteristic Tension perpendicular to the grain	*0.5*	*0.6*	*0.4*	*0.6*	*0.6*	*0.40*	*0.6*
	Characteristic Compression parallel to the grain	*21.8*	*24.7*	*17.3*	*25.1*	*20.1*	*20.7*	*27.2*
	Characteristic Compression perpendicular to the grain	*2.3*	*6.2*	*1.9*	*6.0*	*8.4*	*3.5*	*6.0*
	Characteristic Shear	*2.7*	*3.4*	*1.8*	*3.5*	*2.4*	*2.5*	*3.8*
Physical properties	*Mean Density (kg/m³)*	*409.8*	*488.2*	*353.1*	*433.7*	*601.5*	*340.2*	*494.8*
	5 Percentile Density (kg/m³)	*333.6*	*412.1*	*276.6*	*400.0*	*557.9*	*235.3*	*399.6*
	Mean Moisture Content (%)	*36.4*	*58.5*	*32.0*	*16.5*	*63.4*	*67.9*	*75.1*
Stiffness Properties (kN/mm²)	*Mean Modulus of Elasticity Parallel to grain*	*7.1*	*11.2*	*7.2*	*9.5*	*7.0*	*7 .2*	*12.0*
	5 Percentile M.O.E parallel to grain	*4.8*	*9.4*	*4.8*	*8.0*	*5.9*	*6.1*	*10.0*
	Mean Modulus of Elasticity perpendicular to the grain	*0.24*	*0.74*	*0.24*	*0.63*	*0.47*	*0.48*	*0.80*
	Mean Shear Modulus	*0.44*	*0.70*	*0.45*	*0.59*	*0.44*	*0.45*	*0.75*

In February 2022, the school of Forestry, Environmental and Geographical Sciences, Makerere University carried out tests on kiln dried pine from Busoga Forestry Company/Green Resources, to determine its properties. The comparison of properties with those of timber from Bwaise and Ndeeba in figures 4-2 and 4-3.

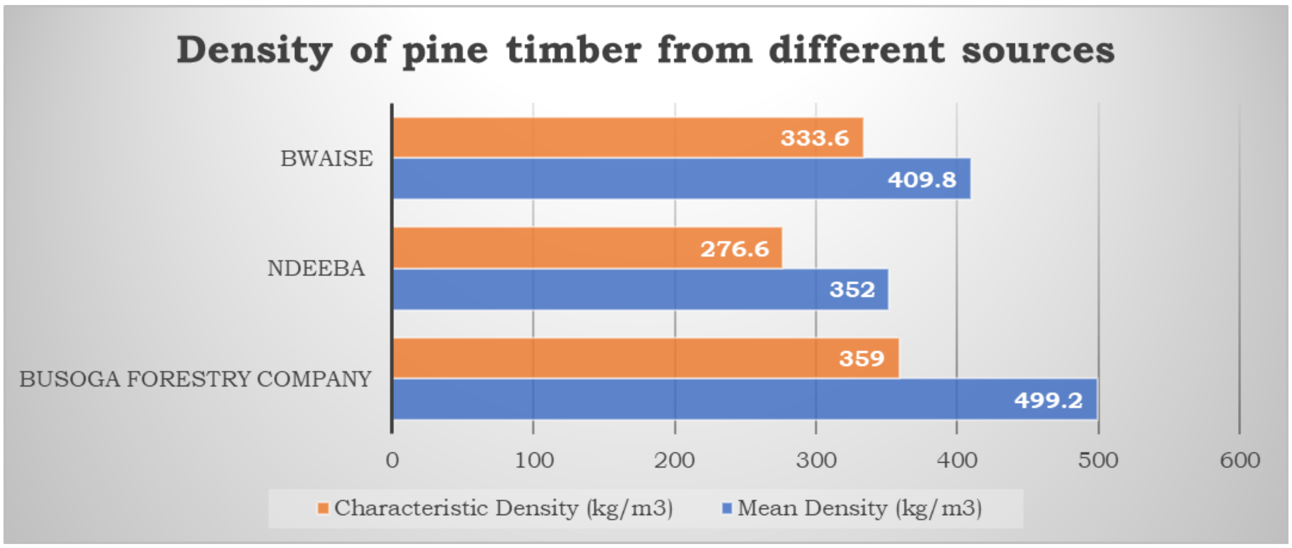

Figure 4-2: Density of pine timber from different Sources

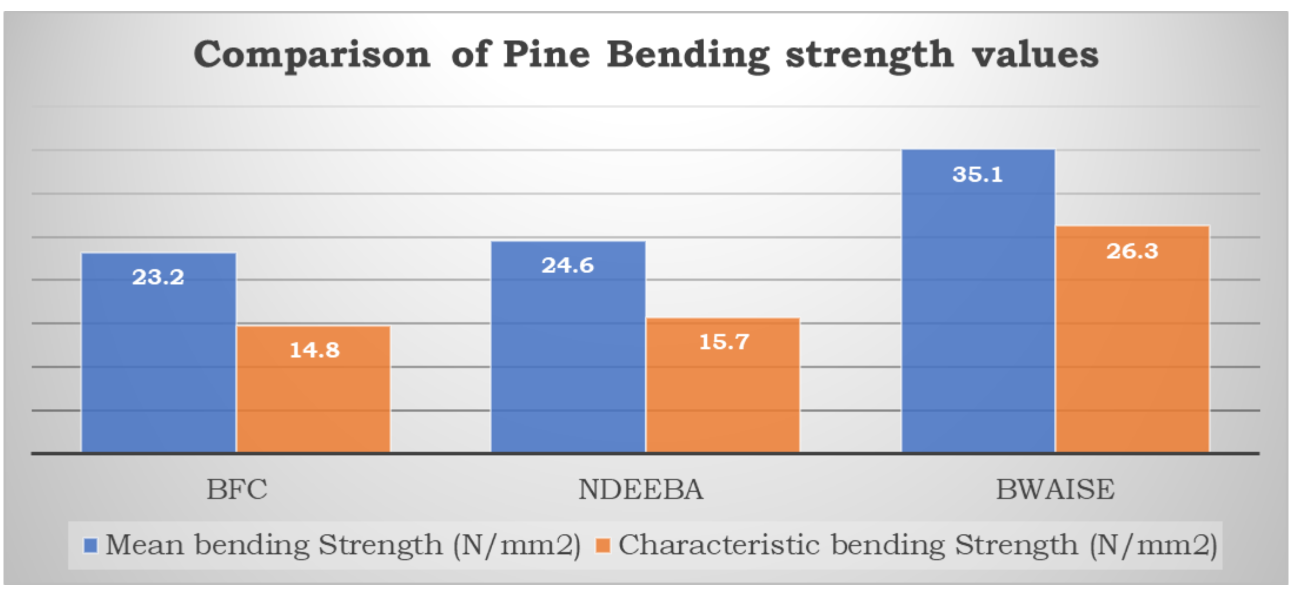

Figure 4-3: Comparison of bending strength of pine timber

The results from BFC, being different from the results from the timber sourced from Ndeeba and Bwaise, show that it is not wise to attach a particular strength class to a specie.

4.2.2 Moisture Content

The measured Timber Moisture Content was very high as compared to what would be specified for structural purposes (15-22% at time of erection). The Moisture content as measured was as high as 75.1%.

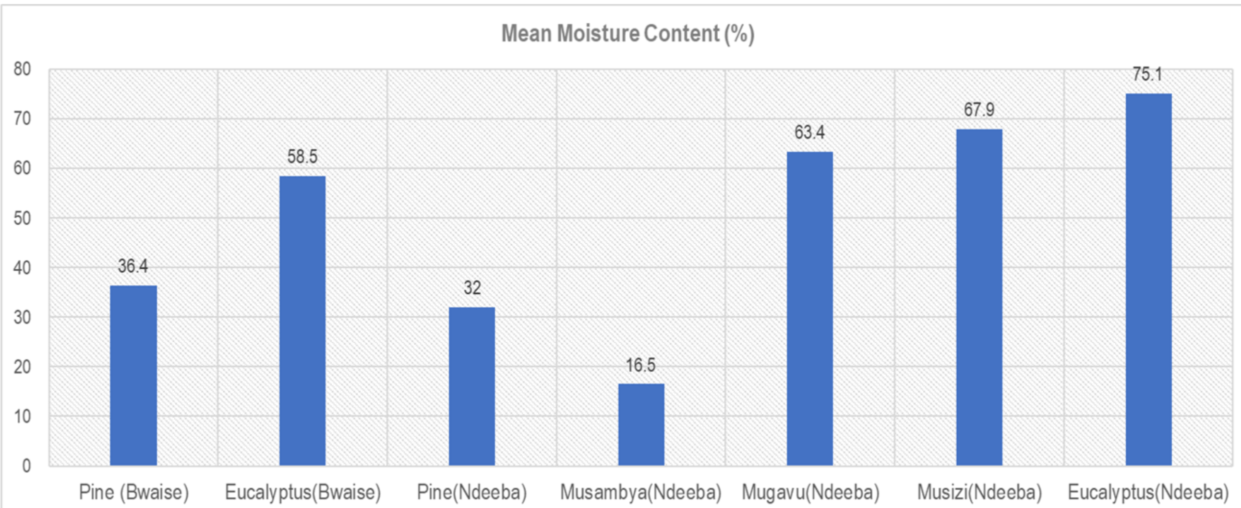

Figure 4-4: Mean moisture content of local common timber species

It is known that the strength of timber reduces with increase in moisture content. It is for this reason that the standards and specifications require a maximum moisture content of timber for use. Schedule 16 Part II of the National Building (Structural Design) Code, 2019 specifies a moisture content of 15-22% of timber at time of erection. By this standard alone, only one of the seven samples would be acceptable for use in construction.

4.2.3 Strength Classification

According to BS EN 338:2016, a timber population may be assigned to a strength class if its characteristic values of strength, mean modulus of elasticity and density equal or exceed the values for that strength class given in Table 4-2 or Table 4-3.

Table 4-6: Allocation of tested pieces to strength classes in accordance with BS EN 338:2016

	***Pinus caribaea* (Pine) (Bwaise)**	**Eucalyptus hybrid GC 540 (Eucalyptus) (Bwaise)**	***Pinus caribaea* (Pine) (Ndeeba)**	***Markhamia lutea* (Musambya) (Ndeeba)**	***Albizia coriaria* (Mugavu) (Ndeeba)**	***Maesopsis eminii* (Musizi) (Ndeeba)**	***Eucalyptus grandis* (Ndeeba)**
Classification on the basis of characteristic bending strength	C24	D35	C14	D35	**D18**	D24	D40
Classification on the basis of 5th percentile modulus of elasticity parallel to the grain	**C14**	D30	C14	**D18**	Out of range	Out of range	D35
Classification on the basis of mean modulus of elasticity parallel to the grain	**C14**	D30	C14	**D18**	Out of range	Out of range	D35
Classification on the basis of mean modulus of elasticity perpendicular to the grain	**C14**	Out of range	C14	**D18**	Out of range	Out of range	D35
Classification on the basis of mean density	C22	Out of range	C14	Out of range	D24	Out of range	Out of range
Classification on the basis of 5th Percentile density	C20	Out of range	Out of range	Out of range	D40	Out of range	Out of range
Designated Class	**C14**	**D18***	**C14***	**D18***	**D18***	**D18***	**D18***

**: Allocated lowest class due to out of range parameters in the classification of timbers.*

From the results, the majority of the hardwoods had density values out of range of the typical as captured by BS EN 338:2016. A possible explanation for this variance is the common practice of harvesting before the trees' rotation age[5].

[55] Rotation age is the age at which trees have reached an optimal size and maturity for harvestings.

For initial design/sizing, C14 and D18 classes can be used subject to confirmation through testing and statistical analysis to obtain characteristic values which are applicable to limit state design.

On the basis of the classification criteria based on bending strength and density, the designated class for the BFC pine would be C14, which is the same proposed for the Bwaise and Ndeeba pine Timbers.

4.3 Durability of Timber

4.3.1 Fungal Attack on Timber in STC

If not properly maintained, wood can suffer from decay damage through the actions of white rot and brown rot fungi of the group Basidiomycetes. Decay is caused by damp wood being attacked by one or more types of fungi. There are various species of fungi which can cause rot, the most common being *Coniophora puteana* and *Serpula lacrymans* known as wet rot and dry rot respectively.

As dry rot usually develops out of sight behind panelling or under floors, it is unlikely it will be noticed at the initial stage. Where wet rot exists, there is usually some underlying failure in a building structure which is allowing excessive moisture to penetrate into wood such as a leaking pipe, gutter or roof. *This can be a cause of concern if there are plumbing failures within the slab.*

Dealing with rot

a) Remove the source of moisture which created the conditions for the rot to develop in the first place. If the moisture level drops below 20%, decay will cease. In a well-maintained building, the moisture level in timber will not rise above 20% and rot has little chance of developing; however, even in a well-maintained building area which are not properly ventilated can quickly become damp if in contact with water or humid air.
b) Where rot has occurred, it is important to remove any timber which has

been affected. This will reduce the likelihood of the rot reappearing. *Maintenance of this nature is difficult with STC slab systems.*

c) Once the source of moisture has been removed, it is important to allow any affected area to dry out as quickly as possible. It may involve the use of dehumidifiers or ventilation.

d) Remedial treatment with chemical preservatives can be done for the affected part of timber. Application of preservative to the affected area of timber is a simple short-term treatment.

4.3.2 Pests Attack on Timber

The STC system uses timber as the main structural element supporting the floors. One of the causes of timber failure could be pest attacks. These could be insects such as termites, beetles, bees, moths, other insects, and rodents. Of these, termites and rodents present serious threats that warrant attention.

Attack by Termites

Structural failure due to termite damage can be prevented through the use of regular inspections and treatments. *This may be difficult where timber is enclosed such as in the STC system.*

In Uganda, termites are found almost in all regions of the country, and untreated timber often used in construction is highly susceptible to termite attacks (Nakabonge and Matovu, 2021; Ssemaganda et al, 2011).

Given the extensive and serious damage caused to buildings by termites in the tropics, it is necessary to take all relevant precautions in the design and building specifications to incorporate anti-termite constructional measures in all buildings.

Practices leading to termite entry into buildings include:

- Poor workmanship relating to timber selection and placement, foundation work, interior and exterior wall work and roof sealing

- Use of unseasoned or low durability untreated wood
- Use of timber components in badly ventilated dark places
- Structural timber in contact with the ground

The simplest method of dealing with structural damage due to termites would be building the structures in a preventative manner to avoid initial termite damage. The following steps are hereby recommended as preventive and corrective measures in combating the harmful impact of termites (Olaniyan et al, 2015):

a) Timber treatment: Treatment with creosote or other appropriate wood preservatives:
b) When termites have already penetrated a building, the first action is usually to destroy the colony with insecticides before removing the termites' means of access and fixing the problems that encouraged them in the first place.
c) Avoid contact of susceptible timber with ground by using termite-resistant concrete, steel or masonry foundation with appropriate barriers such as a damp-proof course (DPC).
d) Termite barriers (whether physical, poisoned soil, or some of the new poisoned plastics) should be provided to prevent the termites from gaining unseen access to structures.
e) Use timber that is naturally resistant to termites such as Syncarpia glomulifera (Turpentine Tree), Callitris glaucophylla (White Cypress) or Markhamia lutea (Musambya)

4.3.3 Attack of Timber by Rodents

In construction, apart from fire and termites, rodents can also do significant damage to timber in buildings. Rodents can severely undermine and compromise the performance of timber in construction. The damage may lead to increased risks of fire outbreaks, for example, when they damage electrical wiring and cause timber decay, in addition to other health problems for the residents in buildings (Murphy and Todd, 2008). Therefore, the timber used in STC should be protected using rodent-proof construction techniques.

4.4 Cost of Timber for Use in Structural Applications

Although timber of several species is available unseasoned, only *P. caribaea* timber is available on the market kiln-dried. The cost of the unseasoned and kiln-dried P. caribaea timber is shown in Table 4-7.

Table 4-7: Comparison of cost of kiln-dried and unseasoned P. caribaea timber

Board size	**Price per piece of Kiln-dried *P. caribaea* timber from large producers (UGX)****	**Price per piece of Unseasoned *P. caribaea* timber from Ndeeba and Bwaise hotspots (UGX)****	**Percentage Difference- saving by using unseasoned timber**
50 mm x 100 mm x 3.6 m	10,000	9,000	10%
50 mm x 150 mm x 3.6 m	15,000	12,000	20%
50 mm x 200 mm x 3.6 m	20,000	24,000	-20%

*** February 2024 Costs*

The cost of kiln-dried timber is comparable to that of unseasoned timber, as can be seen in Table 15. The large sizes may even be cheaper at large suppliers' yards as compared to local timber sheds, as demonstrated in the costing of the 50mm*200mm*3.6m.

5 STRUCTURAL STEEL

Structural steel is a critical component in the construction of STC structures. It is used in the construction of the vertical elements and in the construction of the beams that support the timber framework.

5.1 Declared Properties of Structural Steel Produced in Uganda

The strength of structural steel (Rectangular/Square Hollow Sections & I-beams) on the Ugandan market is not specified on the sections. The following paragraphs give information obtained from product data sheets of some of the major steel producers in Uganda.

5.1.1 Roofings Uganda

Roofings Uganda produces hollow sections. Properties of the round, square and rectangular sections including size, moment of inertia, radius of gyration and modulus of section are given in their catalogue. According to their catalogue, the declared grade is Grade 210, with a tensile strength of 340MPa, Minimum Yield stress of 210 MPa, Minimum elongation of 24%, a carbon content of 0.2 and phosphorus content of 0.25. These limits correspond to the limits prescribed by US EAS 134:2019 Cold rolled steel sections-Specification, which specifies the requirements and section properties for cold rolled steel sections for use in structural applications.

No grade is given for the I-Beams (IPE sections) that they trade in. *A catalogue for the public should at the bare minimum have information on the grade of the product for quality assurance.*

5.1.2 Steel and Tube Industries (STIL)

Whereas Steel and Tube does not produce plates themselves, they claim to only use sections which are Grade 43A or equivalent. According to NBC, 2019, Grade 43 is equivalent to Grade S275 and meets requirements of US ISO 630-1.

For the hollow sections manufactured by STIL, i.e. round, rectangular, square and D-pipe sections, the standard they follow is US EAS 134 which prescribes Grades 210, 250 and 360. The lowest grade (Grade 210) should have a tensile strength of 340MPa, Minimum Yield stress of 210 MPa, Minimum elongation of 24%, a carbon content of 0.2 and phosphorus content of 0.25.

5.1.3 Tembo Steels Uganda Limited

Tembo steels produces hot rolled sections and cold rolled hollow sections. The declared Grade is Grade 360, minimum yield stress of 400 MPa, minimum tensile stress of 480 MPa, Maximum Carbon content of 0.2%, maximum silicon content of 0.18%, maximum manganese content of 0.6%, Maximum Phosphorus content of 0.05%, and maximum sulphur content of 0.04%. This is largely in conformance with US EAS 134.

5.1.4 Doshi Steels

Doshi steel has hollow sections and I-sections on the market. For rectangular and Square hollow sections, they market both Grade S355 and Grade 210. Results from their testing reveal that the steel meets the requirements for Tensile strength, Yield Strength, Elongation and the Carbon, Manganese, Phosphorus and Sulphur contents are lower than the maximum prescribed. They also have S355 plates and I-beams on the market, with in-house tests showing that they meet the strength requirements (Yield and Tensile). For the angles, the declared average Carbon Equivalent Value is 0.25 which is less than the recommended 0.4 for weldable steels. Results shared for channels also show that the strength is higher than the absolute minimum of the lowest grade of 210.

5.2 As-tested Properties of Structural Steel Commonly Used in STC in Uganda

5.2.1 Mechanical Results – Yield and Tensile Strengths Values

The results on yield strength fell within the range of 270-520 MPa, indicating that it is feasible to achieve the expected yield values. Notably, structural steel yield values typically range from 235 to 450MPa.

According to the National Building (Structural Design) Code, 2019, Par.30(5), general steel grades 43, 50 and 55 shall be used for Structural Steelwork and shall have minimum corresponding design strength indicated in Table 5-1. The implication is that structural steel in Uganda should have a minimum yield strength of 275 MPa.

Table 5-1: Minimum yield strength of structural strength

Grade	**Thickness of Material (mm)**	**Sections, Plates, Hollow Sections (N/mm^2)**	**Other Properties**
43	16	275	Modulus of elasticity = 205 x 10^3 N/mm^2
	40	265	
	63	255	
	100	245	Poisson's ratio = 0.3
50	16	355	
	40	345	
	63	340	Coefficient of linear expansion = 12 x 10^{-6} ^0C
	100	325	
55	16	450	
	40	439	
	63	415	
	100	400	

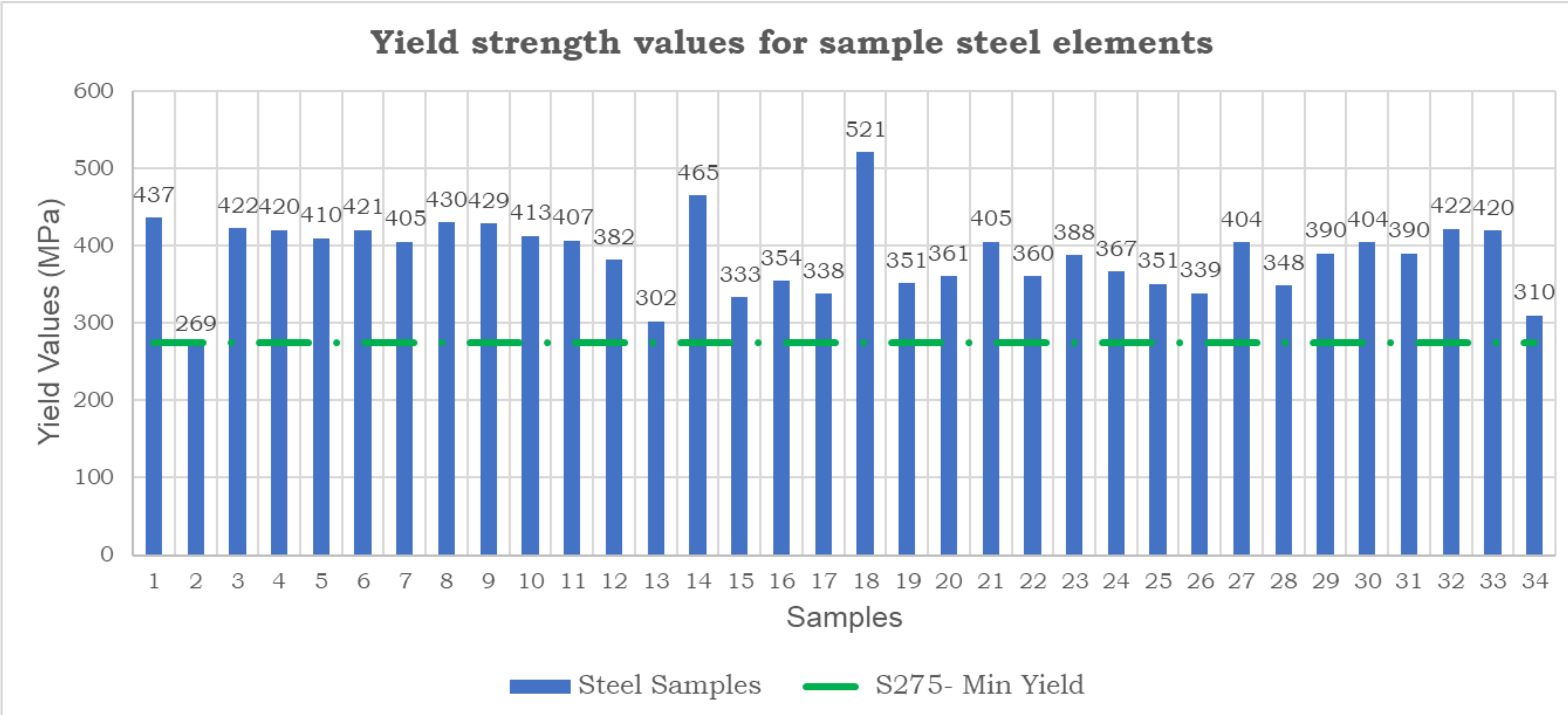

Figure 5-1: Yield Values of tested samples (MPa)

33 of the 34 samples achieved a yield strength value of more than 275MPa, with a characteristic value of 332 MPa.

The National Building (Structural design) code, 2019 requires that structural steel for general structural use should conform to Uganda Standard US ISO 630-2:2011, Structural Steels-Part 2. According to US ISO 603-2, the minimum tensile strength is 410 MPa for S275, 470 MPa for S355 and 550 MPa for S450.

Nine of the 34 samples failed to achieve the minimum tensile strength of 410MPa, as shown in Figure 5-2. The Characteristic value of tensile strength from the samples tested was only 356 MPa which is lower than 410 MPa.

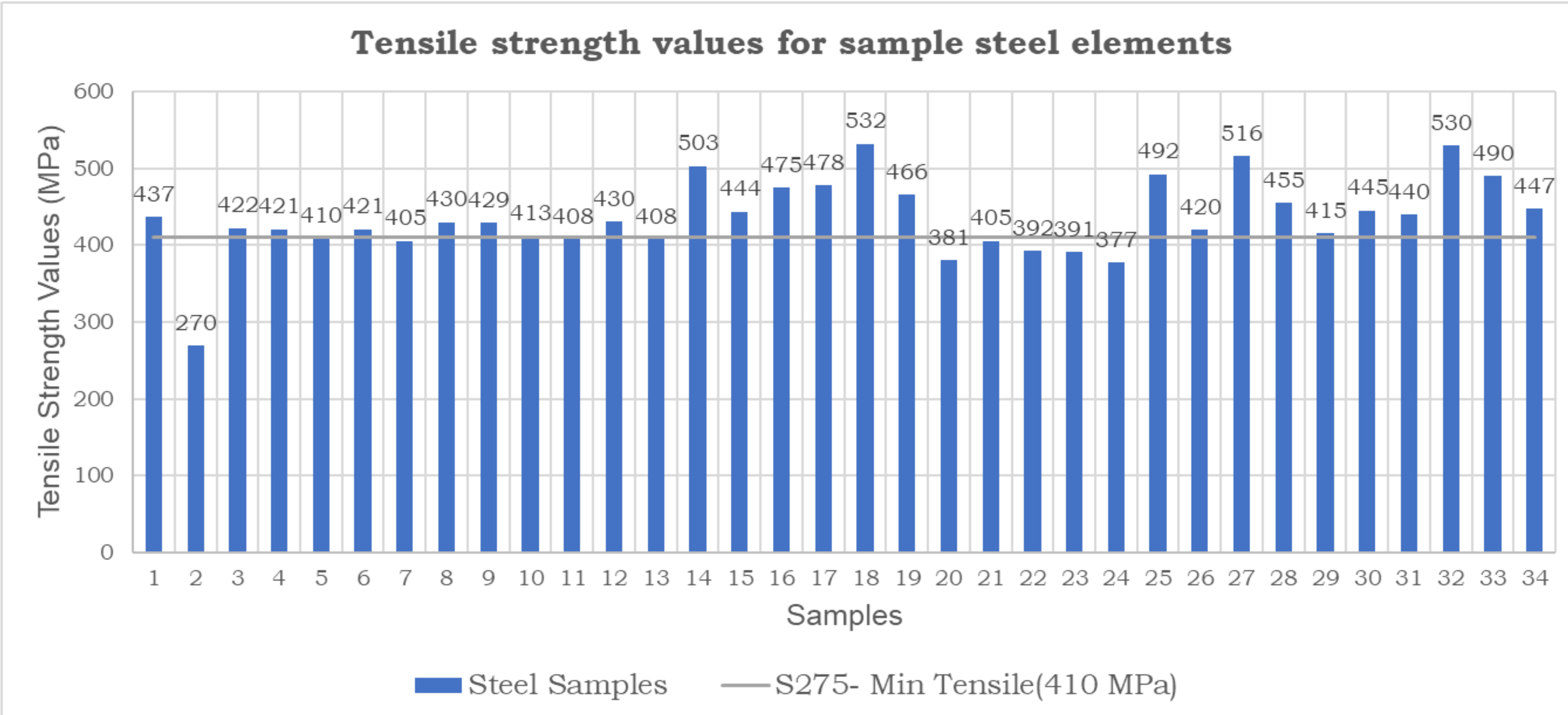

Figure 5-2: Tensile strength for the steel elements

5.2.2 Mechanical Results – Elongation Values

Elongation values provide insights into the potential ductility of the material. Higher values indicate greater ductility, implying the material can undergo significant stretching or elongation before breaking. Figure 5-3 shows the percentage elongation as tested on the samples.

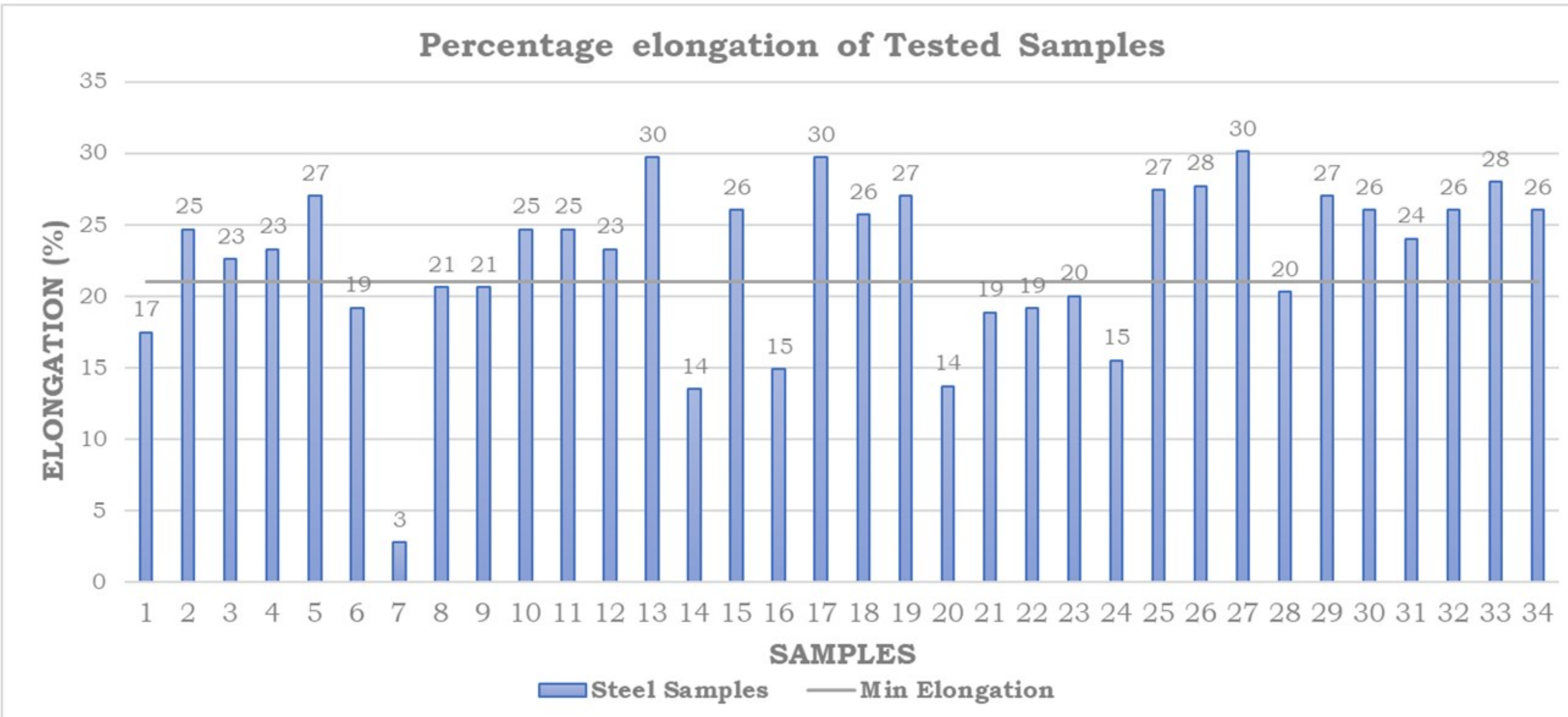

Figure 5-3: Percentage Elongation of Tested Steel Samples

According to US ISO 603-2, the elongation for S275 is 21-23%, 20-22% for S355 and 17% for S450 for thicknesses up to 40mm. 16 of the 34 samples (47%) did not meet the minimum required for elongation, which implies low ductility. In seismic design situations, ductility is a critical design criterion to consider.

5.2.3 Chemical Composition Results

There are several reasons for conducting chemical tests on steel. These tests are not only used to identify the composition of the steel and confirm its grade to ensure compliance with specified standards, but they also help assess its weldability and durability.

Various industry standards and specifications define acceptable chemical compositions for different steel grades. The key parameters of interest are as summarised in Table 5-2.

a) Carbon Content

The carbon content is one of the aspects that significantly affects the properties. The higher percentages generally increase the hardness and strength but sacrifice the ductility.

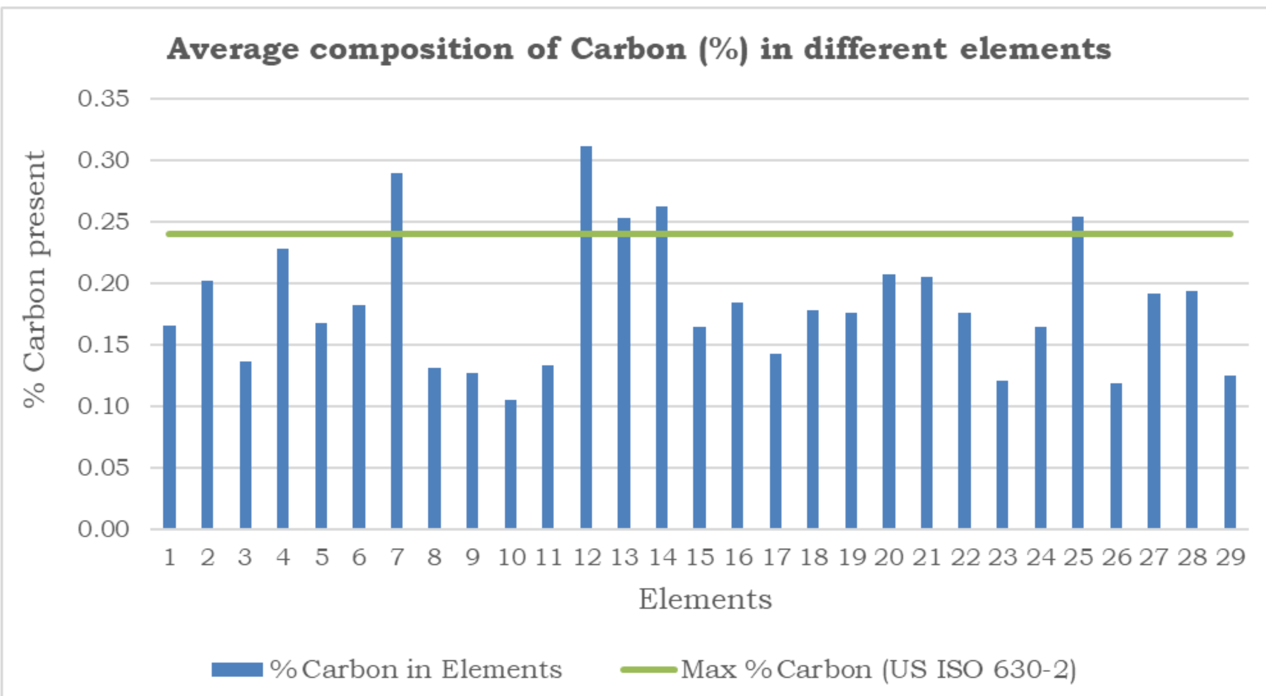

Figure 5-4: Carbon composition in tested steel samples

Five of the 29 samples had higher carbon content than the maximum recommended for ductility.

Table 5-2: Significance of key elements in steel composition

Element	Importance	Limits and References
Carbon	Increasing carbon content increases strength, hardness and lowers ductility	• 0.15 – 0.30 % (ASTM) · > 0.3% - Ductility is too low · < 0.15% - strength not satisfied • Max (0.22%, 0.25%, 0.23%) for S235, S275, and S355- EU • US ISO 630-2: Max 0.18-0.24% for S275, 0.2-0.27% for S355; 0.2-0.23% for S450
Manganese	• Effects similar to carbon (strength and hardness) • Also decreases ductility and weldability in high quantities	• 0.50 – 1.70 % (ASTM) • 1.6% max for S235, S275, and S355 • US ISO 630-2: 1.5-1.6% for S275, 1.6-1.7% for S355; 1.7-1.8% for S450 • US ISO 4951-1: <1.7%
Silicon	One of the principal deoxidizers for structural steel	• Normally appears in amounts less than 0.4% • US ISO 630-2: <0.55-0.6% • US ISO 4951-1: <0.6%
Phosphorus and Sulphur	• Generally undesirable in structural steel. • Both reduce the ductility of the material. • Their detrimental effect on weldability and toughness is significant	• 0.04 % max (ASTM), 0.035% Max (EU) -Phosphorus • US ISO 630-3: Phosphorous <0.025-0.035% Sulphur: <0.02-0.03% • 0.05% max (ASTM), 0.035%, max (EU) – Sulphur • US ISO 630-2: Phosphorous 0.025-0.045% for S275, 0.025-0.045% for S355, 0.03-0.04% for S450; Sulphur 0.025-0.045% for S275, 0.025-0.045% for S355, 0.03-0.04% for S450 • US ISO 4951-1: Phosphorus and Sulphur <0.035%

Element	Importance	Limits and References
Copper, Nickel, Chromium, Molybdenum, and Tin	• They all primarily increase the corrosion resistance of the material • Mo, Increases strength at higher temperatures	• US ISO 630-3: Cu<0.6%, Ni <0.35%(S275) and 0.55% (S355), Cr <0.35%, Mo<0.13% • US ISO 630-2: Cu<0.55-0.6% • US ISO 4951-1: Cu 0.35-0.7%, Nickel <0.8%, Chromium <0.3%
Vanadium, Columbium, and Titanium	• These are strength enhancing elements and are added singly or in combination. • In very small quantities, their effects are very significant hence termed micro-alloys	• ASTM steel have vanadium content between 0.02% to 0.15% (A572, A588) and (0.03 to 0.08% for A514) • US ISO 630-3: Vanadium <0.1% (S275), <0.12% (S355) • US ISO 630-3: Titanium <0.06% • US ISO 630-3: Columbium<0.06% • US ISO 4951-1: V <0.2%, Columbium <0.05%, Ti <0.05%
Nitrogen	Nitrogen can reduce the ductility of structural steel when present in excessive amounts	• US ISO 4951-1: N < 0.025% • US ISO 630-2:2011 Max 0.014%[6] • US ISO 630-3:2012 Max 0.017%
Carbon Equivalent Value (CEV)	As the CEV increases, the weldability of the materials decreases. Materials with low carbon equivalent offer excellent weldability with minimum precautions.	According to US ISO 630-3:2012, Carbon equivalent value, CEV=C+Mn/6+ (Cr + Mo +V)/5 + (Ni+Cu)/15 • US ISO 630-3: Max CEV 0.4% for S275 and 0.43% for S355. • US ISO 630-2: Max CEV 0.4% for S275, 0.45% for S355 and 0.47% for S355.

b) Manganese Content

Manganese is the other element that affects strength. All the elements tested had manganese less than the maximum prescribed by the standards.

[6] The Maximum value of Nitrogen does not apply if the chemical composition shows a minimum total Al content of 0.015% or if sufficient other N-binding elements are present.

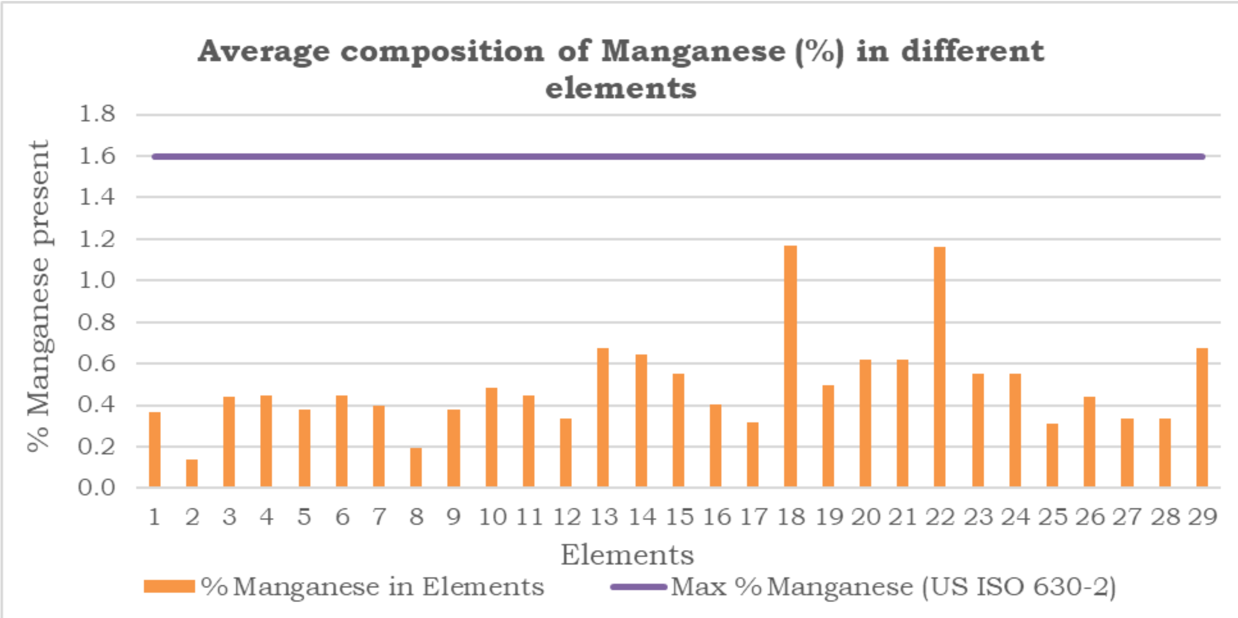

Figure 5-5: Manganese composition in the Steel elements

c) ***Phosphorous and Sulphur content***

Phosphorus and Sulphur affect the ductility and weldability in particular if found in high amounts.

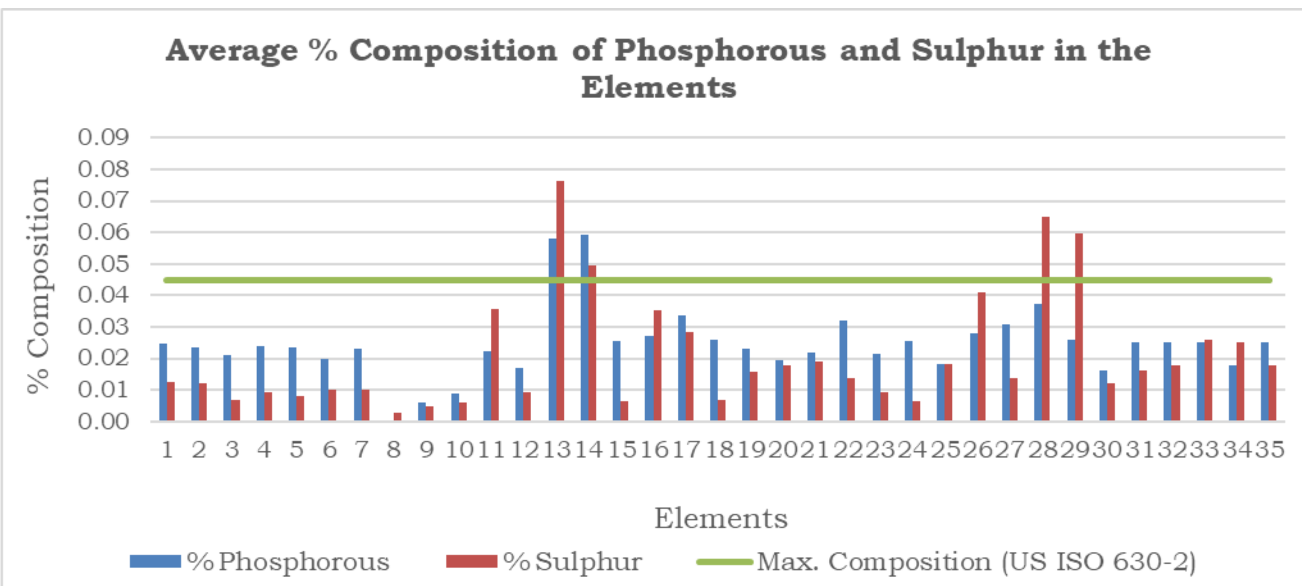

Figure 5-6: Phosphorous and Sulphur Composition in the Steel

2 of the 35 samples had phosphorus amounts higher than the maximum recommended by the standards. 3 of the 35 samples had sulphur amounts

higher than the maximum recommended by the standards.

d) Silicon and Copper content

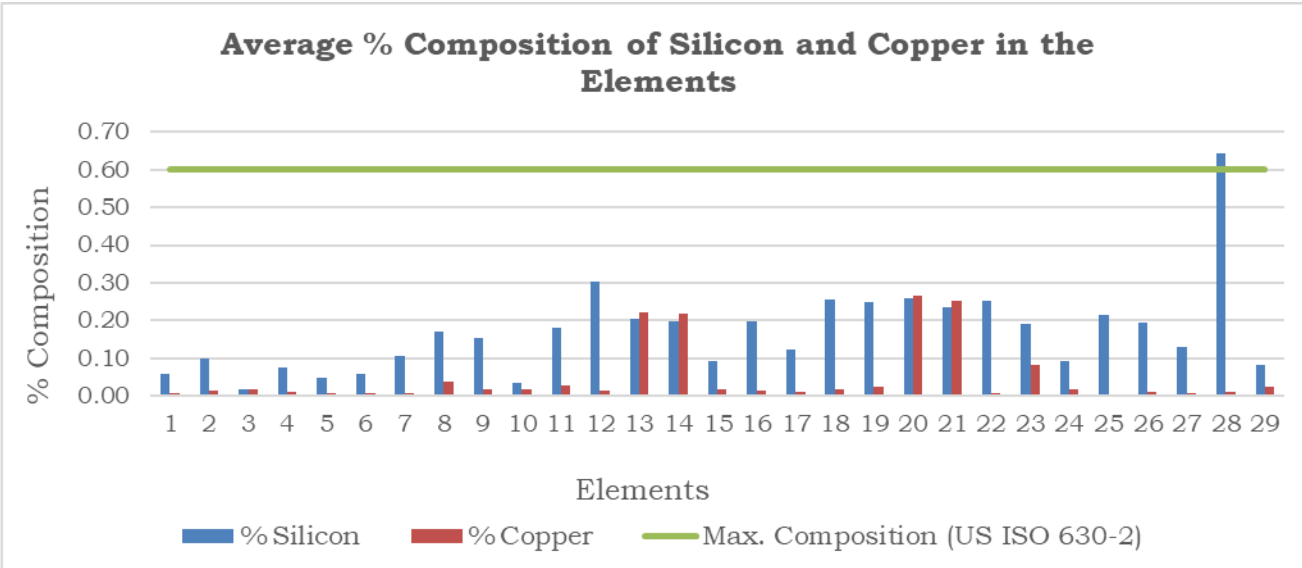

Figure 5-7: Silicon and Copper Composition in the Steel

All samples had a Copper composition less than the maximum permissible. Only one of the 29 samples had a silicon composition higher than the standard maximum.

e) Nickel and Chromium content

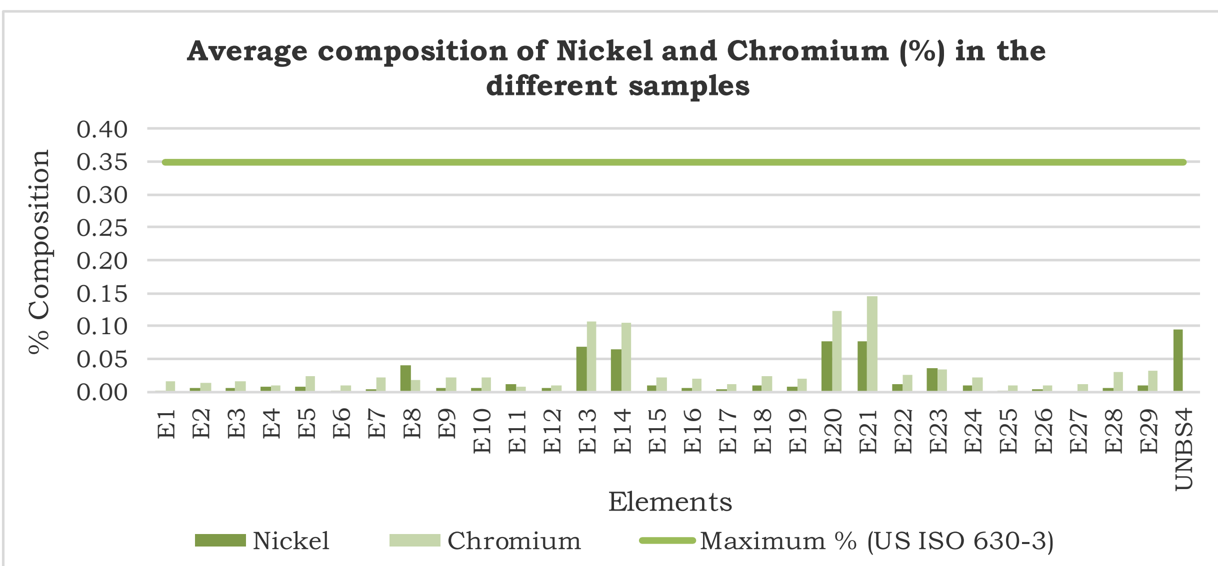

Figure 5-8: Composition of Nickel and Chromium in the steel elements

All samples had Nickel and Chromium compositions less than the maximum permissible.

f) Molybdenum content

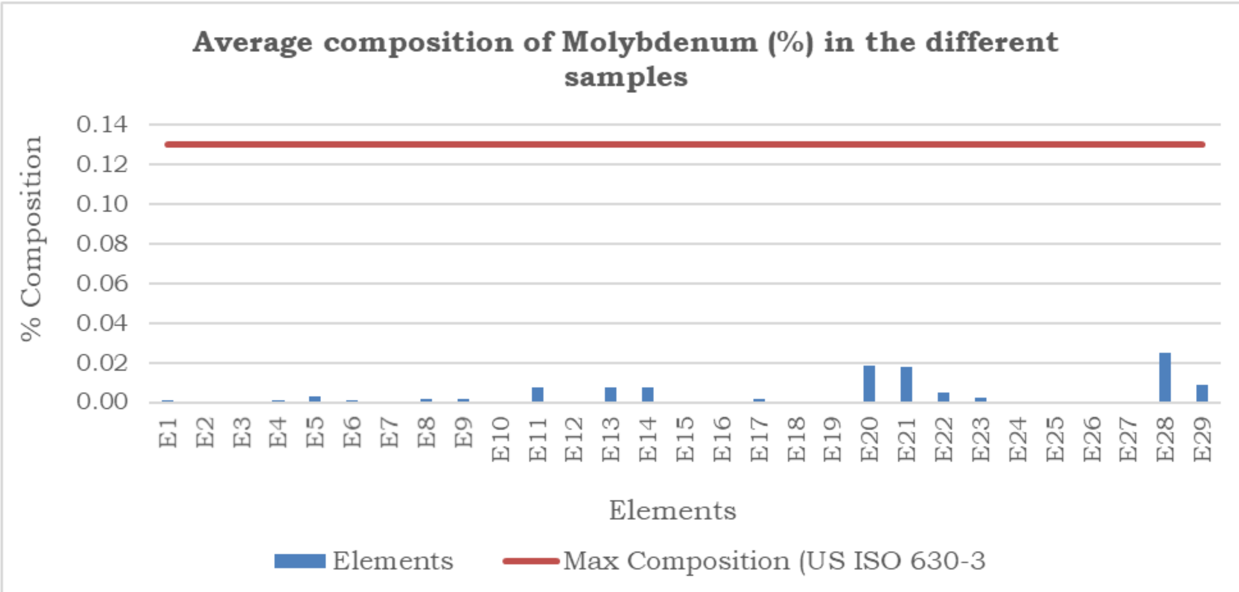

Figure 5-9: Composition of Molybdenum in the steel elements

All samples had Molybdenum composition less than the maximum permissible.

g) Vanadium Content

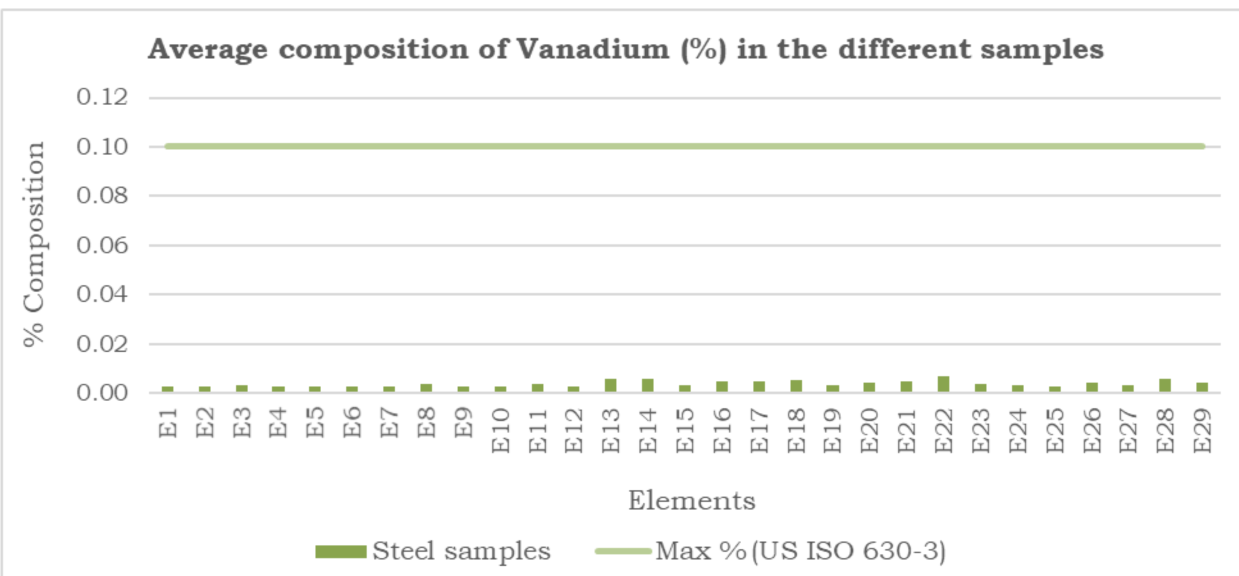

Figure 5-10: Composition of Vanadium in the steel elements

All samples had Vanadium composition less than the maximum permissible.

h) Niobium (Columbium) and Titanium content

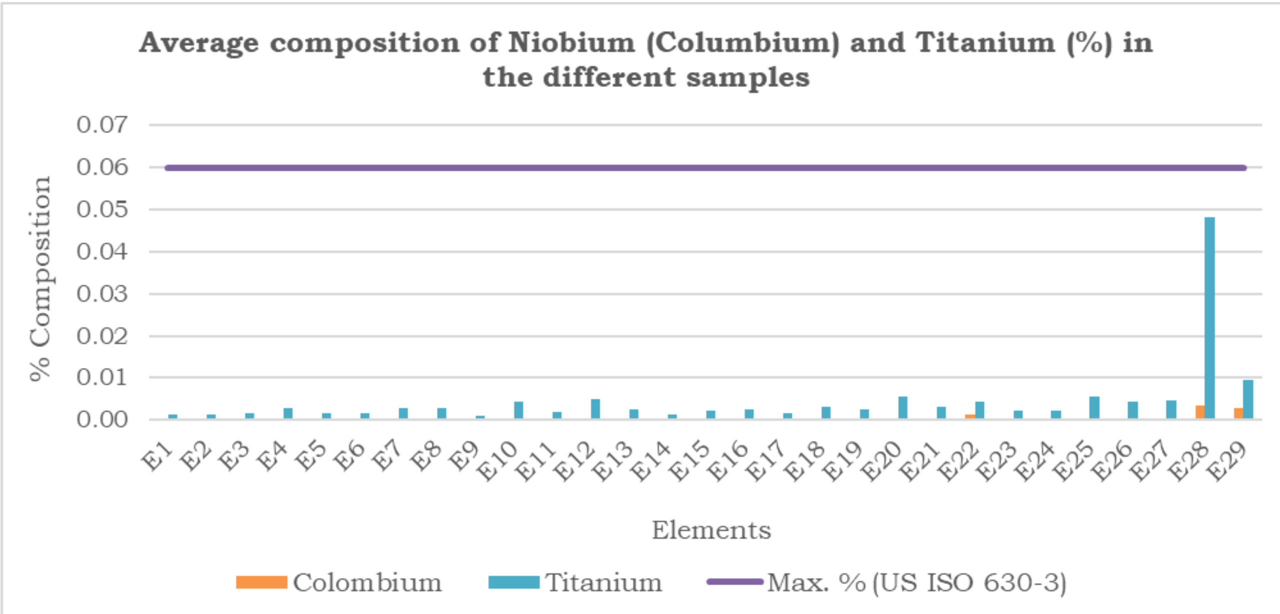

Figure 5-11: Composition of Niobium/Columbium and Titanium in the steel elements

All samples had Columbium and Titanium compositions less than the maximum permissible.

i) Nitrogen content

The Nitrogen composition of all 29 samples had a nitrogen content in excess of 0.036%, against a maximum recommended of 0.025% (US ISO 4951-1), 0.014% (US ISO 630-2) and 0.017% (US ISO 630-3).

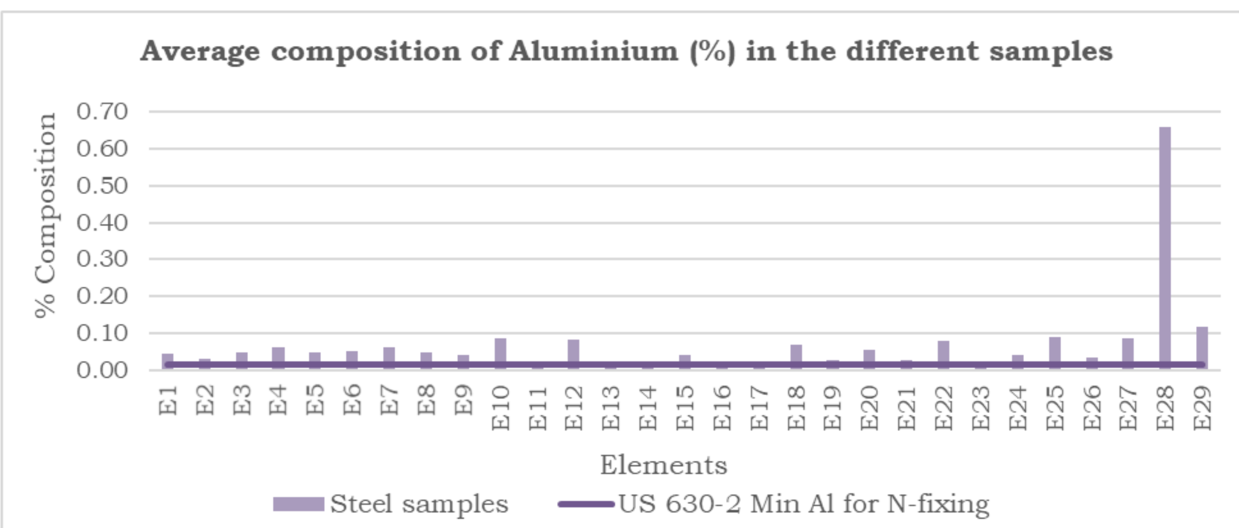

Figure 5-12: Aluminium composition in steel samples

It should be noted that according to US ISO 630-2, the limit does not apply if the chemical composition shows a minimum total Al content of 0.015% or if sufficient other N-binding elements are present. The results in Figure 5-12 show that the aluminium content is higher than the minimum required for Nitrogen-fixing.

j) Carbon Equivalent Value

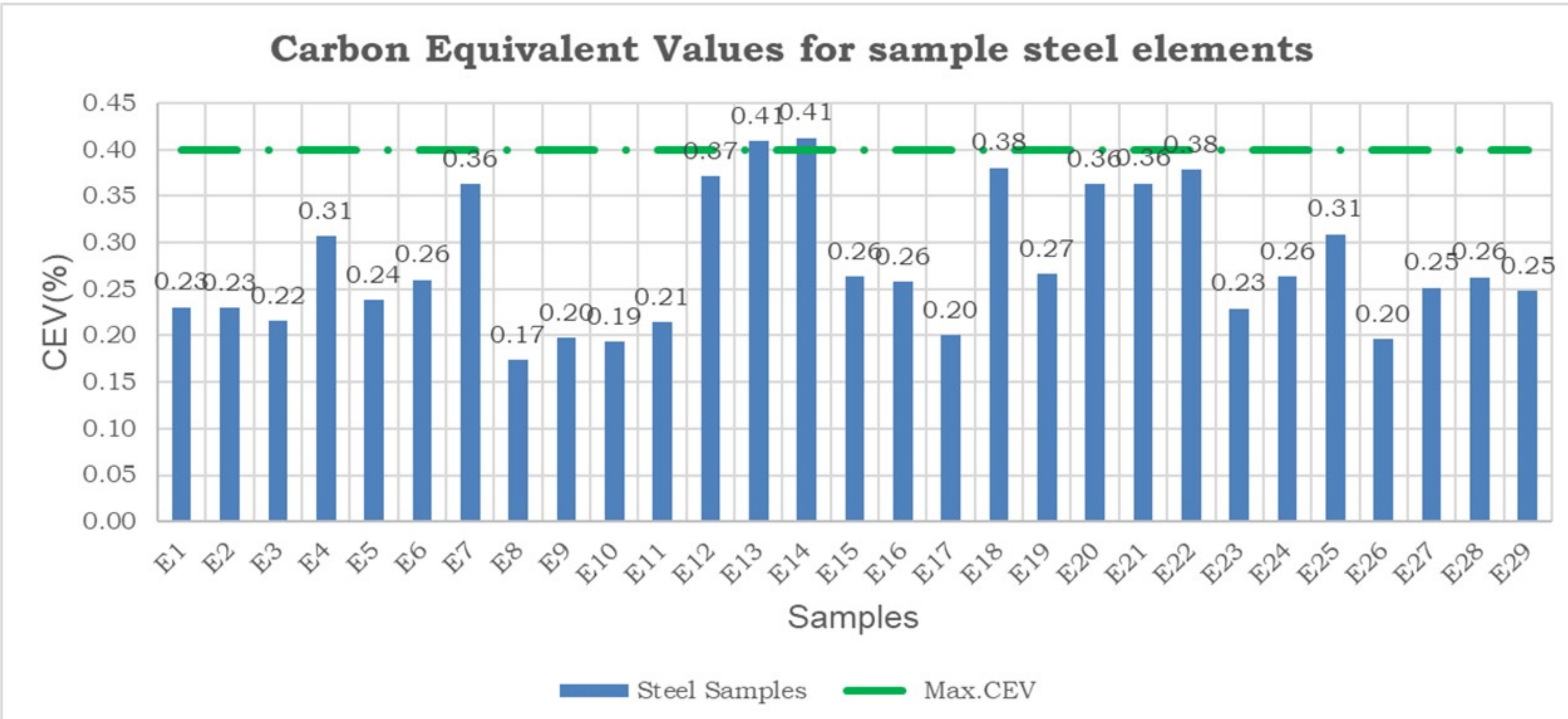

Figure 5-13: Carbon Equivalent Values for the Tested Steel Samples

Only two of the 29 samples returned Carbon Equivalent Values higher than the recommended. It can then be inferred that generally structural steel as sampled are weldable.

6 CONCRETE PRACTICES IN UGANDA

6.1 Compressive Strengths Results

6.1.1 Lira site

A mix design ratio of 1:2:4 Cement (CEM IV) was used and yielded the following results:

Table 6-1: Lira site compressive strength results

Item No.	Structural Element	28th day Strength (kPa)
1	Slab	27.4
2	Ramp	27.9
3	Column 1	25.3
4	Column 2	28.0
5	Column 3	26.9
6	Column 4	28.5
7	Column 5	36.4
	Average	**28.6**

The mix ratio of 1:2.1:4.1, in consideration of constituent materials for the select region as highlighted, yielded compressive strength results greater than the C25 target strength.

6.1.2 Fort Portal Site

A mix design ratio of 1:1.5:3 with CEM IV cement was used and yielded the results as shown in Table 6-2.

The mix ratio of 1:1.5:3, in consideration of constituent materials for the select region as highlighted, yielded compressive strength[7] results marginally greater than the C25 target strength.

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[7] *Cube results must be assessed for validity using the following rules for 20 N/mm^2 or above:*

Table 6-2: Fort Portal site compressive strength results

Item No.	Structural Element	Average Compressive strength after 7 days	*Strength achievement at 7 days*	Average Compressive strength after 28 days
1	Staircase and Ramp	18.3	*72.0%*	25.4
2	RCC Column Bases and strip foundation slump	18.5	*67.0%*	27.6
3	First Floor Slab	20.7	*78.7%*	26.3
4	First Floor Slab	17.8	*67.4%*	26.4
	Average	**18.8**	***71.2%***	**26.42**

6.1.3 Luwero Site

A different cement type of CEM II than the typical one used (CEM IV) was used and yielded the following results at the 7th day and 28th day.

Utilising CEM II Concrete, and in consideration of constituent materials for the select region as highlighted, a mix ratio of 1:3:4 (Equivalent Mix 1:2.3:3.1) yielded the C25 target strength.

The slump test results were fairly in range and generally depicted a fairly consistent workability of the concrete. The results for the slump test were within the limits provided for in Schedule 7 Part I of the National Building (Structural Design) Code, 2019.

• A cube test result is said to be the mean of the strength of two cube tests. Any individual test result should not be more than 3 N/mm2 below the specified characteristic compressive strength.

• When the difference between the two cube tests (i.e. four cubes) divided by their mean is greater than 15%, the cubes are said to be too variable in strength to provide a valid result.

• If a group of test results consists of four consecutive cube results (i.e. eight cubes), the mean of the group of test results should be at least 3N/mm^2 above the specified characteristic compressive strength.

A sample consists of two concrete cubes for each test result. Where results are required for 7-and 28-day strengths, four cubes should be prepared.

Source: BS 5328: Part 1:1997; BS EN 206-1: 2000.

Table 6-3: Luwero site compressive strength results

Mix Design 1:3:4 (Equivalent Mix 1:2.3:3.1)					
Item	**Slump**	**Location**	**Average Compressive Strength after 7 days**	***Percentage strength gain at 7 days***	**Average Compressive Strength after 28 days**
1	57,53	RMB Service pit steps	20.7	*72.1*	28.7
2	66,54,53	Gate house slab	23.3	*89.6*	26.0
4	68,54,60	Routine Maintenance (Workshop)	15.5	*59.2*	26.2
5	62,54,55	Routine Maintenance (Workshop)	16.2	*66.4*	24.4
	Average		**18.9**	***71.9***	**26.3**

6.1.4 Yumbe Site

A mix design ratio of 1:2.3:3.1 was used with CEM IV and yielded results which met the target strength of 25MPa.

Table 6-4: Yumbe site compressive strength results

Item No.	**Slump**	**Target Strength**	**Structural Element**	**Average Compressive strength after 7 days (MPa)**	***Strength achievement at 7 days***	**Average Compressive strength after 28 days (MPa)**
1	58, 55	C25	Bases	18.7	*64.0*	29.2
2	62, 55	C25	strip	19.1	*72.1*	26.5
3	62, 54, 66	C25	Plinth Columns	11.7	46.1	25.4
Average				**16.5**	**61.0**	**27.0**

6.1.5 Tororo Site

A mix design ratio of 1:1.5:3 was used with CEM IV and yielded the following results on the 7th day and 28th day.

Table 6-5: Tororo site compressive strength results

Item No.	Location	Average Compressive strength after 7 days	*Strength achievement at 7 days*	Average e Compressive strength after 28 days
1	Ground Beam Block D	20.5	*70.7%*	29
2	Middle Ground Slab of Block A & B	22.7	*77.5%*	29.3
3	Middle Ground Slab of Block C & D	24.2	*77.6%*	31.2
4	First Floor Beams Block B	20.6	*71.3%*	28.9
5	First Floor Beams Block A	19.6	*72.3%*	27.1
	Average	**21.5**	***74%***	**29.1**

A mix design of 1:1.5:3 yielded consistent and favourable results that exceeds the target strength of C25 concrete.

6.1.6 Mengo (Kampala) Site

A mix design ratio of 1:1.6:3.1 was used at the Mengo site with CEM IV and yielded the following results on the 7th day and 28th day.

Table 6-6: Mengo site compressive strength results

Item	Location	Average Compressive Strength after 7 days	*Percentage strength gain at 7 days*	Average Compressive Strength after 28 days
1	Ground Floor Columns	26.3	*78.0*	33.7
2	1st Floor Slab	17.6	*57.3*	30.7
3	1st Floor Columns	9.4	*44.3*	21.2
4	2nd Floor Slab	17.6	*65.7*	26.8
	Average	**17.7**	***63.1***	**28.1**

Apart from the 1st floor columns, the mix yielded favourable results that exceeded the target strength of C25 concrete.

6.1.7 Results Summary

The results from the various mix ratios provided Compressive strength results ranging between 26 MPa and 29 MPa (See Table 6-7 and Figure 6-1). The desired target strength of 25 MPa was achieved with different mixes which underscores the need for a project specific mix design and trial mixes for each batch of materials delivered, rather than adopt an off-shelf mix design ratio, which is also many times misinterpreted by artisans.

The concrete from different sites had at 7 days also gained between 61% and 74% of the 28-day strength (See Figure 6-2). The practice of assuming that the concrete gains about 70% of the 28-day strength, may not be applicable in all circumstances. Factors like cement type (Pozzolanic or OPC) as well as other factors such as materials may affect that strength gain. This further underscores the need for concrete mix designs for every project, and for every batch of materials delivered.

Table 6-7: Overall site concrete compressive strength results summary

S/N	**Site Location**	**Mix Ratios by Volume**	**Method of Mixing**	**Cement type**	**Average Compressive strength at 7 days (N/mm²)**	*Percentage strength gain at 7 days*	**Average Compressive strength at 28 days (N/mm²)**
1	Yumbe	1:2.3:3.1	Machine Mixing	Tororo CEM IV	16.5	***61.0***	27.0
2	Fort Portal	1:1.5:3	Machine Mixing	Tororo CEM IV	18.8	***71.2***	26.4
3	Tororo	1:1.5:3	Machine Mixing	Tororo CEM IV	21.5	***74.0***	29.1
4	Mengo (Kampala)	1:1.5:3.1	Machine Mixing	Tororo CEM IV	17.7	***63.1***	28.1
5	Lira	1:2:4	Machine Mixing	Tororo CEM IV			28.6
6	Luwero	1:2.3:3.1	Machine Mixing	Tororo CEM II	18.9	**71.9**	26.3

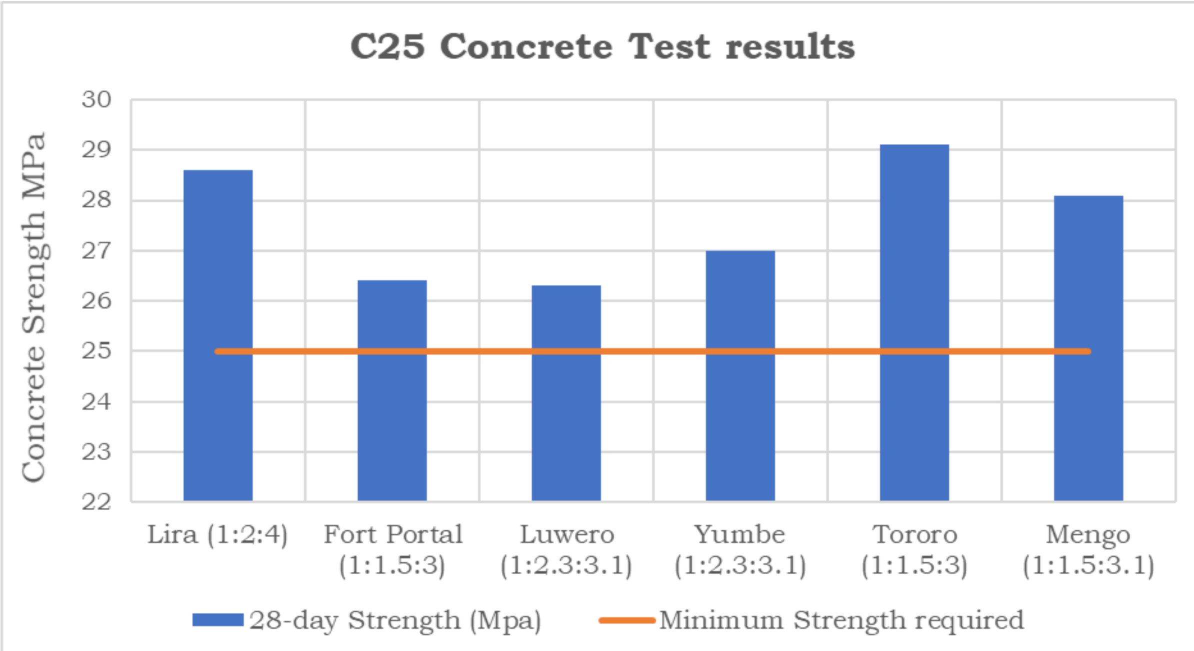
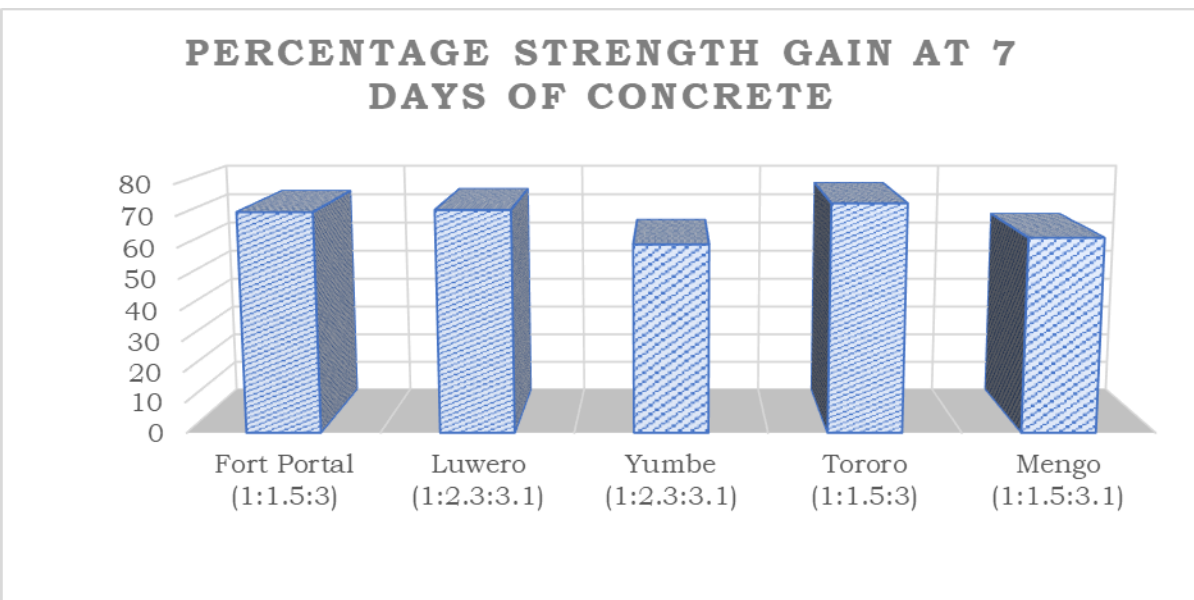


Figure 6-1: C25 Concrete test results



	Fort Portal (1:1.5:3)	Luwero (1:2.3:3.1)	Yumbe (1:2.3:3.1)	Tororo (1:1.5:3)	Mengo (1:1.5:3.1)
Series1	71.2	71.9	61.0	74.0	63.1

Figure 6-2: Percentage strength gain in 7 days of concrete

6.2 Common Practices in Concrete Use

The following observations were made from the survey conducted on sample random sites with regards to concrete practice in Uganda.

i. Cement CEM IV of Tororo brand was used on most (86%) of the sites. It was apparently the preferred option because of its dark grey colour. *It is believed to provide a consistent and uniform colour of concrete; a parameter that the local artisans use to determine the quality of concrete. It is also noted that the Tororo cement brand is generally cheaper than the other brands and the low price possibly influences its preference as well.*

ii. Most of the sites had no defined material quality assurance procedure. *Both coarse and sand aggregates were basically sourced by the clients or their "Yinginiyas" and the purchasers (often unqualified to make an informed choice) made up their own opinion or visual judgement of the quality of materials.*

iii. A number of sites (26%) had dug shallow Wells/boreholes for their site water provision. 74% of the respondents used water off the National Water and Sewerage Corporation mains.

iv. Generally, construction sites with more than 2 storeys use Concrete Mixers for mixing concrete, while a number of sites with 1- to 2- storeys used Hand mixing of concrete. Sites that used Hand mixing did not employ the use of a vibrator for the compaction of concrete but instead used tamping rods.

v. For batching of concrete, a number of the sites used wheelbarrows in tandem with half cut 20 litre jerry cans as batch boxes. 6% of the sites investigated used timber batch boxes. It was also observed that even those that endeavoured to use batch boxes, did not use standardised ones.

vi. Concrete Mix ratios used varied from site to site but a common prevalence of a similar/constant fine aggregates to coarse aggregate ratio for example 1:4:4 batching by volume was noted.

vii. There was no defined procedure or formula on how the water quantity is determined for 86% of the sites. The workers simply used visual assessment to determine the quantity of water needed in any given concrete mix. Others had a known quantity of water that they constantly used on site but did not understand its derivation from the water cement ratio nor cared to vary it for wet aggregates in case it had rained on the uncovered aggregates.

viii. It was noted that Fundi Cement was not typically used on the sites for concrete works as it was considered by the workers to be of an inferior quality.

ix. A number of the sites were not aware about quality control testing of concrete and consequently conducted none. Quality of concrete on site was judged simply by visual assessment with a main focus on the workability and colouration of the concrete. Fading or discoloration of concrete was considered to be 'bad' concrete.

x. It was also observed that a number of the site personnel were not aware about concrete grades and what they meant.

xi. It was also noted that most site personnel had no formal training with regards to Concrete technology. Most (almost 100%) learnt about concrete and its practice on the job, on site.

6.3 Conclusions

Basing on the field investigations done around concrete: -

i. It was observed that there was a general lack of understanding and awareness on the best practices around concrete.
ii. The informal sector of the concrete industry is unaware about the best practices in the handling and use of concrete.
iii. The majority of private clients have neither recognised the need for professional engagement of the Engineers and Architects nor have the capacity to verify the quality of concrete delivered. It is basically a gamble in the industry.
iv. The informal sector of the concrete industry is unaware about the different grades of concrete, quality control measures nor are they familiar with Mix ratios and importance of proper proportioning.
v. The Concrete practice within the majority of the building in-situ construction industry is not ardently regulated nor its compliance enforced.

7 WELD CONNECTIONS PRACTICES

7.1 Introduction to Weld Connections

Joints/connections are critical components of structures, to the extent that a structure may be defined as ‘a constructed assembly of joints separated by members’. Joints are critical in the design of structures.

Structural steel is produced under well controlled manufacturing operations, and failure due to deficiencies directly related to material strength almost never occurs. But brittle fracture failures of steel are occasionally recorded. According to Kaminetzky (2001), the most frequent reasons why steel structures fail are: Insufficient temporary bracing during construction, Errors in design/construction mainly of connections and details, Deficient welding, and Excessive flexibility and non-redundant design.

The four common welding processes used in Uganda are:

- Gas metal arc welding (GMAW/MIG)
- Gas tungsten arc welding (GTAW/TIG)
- Shielded metal arc welding (SMAW) *(the most common process in Uganda)*
- Flux cored arc welding (FCAW)

7.2 Review of the Connection Details in STC

7.2.1 Introduction – Weld Connection Quality

In all the buildings inspected, the steel members were connected using welds. The connections were typically column-beam-column, beam-column, and beam-beam, using either fillet or butt welds. It was observed that not only were the connection details quite unconventional, but the workmanship was extremely poor, several common defects clearly visible, such as **incomplete fusion, incomplete joint penetration, under-cutting, and over-reinforcement** (see illustration in Fig. 7-1). All the welding appears to have been done on site.

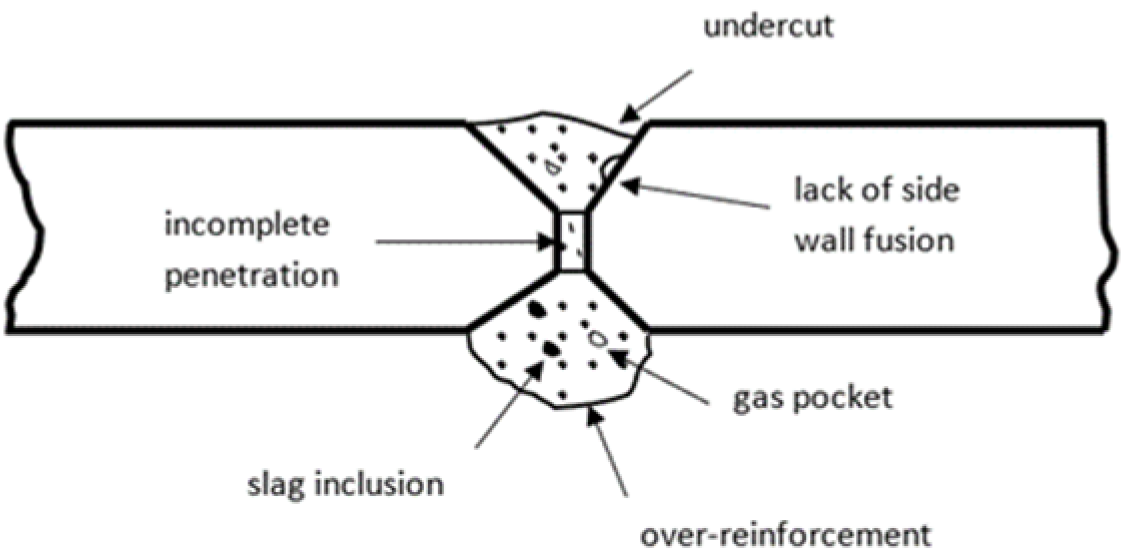


Figure 7-1: Common weld defects (Credit: Jackson A. Mwakali)

The structures visited during the study consisted mainly of welded steel sections used primarily as columns and beams. SMAW fillet welds were encountered the most at the sites with a few butt welds scattered about. Figures 7-2 to 7-4 and the sections that follow show the different welded connections encountered at least five (5) sites, with none of the sites having steel sections thicker than 6mm.

Figure 7-2: School in Nansana with little-to-no weld connection of the steel sections

Weld Porosity, Incomplete weld

Slag inclusion, poor profile

Figure 7-3: Commercial project in Seguku

Figure 7-4: School in Nansana with welds showing lack of proper fusion at joints

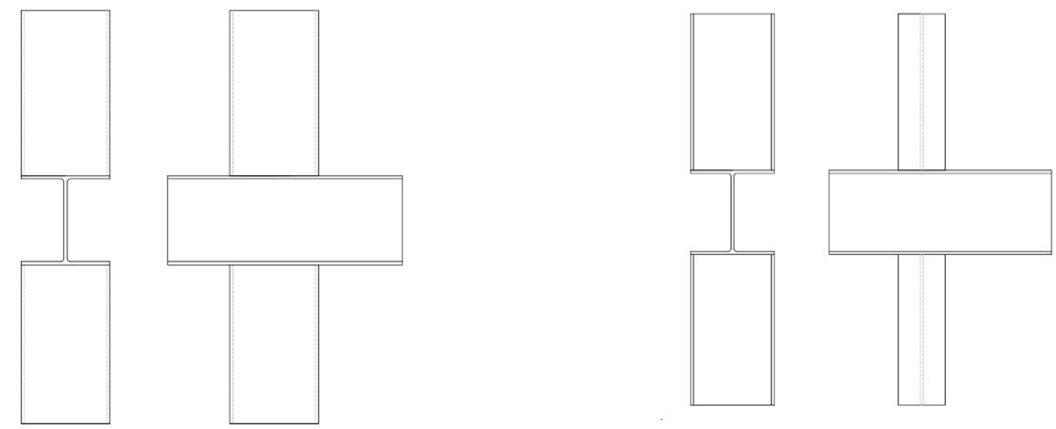
7.2.2 Column-Beam-Column Connections

In the column-beam-column connection, a square hollow section (SHS) or I-section column sits directly on the top flange of an I-beam, while the same I-beam's bottom flange in turn sits on top of another SHS or I-section column below it, respectively (see Fig. 7-5). The three members are connected to each other with fillet welds, with or without the use of gusset plates or angle cleats. Abutting onto the beam were other (secondary) transverse beams on one or both sides (in some cases), although there were also cases without any transverse beams altogether.

(a) As-built connection with SHS columns and I-beams

(b) As-built connection with I-section columns and I-beams

Figure 7-5: Column-beam-column welded connections with transverse secondary beams abutting on to the main beam.



(a) With SHS columns and I-beams

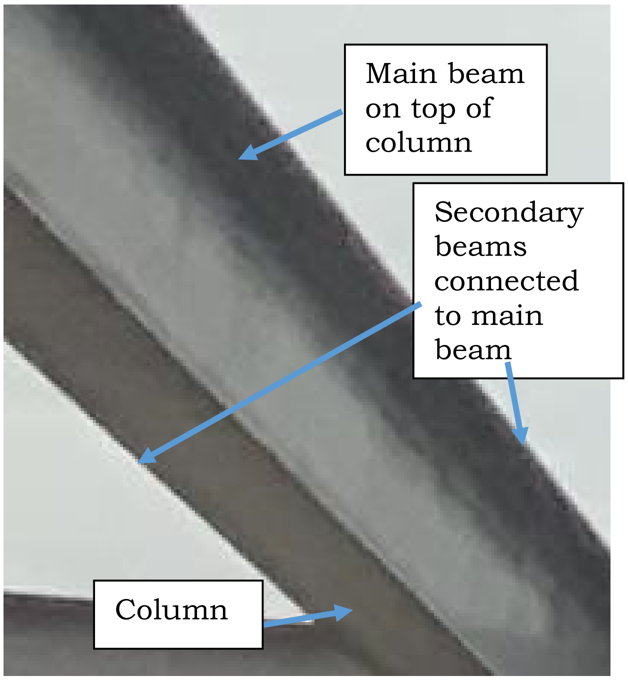
(b) With I-section columns and I-beams

Figure 7-6: Illustrated sketches of the column-beam-column connections

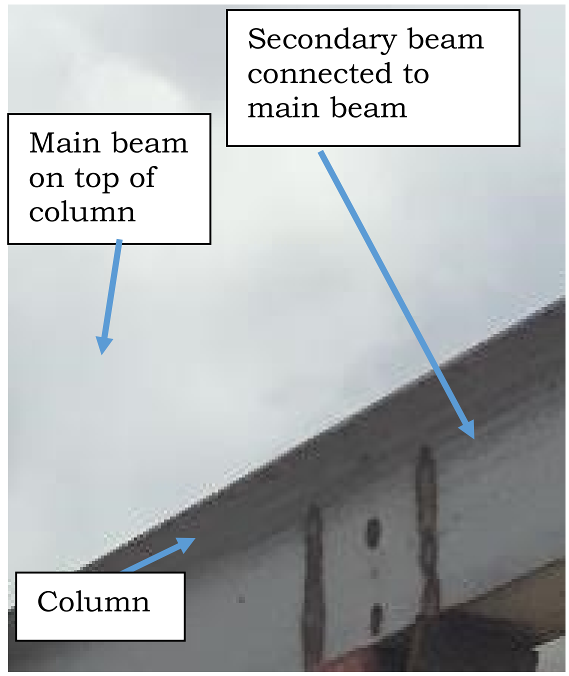
A connection is meant to transfer the load from the upper column to the lower column. However, the sandwiched beam on which the two columns are connected itself needs to be checked for member capacity to bear any flexural-torsional and local buckling actions, in addition to the connecting welds. It can be seen in Fig. 7-5(a) that the web of the I-beam sandwiched between the top and bottom columns is buckled, a sign of failure, even before the building is occupied and loaded to capacity.

7.2.3 Beam-Column Connections

The beam-column connection is where a beam sits on the column but, unlike the column-beam-column connection, there is no column sitting on top of the beam (see Fig. 7-7). This connection is part of the roof support system beyond which there is no floor for human occupancy.



(a) With secondary beams on each side of primary beam



(a) With secondary beam on only one side of primary beam

Figure 7-7: Beam-column connections

7.2.4 Beam-Beam Connections

In the beam-beam connection, the web of a horizontal (secondary) I-beam in the longitudinal direction is attached to the web of another (main) I-section in the transverse direction. Since the two sections are of equal depth, the flanges of the secondary beam are cut off to allow the web to be inserted between the flanges of the main beam and contact its web. Fig. 7-8 shows one such as-built beam-beam connection. In some cases, the flanges were not cut off and instead an angle piece was used to bridge the gap between the two flanges. The concern in this setup is on the effectiveness of load transfer.

(a) T-connection (a) Cross tee connection

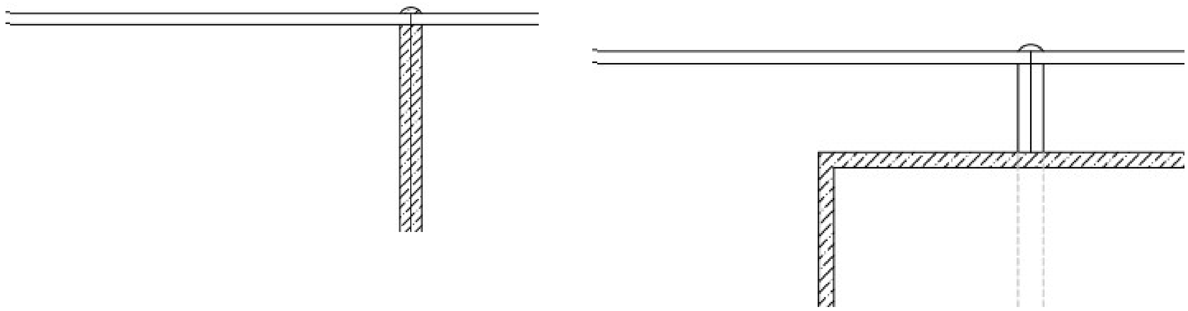
Figure 7-8: Beam-beam connections

7.2.5 Spliced Connections

In the spliced connections, beams were joined together to create longer beams to be able to span between supporting members (beams or columns). The joining was by use of butt welds and/or fillet-welded cover plates, as shown in Fig. 7-9. In some cases, there was only one cover plate. Fig. 7-10 is an illustrated depiction of the joint. None of the columns seen had been spliced, partly because they were only storey high with beams sitting on the respective column sections.

(a) With rectangular cover plate (b) With square cover plate

Figure 7-9: I-beam splice connections



(a) With butt welds (a) With square cover plate

Figure 7-10: Illustrated depiction of I-beam splice connections

7.3 Test Results for Sample Weld Connections in STC

7.3.1 Sampling

Out of the 5 sites visited, weld quality and integrity tests were carried out in Seguku (site 3) and Nansana (site 5).

In Seguku, a steel structural building (possible apartments) constructed with Concrete, Block work, Timber, and Steel materials was inspected, for weld integrity of all welded joints. The steel sections used on the Seguku building were as below.

- Carbon Steel Square Hollow sections of 100x100x4mm, these were used as columns for the upper floor building (level 1),
- Carbon steel I- Beams of 180x92, these were used both as Columns for the ground floor building (level 0) and beams for both Ground Level and 1st Level.
- Carbon steel Plate and Angle Irons – these were used to join both beams and columns to each other.

In Nansana, a steel structural building (school) constructed with Concrete, Block work, Timber, and Steel materials was inspected for weld integrity of all welded joints. The steel sections used on this building were;

- Carbon Steel Square Hollow sections of 100x100x4mm, these were used as columns for the entire building (All levels 0-3),
- Carbon steel I-Beams of 140x73x10.1 kg/m, these were used both as beams from the ground floor up to 3rd floor.
- Carbon steel Angle Irons – these were used to join beams and columns, and some areas beam to beam.

7.3.2 Visual Inspection

Visual inspection is the easiest method of assessing welding quality, and is often a first option before follow-on tests are employed. It is crucial that such an inspection is done by an experienced welding inspector.

7.3.3 Non-Destructive Tests

As per AWS D1.1 -2020, Section 8, item 8.11 (NDT Requirements), only welds subject to NDT shall have been found acceptable by visual inspection in conformance with section 8.9. Only a limited number of joints were inspected by other NDT methods.

7.3.4 Weld Inspection Schematic

The schematics of the inspected welds for which NDT testing was done are given in Figures 7-11 to 7-15.

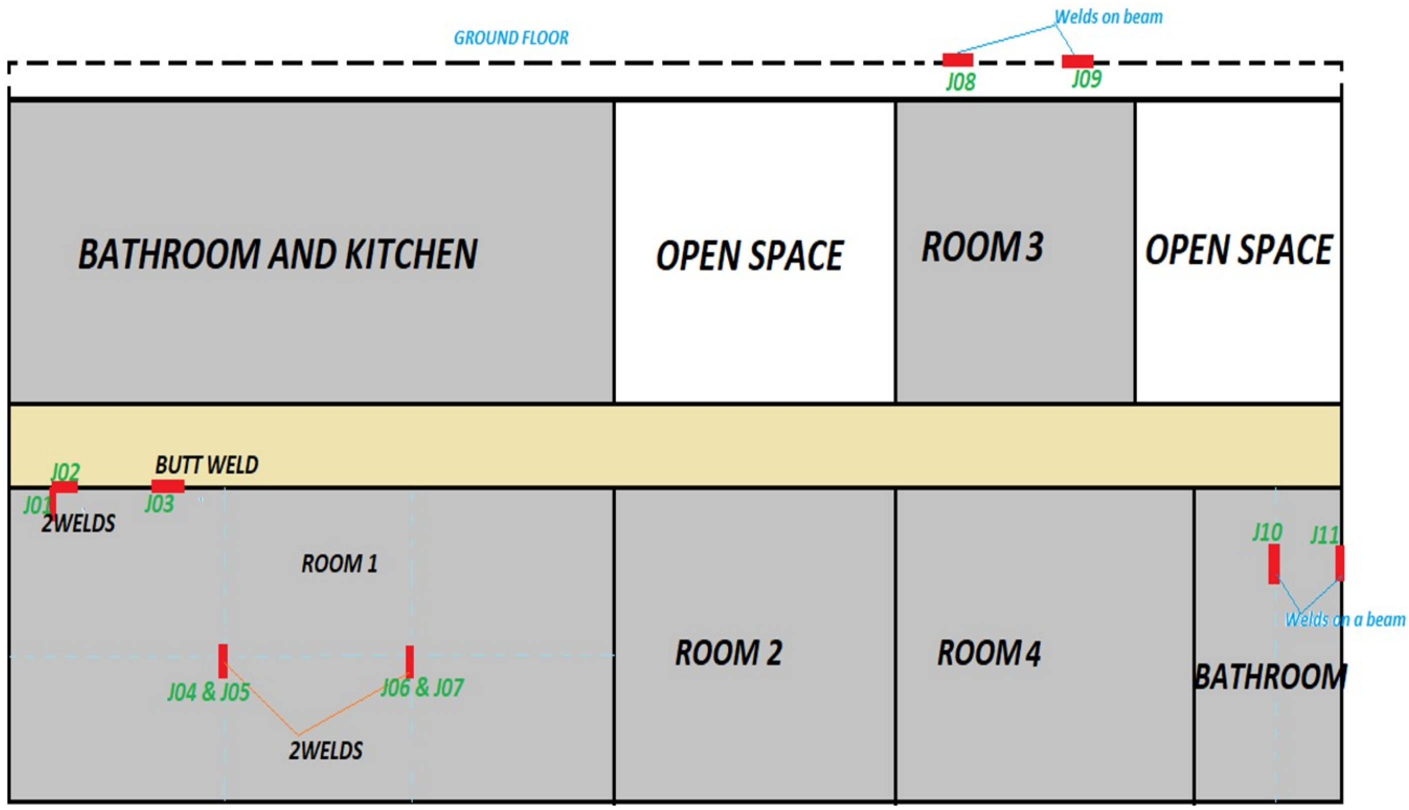


Figure 7-11: Seguku Ground floor weld map

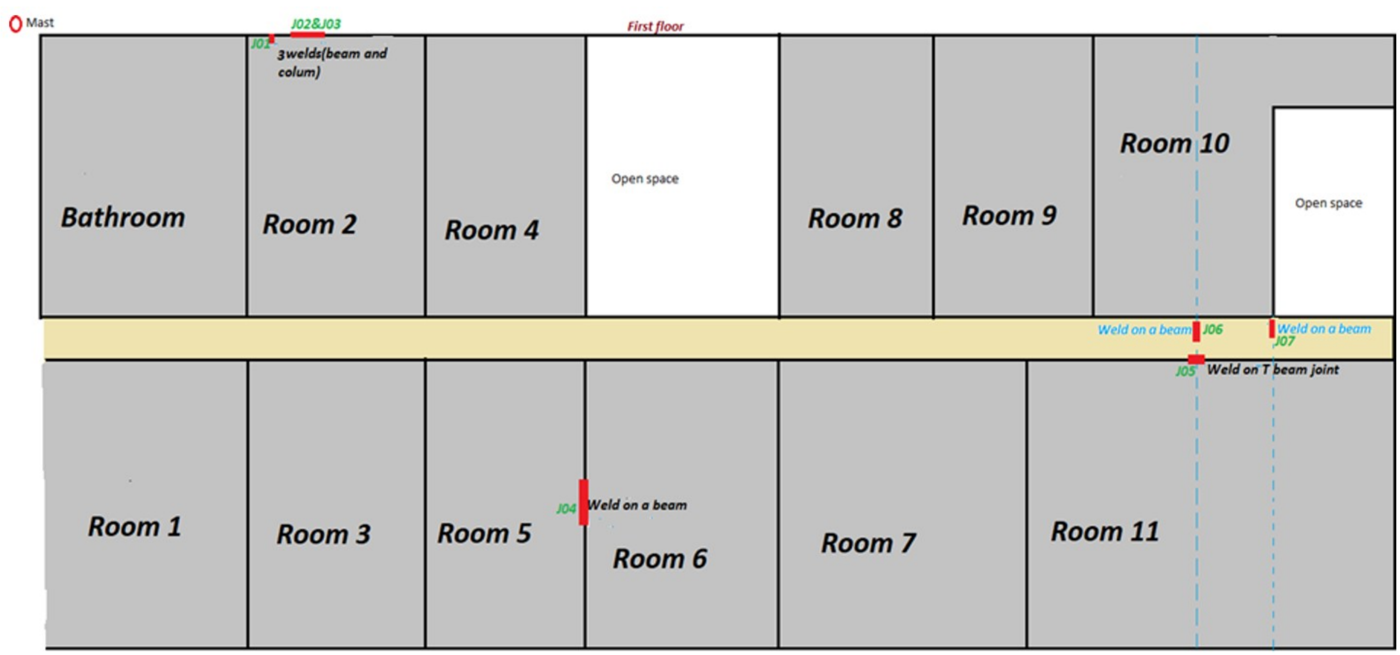


Figure 7-12: Seguku level 1 weld map

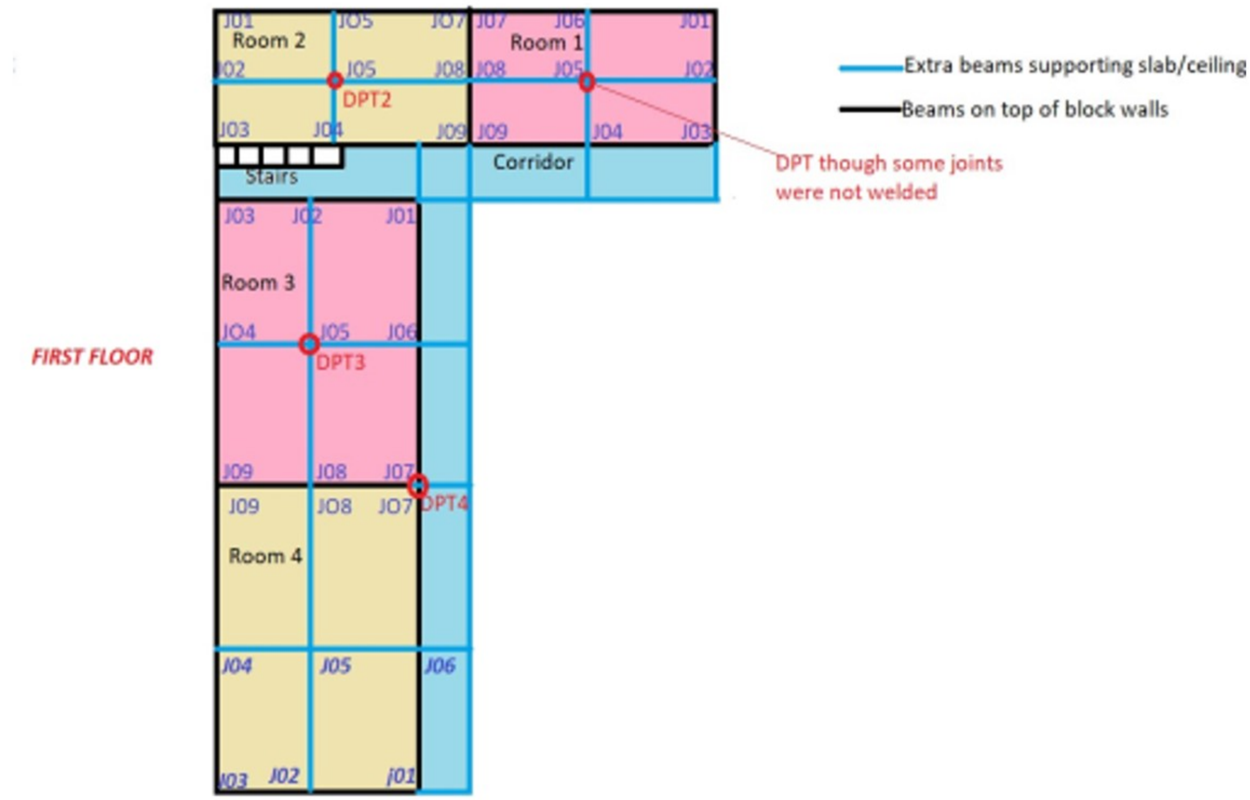


Figure 7-13: Nansana 1st Floor weld map

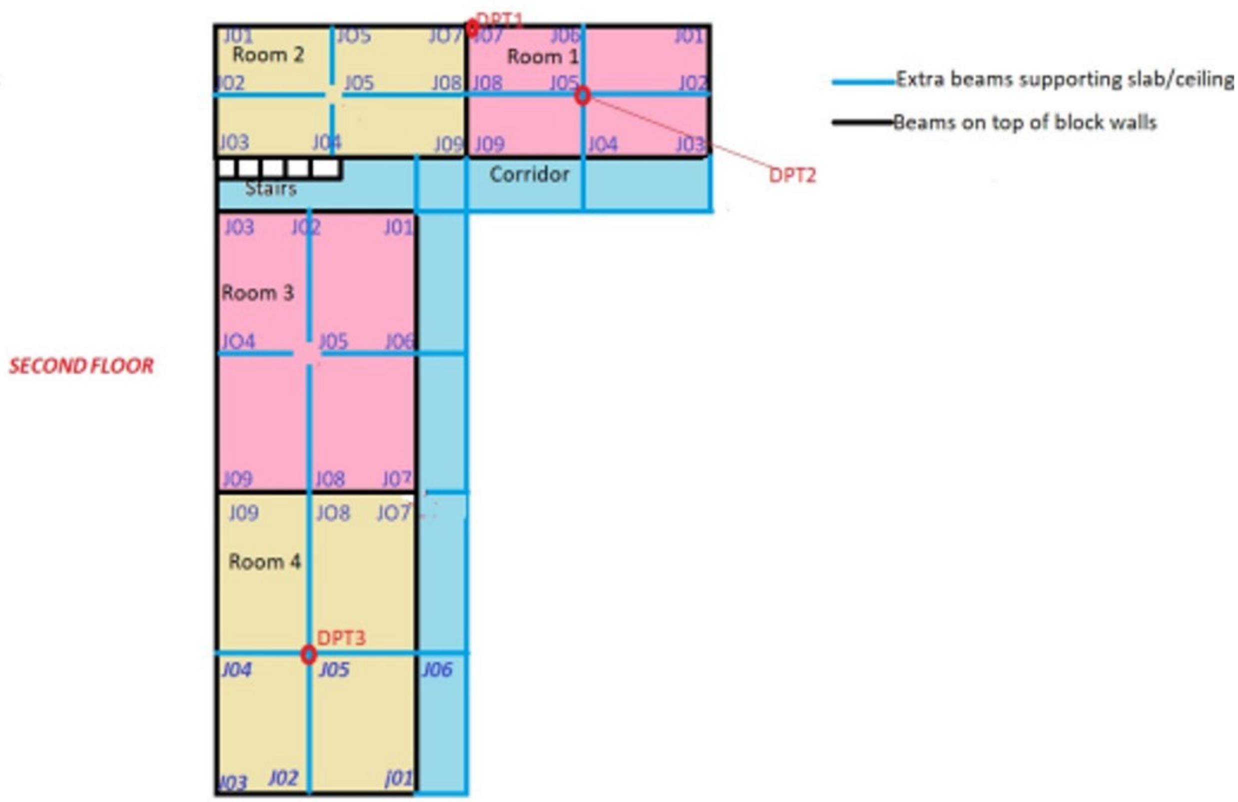


Figure 7-14: Nansana 2nd Floor weld map

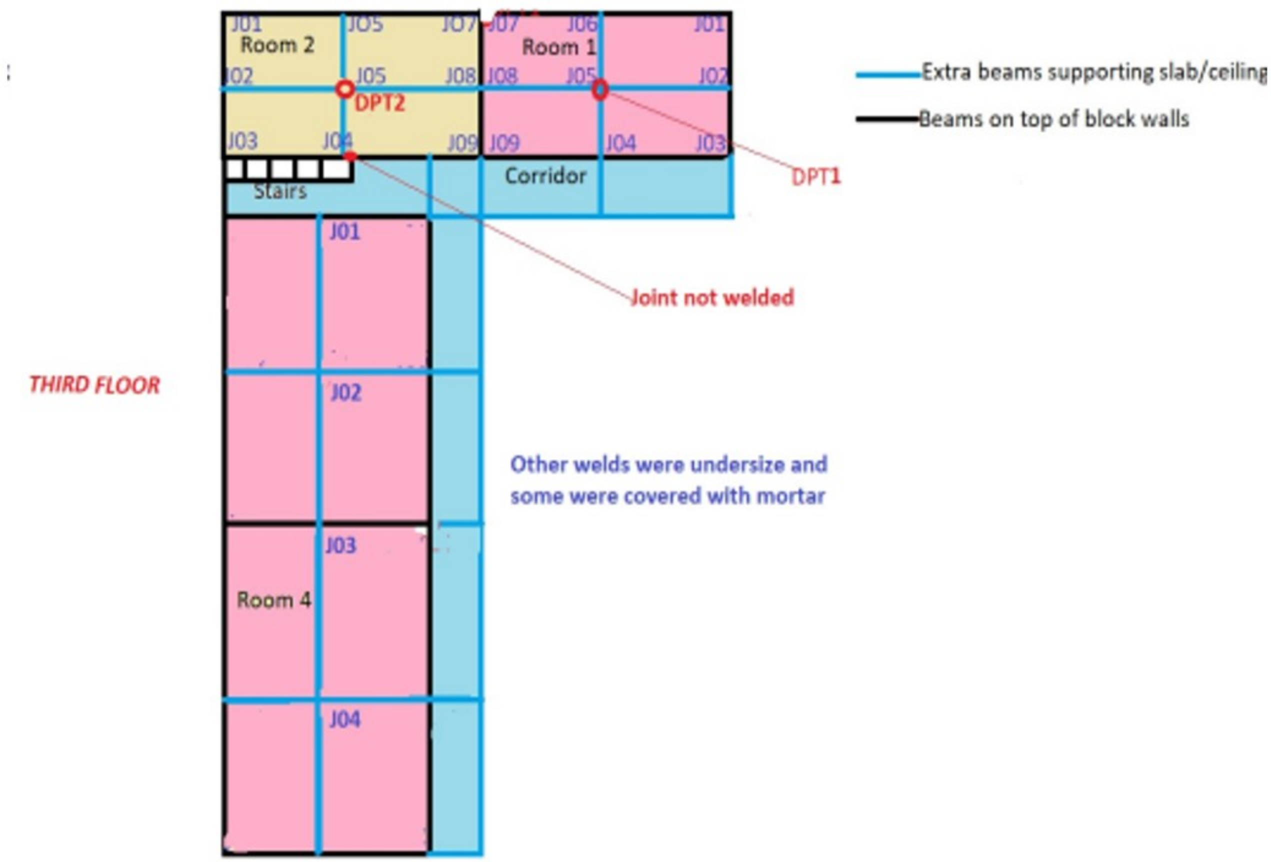


Figure 7-15: Nansana 3rd Floor weld map

7.3.5 Visual Weld Inspection Findings

The conclusions from the reports for both sites is that all welds **must be redone**, with a proper welding procedure specification (WPS) made and provided to the welders and fitters. This shall describe the type of welding rods to use, how many passes per weld, under what conditions the welding rods be kept, what Interpass temperature should be maintained to minimise interference with the mechanical properties of the sections being welded, which fit up gap to be kept, so that there is enough weld penetration.

Most of the fillet joints were unacceptable since they were noted to be undersized, incomplete welds, under fill welds, undersized welds, with pin holes, no capping, and lack of fusion at some points. Some of these defects are shown in Figures 7-16 and 7-17.

Figure 7-16: Poorly welded joints with inconsistency in weld profile.

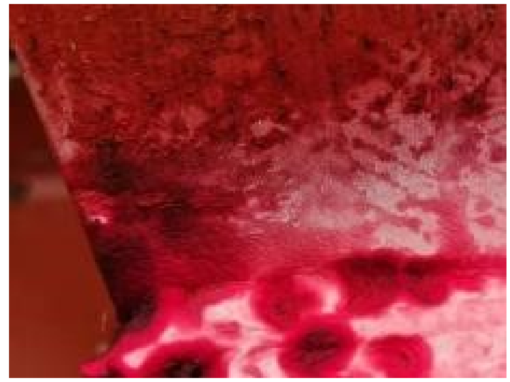
Figure 7-17: No fusion between the base metal and the angle iron

7.3.6 Liquid Penetrant Test Inspection

Many defects observed through visual inspection such as incomplete welds, lack of penetration, poor weld profiles, no capping, slag inclusion, lack of fusion, under fill sized welds were confirmed.

Non-uniform weld size and poor penetration

Intermittent weld length



Over-reinforcement, Roughness and Overlap

Figure 7-18: Inspection of welds under Liquid penetrant

Figure 7-19: One of the few acceptable butt welds

7.3.7 Ultrasonic Testing

Manual UT Shear wave scanning was not carried out at the two sites; Seguku and Nansana due to code restrictions. Most of the steel thickness and joints were a maximum of 4mm. As per ASME sec V and BS standard, Shear wave scanning Material Thickness required is above 8mm.

7.3.8 Weld Quality Overview

Of the 132 welds tested, 85% were rejected as unacceptable and should be redone.

7.4 Skill Level of Welders in Uganda

The industry requirements for welders as viewed from the visits to the construction sites is Level 4 certification, which ensures that the welder is proficient at;

- Welding when the workpiece is on the floor or workbench
- Welding on the upper side of the horizontal and vertical surfaces
- Overhead welding, which is required when the workpiece is fixed and cannot be moved.

It was also observed that the welders for the Seguku site could only try the 1G/F[8] welding position. All other positions; 2G[9], 3G[10] and 4G[11] could not be done, since they lack training and prequalification, hence a need for welder training and certification before getting them back to site.

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[8] Flat position, denoted as 1G/1F
[9] Horizontal position, denoted as 2G/2F
[10] Vertical position, denoted as 3G/3F
[11] Overhead position, denoted as 4G/4F

8 GUIDELINES FOR THE DESIGN AND CONSTRUCTION OF STRUCTURES WITH STEEL, TIMBER AND CONCRETE

8.1 Preamble

Designers and builders who wish to use the three materials of steel, timber and concrete as structural materials in combination, either as independent members or behaving as composite assemblies need to follow basic guidelines to ensure safe and durable structures. The flowchart in Fig. 8-1 illustrates the process that the designer and builder would normally go through to arrive at a satisfactory construction involving the three materials.

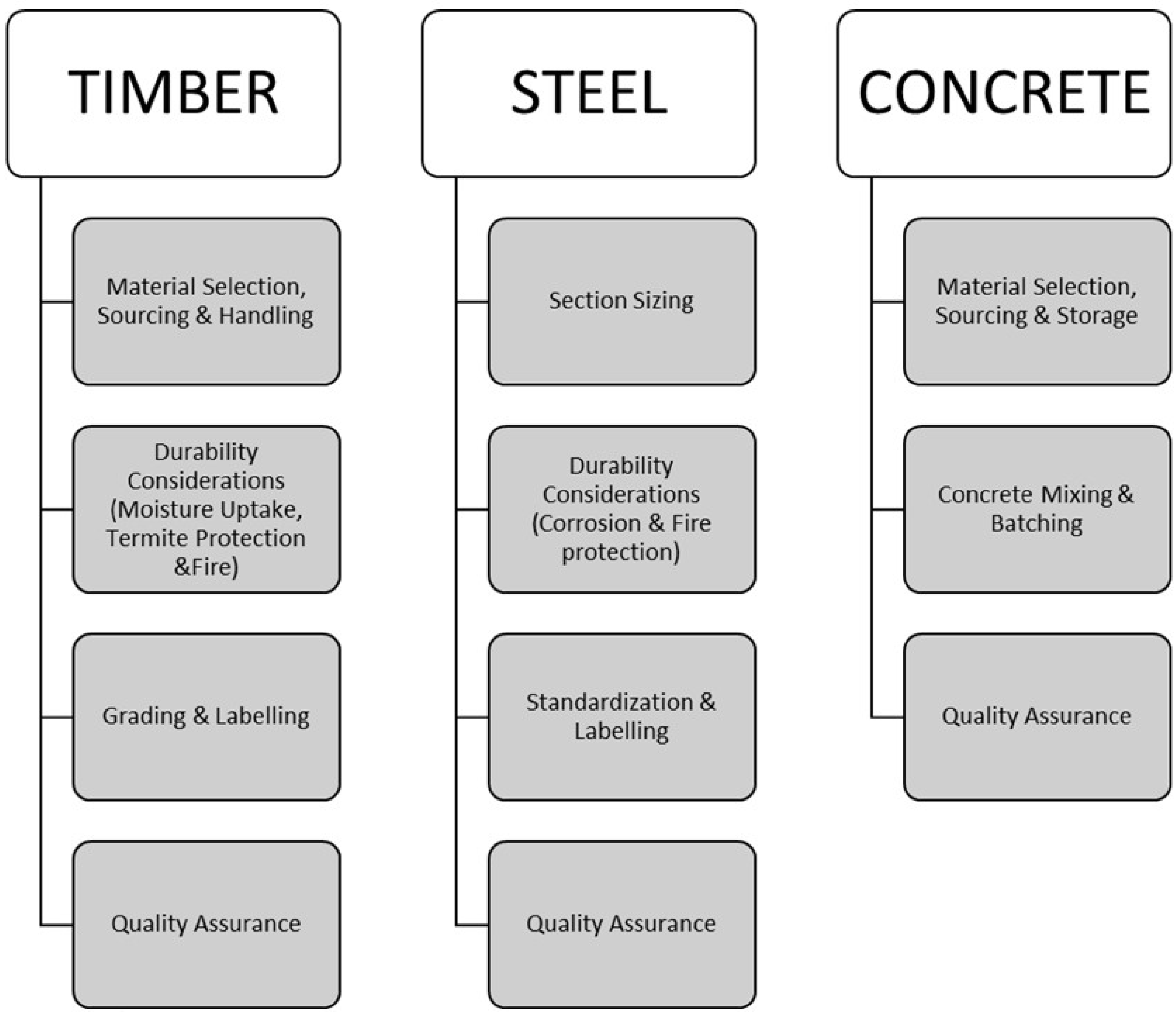

Figure 8-1: Process flow in the use of the three materials in construction

8.2 Structural Timber

8.2.1 Source of Timber

Most of the construction timber in Uganda comes from timber hotspots such as Ndeeba and Bwaise, and smaller ones in urban centres around the country. Timber from these hotspots is normally green (not seasoned) and not graded. It is purchased at varying moisture contents with most pieces still above the Fibre Saturation Point i.e. >30% Moisture Content, without the knowledge of most clients.

Guidelines

i) Developers should procure structural timber from reputable suppliers who are certified by UNBS or any other authorised regulator/ certification organisation.

ii) Developers should procure structural timber that is graded and with a grading certificate, showing the strength properties and nominal sizes

8.2.2 Selection and Handling of Structural Timber

Selection of structural timber largely depends on the purpose as this will determine the size, grade and sometimes the species required. For wall cladding and ceiling boards for instance, the visual aspects of timber may be more important than in roof trusses where stress grades and absence of defects may be preferred. Timber should be seasoned to avoid stain, fungal attacks, post harvesting defects, and reduction in strength. Fungi require Moisture Content (MC) above 20% in order to attack wood.

Guidelines

1. *Timber should be seasoned to a maximum of 18% Moisture Content in accordance with US 833-2:2020.*
2. *Timber should be machine stress graded and marked visibly with the grade.*
3. *Timber pieces of the same species, grade, and nominal thickness should be packed together in a bundle, and then the bundles properly stacked in*

store.

8.2.3 Keeping Timber Free From Moisture Uptake

Wood can pick up water by absorption when in contact with water or a wet material but it can also pick up moisture by adsorption from humid air and lose moisture by desorption to the surrounding air if the air is relatively drier. It is expected that seasoned timber members would pick water from wet concrete a few days after casting but later on it is expected to attain the Equilibrium Moisture Content (EMC) with the surrounding environment.

Guideline

A plastic material or any other suitable waterproofing material should be placed between the concrete and timber members to avoid contact of water from wet concrete with seasoned timber as well as from any water leakages before or after the building is occupied.

8.2.4 Durability Treatment against Termite Attack

Termites are one of the leading agents of wood degradation in service in buildings. They degrade all wood components including cellulose, hemicellulose and lignin.

Guidelines

i) Ensure that the site is treated with approved anti-termite chemicals.

ii) Ensure regular inspection and maintenance of the structures, for timely regular anti-termite treatments.

iii) Apply anti-termite treatment to the timber. For structures whose design life is at least 50 years, the timber should undergo pressure treatment with preservatives such as Chromated Copper Arsenate (CCA).

8.2.5 Grading and Marking

Grading and Marking is essential for traceability of timber and to allow designers access to available grades of timber and properties for initial design. The Uganda Standards US 833-2:2020 and US 2248: 2021 require that timber conforming to these standards be marked with information including the manufacturer's name, the appropriate stress-grade identification; and the date or the batch number of the stress grading.

Guidelines

i) Timber suppliers should comply with the US 833-2 and US 2248 requirements and mark their structural timber before dispatch to the market with:

- *Manufacturer's name*
- *Appropriate stress grade*
- *Date or batch number of the stress grading*
- *Certification body logo or mark*

ii) Developers should only procure timber that is appropriately marked as advised in (i)

8.2.6 Quality Assurance

Uganda has local capacity to test for timber properties and machine stress grading. Laboratories at UNBS, MoWT Central Materials Laboratory and Universities are equipped for the typical tests.

Guideline

Developers and their professionals should sample timber and test in order to verify declared properties of timber as part of the typical quality assurance tests.

8.3 Use of Structural Steel

8.3.1 Section Sizing

Selection of structural steel member section sizes and shapes is dependent on:-

- Actions (loads) that the member is expected to be subjected to during its design life.
- The grade of the steel material.

- The availability of sections on the market.
- Economic and commercial considerations.

The selection of a size of structural steel is solely an output of a design, done by a professional engineer.

Guidelines

i) Developers should engage professional engineers to design with structural steel as per Reg.5 and Reg.6 of the Building Control Regulations, 2020.

ii) Building Committees should only consider applications for approval of developments with structural steel which are accompanied by design calculations and certificates of good structural practice, signed off and stamped by registered (professional) engineers.

iii) Structural steel components shall be designed to facilitate fabrication, erection and future maintenance of the works.

8.3.2 Durability of Structural Steel- Protection against Corrosion

One of the shortcomings of structural steel is proneness to rusting when in contact with moisture. It is, therefore, important to protect structural steel members against corrosion.

Guideline

Designers should specify protective measures against moisture and air, which are the key agents of corrosion. These could include coatings/barrier methods, encasement of critical elements, galvanisation, electro-plating or use of stainless steel, among others.

8.3.3 Durability of Structural Steel-Protection against Fire

Fire affects the strength of steel. At temperatures as low as 300 degrees Celsius, steel could lose as much as 50% of its strength. Whereas there may be some residual strength after a fire, usually the connections give way faster than material failure, which is an irreversible failure.

Guideline

Designers should specify protective measures against fire. These could include embedment into or coating with materials that shield the steel against fire exposure, intumescent paints, etc.

8.3.4 Standardisation and Labelling/Marking

It is important that structural steel is labelled/marked at source, and be accompanied by a mill test certificate declaring the properties of the steel. Structural steel sections, especially hollow sections in Uganda are seldom marked, making it unlikely that designers and developers will know its properties unless they run independent tests. The situation is made worse by the fact that catalogues of most manufacturers are rarely comprehensive enough.

Guideline

Steel manufacturers should label their steel products before marketing with information on: -

- *Grade of steel*
- *Manufacturer*
- *Nominal dimensions*
- *Certification mark*

8.3.5 Quality Assurance

Uganda has local capacity to test for structural steel properties. Laboratories at UNBS, MoWT Central Materials Laboratory and Universities are equipped for the typical tests.

Guideline

Developers and their professionals should sample and test structural steel in order to verify the declared properties of steel as part of the routine quality assurance tests.

8.4 Concrete in Construction

8.4.1 Sourcing, Selection and Storage of Materials

The quality of coarse aggregates, fine aggregates, water and cement binder affects the quality of concrete.

Guidelines

i) Cement for use in reinforced concrete should have a minimum strength Class 32.5 MPa.

ii) Aggregates should be stored under cover to avoid water uptake.

iii) Mixing water should preferably be of potable quality, free of algae and clear.

8.4.2 Concrete Mixing – Design and Batching

The majority of construction sites use batching by volume, batch per bag of cement, do not measure the amount of water, use CEM IV 32.5 cement, and hardly test the concrete. The Water-Cement ratio significantly affects the strength of concrete.

Guidelines

i) Industry should use a rigid batching box of size 0.3mx0.3mx0.3m, which shall be taken as equivalent to 1 bag of cement[12].The cement bag need not be poured into batching boxes, to avoid wastage.

ii) A maximum water cement ratio of 0.5 should be used with dry aggregates. For 1 bag of cement, a maximum of 25 litres of water should be used for mixing concrete.

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[12] *Considering a bulking factor of cement of 20-25%*

8.4.3 Quality Control

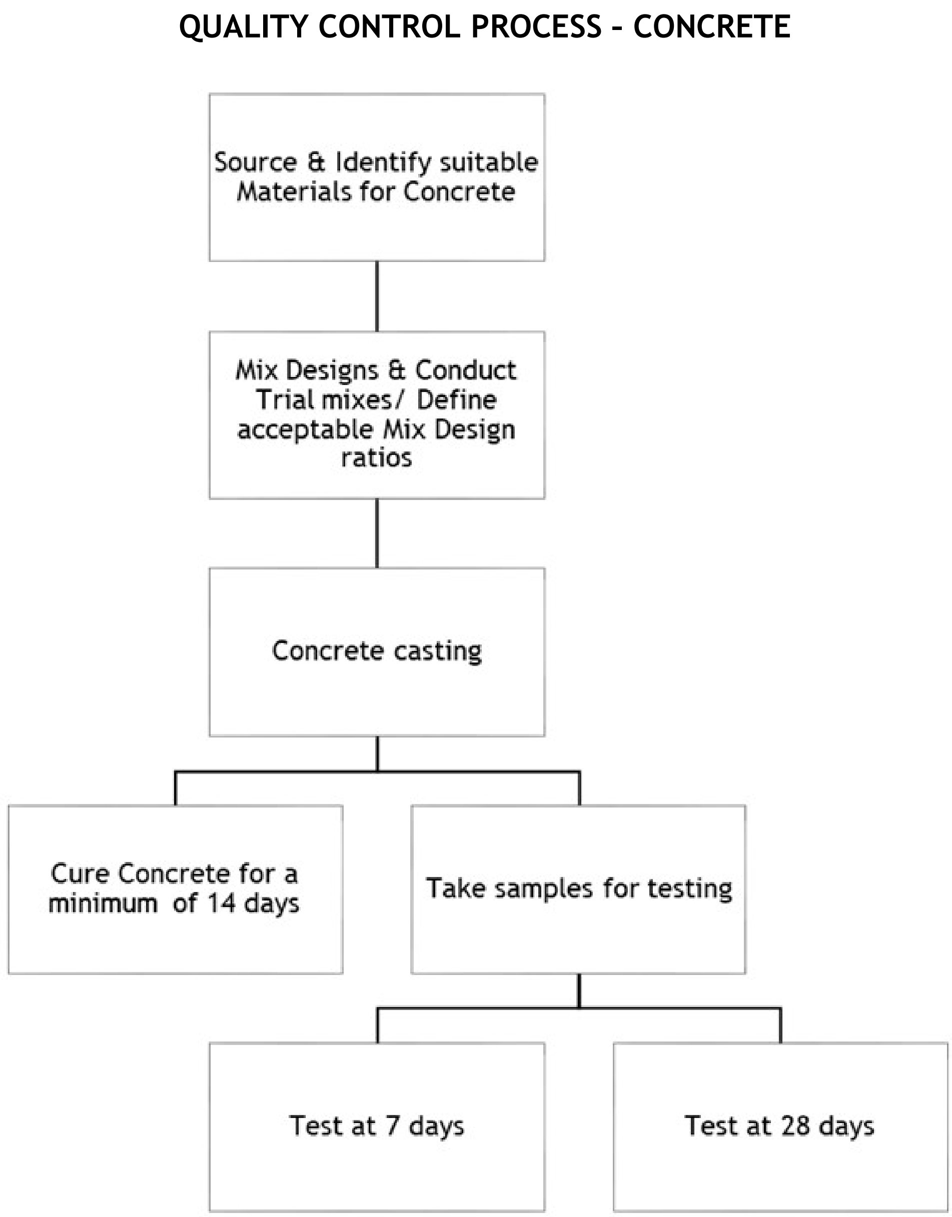


Figure 8-2: Quality control process for concrete

Guidelines

i) Supervision for concrete works should be superintended over by a professional, preferably the designer in compliance with Reg.6 of the Building control regulations, 2020.

ii) At least 6 Concrete cube samples should be picked for each mix, at least 3 of which should be tested at 7 days. At least another 3 should be tested at 28 *days.*

iii) Curing of concrete should be done for at least 14 days.

iv) Compaction of concrete should be by mechanical means such as poker vibrators.

v) For all reinforcing steel, the manufacturer's certificate shall be required as proof of the characteristic strength.

8.5 Connections

8.5.1 Welded Connections

Weld production positions are classified as Flat (F), Horizontal (H), Vertical-Up (V-U), Vertical Down (V-D) or Overhead. A higher skill level is needed for Vertical, horizontal and overhead positions. In actual shop fabrication, welding can be done satisfactorily.

The composition of the base materials affects the strength and quality of a weld. The type of base material influences the choice of filler material/electrode/welding rod.

The choice of filler material/electrode/welding rod is dependent on the thickness of the base materials, and anticipated load. The majority of the welds in Uganda are using 6013 welding rods, which points to lack of design consideration.

Most of the weld connections inspected by the technical committee were single passes and were inadequate. The number of passes and sequence of welding are required before fabrication.

None of the sites inspected by the technical committee were done by certified welders. The lack of readily available skill needed for the connections results in poor quality welds. There are 6 levels of certification of welders (1-3 for artisans, 4 typically for engineers, 5-6 for supervisors).

Guidelines

i) Weld connections should be designed and specified by a qualified engineer.

ii) Developers should adopt shop fabrication as much as possible to avoid quality issues in construction.

ii) Developers should obtain a copy of the mill test certificate that shows the chemical composition and grade of the base materials.

iv) Weld connections should be supervised by competent inspectors during implementation to ensure quality works.

v) The choice of filler material/electrode/welding rod should be a design output by a qualified engineer.

vi) Welding should be done by minimum Welding Level 4 certified persons.

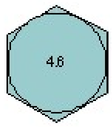
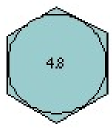
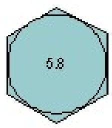
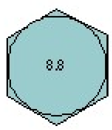
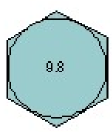
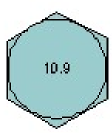
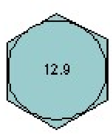
vii) Engineers should furnish welders and fitters with a welding procedure specification (WPS), which shall describe the type of welding rods to use, how many passes, under what conditions the welding rods be kept, what interpass temperature should be maintained to minimise interference with mechanical properties of the sections being welded, which fit up gap to be kept, so that there is enough weld penetration.

8.5.2 Bolted Connections

Bolted connections are a type of structural joint used to join two or more structural components in a steel structure using bolts. The Technical committee did not find any bolted connections in the STC sites that were visited.

Bolt grades are defined by their specific material, as well as the strength of that material and are designated as shown in Table 8-1.

Table 8-1: Metric steel bolts - grades and property classes (ISO 898-1:2013)

Property Class	Head Marking	Size Range *(mm)*	Minimum Proof Strength[13] *(MPa)*	Minimum Tensile Strength[14] *(MPa)*
4.6		M5 - M36	225	400
4.8		M1.6 - M16	310	420
5.8		M5 - M24	380	520
8.8		M1.6 - M36	600	830
9.8		M1.6 - M16	650	900
10.9		M5 - M36	830	1040
12.9		M1.6 - M36	970	1220

Guidelines

i) Bolted connections are the preferred on-site type of connections because of their ease of installation, ease of inspection and speed of installation.

[13] Proof load is the limit of the elastic range of a bolt. If a bolt is tensioned beyond its specified proof load, it can't be used as it experiences plastic deformation.
[14] Tensile strength is the amount of stress or load that the fastener can withstand by a material before it stretches and breaks.

ii) Preferably, grade 8.8 bolts should be used for structural applications.

iii) Bolts should be tested for proof load and ultimate tensile strength against specifications.

8.5.3 Timber to Timber Connections

The connection is mainly beam (primary joist) to secondary joist connection. The same connection is employed for ceilings in Uganda. The beam to joist connection is usually nailed, with the nails inclined at an angle to connect the two members. This is prone to the following failure mechanisms:

- Over time, as the timber dries, the nails become loose.
- If moisture reaches the ceiling the nails may rust.

In either case, the ceilings sag leading to cracked ceilings, leading to the need to refix. It is to be noted that ceilings in buildings only carry their own weight and rarely carry imposed loads except during inspections and repairs, which may occur once a year. On the other hand, STC floors are meant to carry their self-weight and imposed loads almost every second of their service life.

Guidelines

i) The timber-to-timber joints must be constructed in such a way that they do not weaken even when the timber dries and the connectors must resist rusting. This can be achieved by using galvanised plated connections (See Fig. 90) and coated connectors respectively.

ii) The nail connection - number, length, diameter of nails are design outputs which should be left to a professional engineer to specify.

iii) The secondary and primary beams should be properly aligned to ensure proper load transfer.

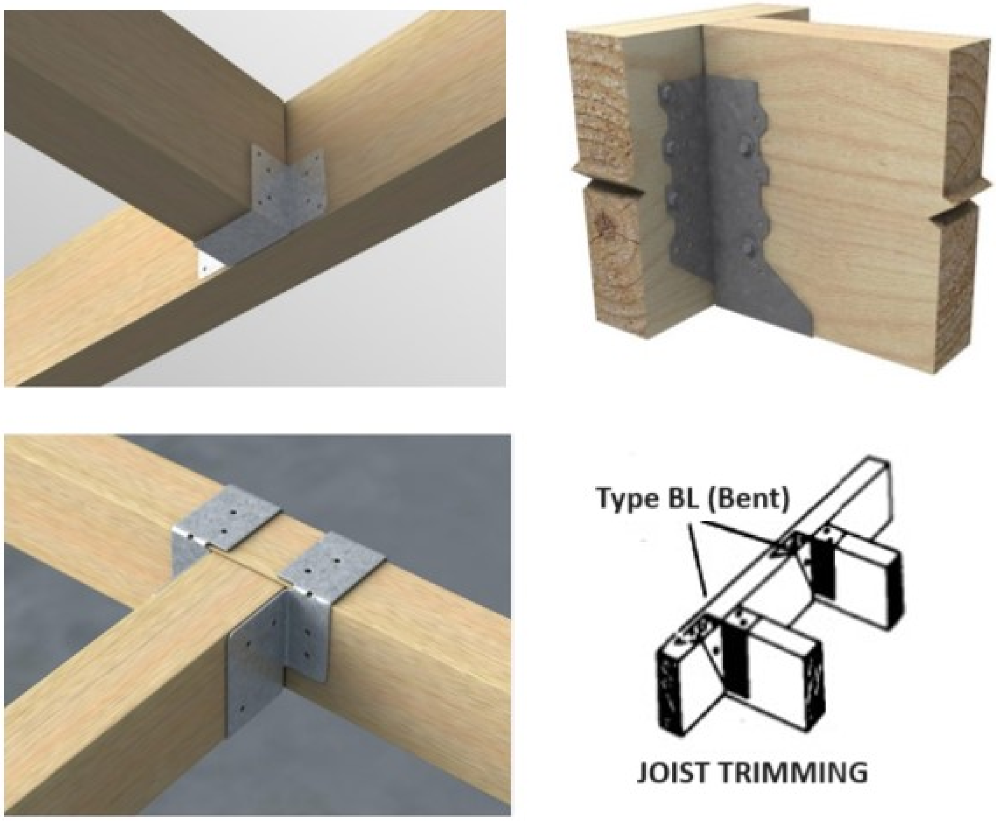


Figure 8-3: Best Practice Timber to Timber Joist Connection

8.5.4 Steel to Concrete Connections

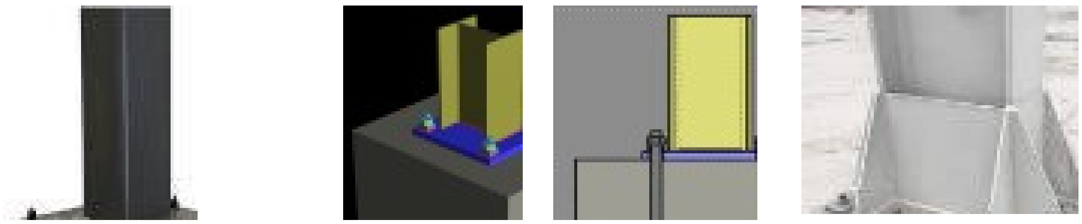
The Technical Committee observed that:

a. Steel stanchions were in contact with the ground without concrete pedestals to prevent contact with the ground. Unprotected steel, in contact with moisture, would corrode, to disastrous effect.
b. There were no shear connectors on the steel beams (protruding into the concrete overlay), making the possibility of composite action unlikely.

Guidelines

iv) *Designed Shear connectors (number, spacing, type, length) should be included on top of the steel beam if any composite action is to be assumed.*
v) *A concrete pedestal should be provided, off the ground to avoid contact of steel with moisture in the backfill, ease of constructability, and reduction in concentrated loads at the foundation. The steel column can then be bolted*

onto the stub column using bolts through base plates and holding down bolts (See Fig. 8-4).



(a) Photos of unstiffened base plate | (b) CAD renders | (c) Stiffened base plate

Figure 8-4: Photos and CAD renderings steel column to concrete base connections

vi) For encased steel columns, the encasing concrete shall extend the full length of members and connections and be reinforced with steel fabric.

8.5.5 Timber to Steel Connections

There are two methods observed in the field that were used to connect the timber to the steel beams.

- Method 1: The timber joists are just laid (placed) on the top flange of the IPE steel beam without any anchors to the steel beam.
- Method 2: The timber joist is just fitted inside the I section (between the top and bottom flanges) without any anchoring.

Both methods 1 and 2 pose a danger and can lead to collapse of the floor slab in case of any movement of the steel beams in the lateral direction. Because the timber members are not anchored to the steel beams, they will not move together leading to collapse. Lateral movement of the structure could be caused by earthquakes, blasts, or strong wing loading.

Guidelines

i. *Timber joist should be bolted to a designed angle cleat which is welded offsite to the web of IPE steel beams.*

ii. *For the case where the timber is placed on top of the IPE steel beam, the timber could be bolted to a designed angle cleat, which in turn is welded offsite onto the steel beam.*

8.5.6 Concrete to Timber Connections

Composite action can only be assumed if there are shear connectors in the STC set ups. Any composite action that could have been generated due to friction between concrete and timber members, is discounted in cases where the polythene is placed on top of timber to prevent ingress of moisture from concrete onto the timber. In cases where there is no polythene on top of the timber, it is arguable that adhesion between the timber and concrete exists, but the extent of this adhesion and composite action cannot be estimated without testing and research.

Guidelines

i. *STC slabs shall incorporate designed shear connectors (size, length, spacing) in order for the Concrete on Timber to behave as composite.*
ii. *Even without assumed composite action, some form of fixity will be needed to prevent relative movement between the two. Studs can be provided at the minimum at the ends to avoid free movement.*
iii. *Concrete should only be allowed direct contact with the timber, if the timber is treated. Timber must be protected from moisture ingress from the wet concrete by Damp Proofing before casting the concrete slab.*

8.5.7 Steel to Steel Connections

The majority of connections in the existing STC structures were welded. The welding was so poorly done that in 85% of the welds tested, the recommendation was that they be redone. None of the weld connections had drawings and specifications guiding the implementation.

Guidelines

i. *Weld connections shall be designed by qualified professionals, with detailed Weld Procedure Specifications (WPS) and preferably done off site.*
ii. *Welders for structural applications should have minimum Skill Level 4.*
iii. *On-site fabrications should as much as possible be restricted to bolted connections, preferably with a single size specified for the project.*
iv. *All welded connections should be inspected for integrity by a qualified Welding Inspector who shall issue a report of compliance that shall form part of the requirements for issuance of an occupation permit.*

8.6 Reinforcement of the Concrete Slab

The STC overlay slab is reinforced with BRC mesh which is placed atop the polythene layer on top of the timber. In the majority of the cases, no concrete spacers are provided. It is not clear whether the mesh was only to control cracking or for flexural strength. In some of the sites, there were visible cracks at the top of the slab along the beam positions.

Guidelines

i. *The reinforcement to control cracking should be designed and appropriately placed near the top the slab in accordance with the standards.*
ii. *The reinforcement for flexural strength should be designed and appropriately placed near the bottom for span regions and near the top for the hogging zones, in accordance with the standards.*
iii. *Concrete spacers shall be incorporated, of the specified thickness for appropriate concrete cover for the mesh.*

8.7 Mechanical and Plumbing Installations

The mechanical and plumbing installations cannot run continuously within the slabs because of the timber framework. Consequently, in the majority of the cases, the wet areas are restricted to the peripheries of the building, which may not be achievable or practical in all cases. The typical slabs being only 100mm thick, are not adequate for plumbing installations to be embedded. The ceiling soffit is not thick enough for anchoring of service lines like HVAC, waste pipes, water pipes, etc. The nature of the slab creates constraints the installation of services across the slab.

Guidelines

i. *Structural designs should consider the impact of any loading of service lines clipped/ anchored on the ceiling soffit for integrity*

ii. *Wet areas (toilets, bathrooms, kitchens, swimming pools) and services routes should be solid slab (reinforced concrete).*

iii. *Mechanical Ventilation and Air Conditioning Installation*
   - *For Air Based Systems where the conditioned air is produced centrally and distributed via ductwork, the ductwork should be able run underneath the soffit with minimal interruption and adequately anchored.*
   - *For Water Based Systems where hot or chilled water is used to convey heat to or from a conditioned space or process through piping, the piping should not be chased in the slab or wall at any point but rather clipped at the soffit or within a service duct.*

iv. *Plumbing Installations*
   - *The water supply pipework may run within the slab; Bends must be minimised (to minimise pressure losses).*
   - *The drainage piping shall be chased underneath the slab ensuring the proper drainage slopes are achieved.*
   - *The plumbing should be accessible for maintenance.*

v. *Fire Fighting Installations: The horizontal network of pipes in the fire fighting system shall be laid underneath the soffit and adequately supported.*

8.8 Electrical Installations

If the cables are directly clipped onto the timber, the risk of fire is high as well as damage to the cables by vermin. The spread of fire in STC frameworks is high, given the timber framework in the slab.

Guidelines

i. *A licensed professional must be engaged in the design and supervision of electrical building services as per regulation 5 (1)(b) and 6 (1) of the Building Control Regulations, 2020.*

ii. *Electricians with a minimum of Class C installation permits from ERA (in compliance with regulation 14 of the Electricity Installations Permits Regulations, 2018) should carry out new installations, repairs and maintenance works of the buildings.*

iii. *All installations shall be in accordance with National Building (Standards for Electrical Installations in Buildings) Code, 2019. Specifically:*

   <u>Protective Devices:</u>

   - *Every installation and circuit shall be protected against overcurrent by devices*
   - *All installations shall be protected against electric residual current with appropriate devices.*

   <u>Cable Conduiting:</u>

   - *All electrical cabling should be embedded in PVC conduits.*

   <u>Cable Wiring:</u>

   - *All cable wiring of the main and final sub-circuits shall be carried out in PVC insulated and PVC sheathed cables*
   - *Connections between flexible cables and conduit wiring shall be made of an approved connector secured in a suitable case or box.*

iv. *All steel elements should be earthed.*

8.9 Fire Resistance

The range of overlay concrete thickness in STC constructions was 50-100mm. The thinner the slab, the less time it will take for the slab's integrity to be compromised in case it is exposed to a fire.

Guideline

The Minimum fire rating should be at least 1 hour, corresponding to a minimum slab thickness of 100mm, as per Table 5 (Schedule 11) of the National Building (Structural Design) Code, 2019.

8.10 Minimum Design Guidelines

There were no design calculations available for the STC constructions inspected. The single drawing, signed by a registered engineer, was disregarded at construction stage, with changes in steel sections, timber sections, orientation of timber elements and connection details changed at construction stage.

All STC constructions inspected did not have lateral force resistance systems (such as shear walls, bracings), making them susceptible to failure under significant lateral forces such as wind gusts, seismic actions and impact loads. It is unlikely that the poor welds observed would be moment resistant. The masonry infill walls which could offer some bracing were not anchored into the main framing elements.

Guidelines

i. *All constructions should be designed by qualified professionals (Architects and Engineers) and the design drawings should be backed up by design calculations and based on appropriate standards.*
ii. *All designs should consider Checks for Stability, Robustness, Durability and Strength against gravity loads as bare minimum limit states.*

iii. *Any Structural detail should allow for ease of inspection and maintenance. As best practice, steel columns should be clear off the ground, supported on reinforced concrete pedestals.*
iv. *Design drawings should be detailed enough to guide the builder, with specifications on materials (size, grades, etc.) and assumptions underlying the design.*

8.11 Skilling for Welding

The poor quality of welds is testament to the lack of adequate skill among welders in construction. Most of the welders in construction are uncertified. There is an urgent need to train a critical mass of welder technicians, welder fabricators, welding Inspectors, welding supervisors, and welding engineers to ensure quality of weld connections in construction.

Guidelines

i. *Welding practitioners should be required to undertake training in welding offered at various UBTEB accredited training institutions.*
ii. *Welding practitioners who have learnt on the job should undertake periodic training through the Directorate of Industrial Training (under Uganda Vocational Qualifications Framework), which offers certification from Level 1 up to Level 5[15].*
iii. *The minimum level of certification for welders for structural construction works should be Level 4.*

8.12 Post Construction and Maintenance

It is common for changes to a design to occur during construction. It is therefore important that a check on fitness for purpose of the as-built is done

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[15] Level 1: working category, Helpers, casuals (Equivalent to Junior certificate) Level 2: WorkersPAS (Equivalent to National Certificate) Level 3 & 4: Supervision and Instructor Level (Equivalent to Diploma level)

before a structure is occupied, to confirm that it is safe for occupation.

Maintenance of structures is essential to ensure that they continue to fulfil the functional requirements over their design life, and to increase the service life of the structure.

Guidelines

i. *Developers should ensure that as-built drawings (Architectural, Structural, MEP) are produced at practical completion of buildings.*
ii. *Developers should obtain certificates of fitness of electrical and mechanical installations issued by engineers.*
iii. *Developers should apply for Occupation permits from the Building Committees after practical completion.*
iv. *Developers should carry out inspections annually and carry out preventive maintenance in case of any observed defects.*

9 CONCLUSIONS AND RECOMMENDATIONS

9.1 Conclusions

The technical committee set out on its work with six specific objectives and concluded on each as follows:

i. ***Review of the NBRB Study report and other documents to form an opinion on the viability of the STC method as marketed before the prohibition.***

Given the lack of adequate engineering basis, lack of composite action, lack of consideration for lateral resistance, lack of hogging moment reinforcement, poor welding quality, lack of specification of timber grades and properties for use, the poor timber on the open market; the lack of engineered connections; the method as marketed by the proponents and deployed in industry with its variations is generally unsafe and not viable from a structural point of view. Whereas the method is called 'composite', technically speaking, given that there is hardly any composite behaviour in the method and its variations. The poor quality of connections is particularly a huge risk since a structure is only as good as the connections bringing the different components together to work as a unit.

The Steel-Timber Concrete 'Composite' Building Method as marketed currently should remain prohibited. The prohibition should not extend to the possible use of the three materials of steel, timber and concrete in other combinations (for which minimum guidelines have been proposed) but to the STC method as marketed.

ii. ***Characterization of local Ugandan timber commonly used in structural members in construction in Uganda.***

The Characteristic bending strength of locally available timber as tested returned the following results: 26.3 MPa (Pine from Bwaise), 34.7 MPa (Eucalyptus from Bwaise), 15.7 MPa (Pine from Ndeeba), 35.9 MPa (Musambya from Ndeeba), 22 MPa (Mugavu from Ndeeba), 23.6 MPa (Musizi from Ndeeba) and 43 MPa (Eucalyptus from Ndeeba).

The results show that even for timber of the same species, the results could be markedly different. It is therefore not wise to allocate a strength class to a species since there are intervening factors such as age, growth conditions, growth defects and post-harvesting handling.

The Moisture Content in the timber was also very high (up to 75.1%) as compared to what would be specified for structural purposes (15-22% at the time of erection).

For initial design/ sizing, C14 and D18 classes can be used subject to confirmation through testing and statistical analysis to obtain characteristic values which are applicable to limit state design.

iii. Survey of structural steel on the Ugandan Market and check compliance to current design standards.

The results on yield strength fell within the range of 270-520MPa, indicating that it is feasible to achieve the expected yield values, as per NBC, 2019. 33 of the 34 samples hit a yield strength value of more than 275MPa, with a characteristic value of 332 MPa. Nine of the 34 samples (26.5%) failed to hit the minimum tensile strength of 410MPa, which is the specified tensile strength as per US ISO 603-2.

16 of the 34 samples (47%) samples did not meet the minimum required for elongation (21-23% according to US ISO 603-2, for S275), which implies low ductility.

Five of the 29 samples had higher carbon content than the maximum recommended for ductility; the higher the carbon content, the lower the ductility. 3 of the 35 samples had phosphorus and Sulphur amounts higher than the maximum recommended by the standards.

Only two of the 29 samples returned Carbon Equivalent Values higher than the recommended. It can then be inferred that generally structural steel as sampled are weldable.

iv. Survey of concrete practices in the construction industry in Uganda to ascertain compliance with specifications for typical works.

The results from the various mix ratios used in industry for target concrete Grade C25 returned Compressive strength results ranging between 26 MPa and 29 MPa. The fact that different mixes were used to achieve the same target underscores the need for project-specific mix design and trial mixes for each batch of materials delivered, rather than adopt an off-the-shelf mix design ratio, which is also many times misinterpreted by artisans.

The concrete from different sites had at 7 days also gained between 61% and 74% of the 28-day strength. The practice of assuming that the concrete gains about 70% of the 28-day strength, may not be applicable in all circumstances. Factors like cement type (Pozzolanic or OPC) as well as other factors such as materials may affect that strength gain. This further underscores the need for concrete mix designs for every project, and for every batch of materials delivered.

Most of the sites had no defined material quality assurance procedure. Both coarse and sand aggregates were basically sourced by the clients or their "Yinginiyas" and the purchasers (often unqualified to make an informed choice) made their own opinion or visual judgement of the quality of materials.

There was no defined procedure or formula on how the water quantity is determined for 86% of the sites. The workers simply used visual assessment to determine the quantity of water needed in any given concrete mix. Others had a known quantity of water that they constantly used on site but did not understand its derivation from the water cement ratio nor cared to vary it for wet aggregates in case it had rained on the uncovered aggregates.

The informal sector of the concrete industry is unaware about the different grades of concrete, quality control measures nor are they familiar with Mix ratios and importance of proper proportioning.

v. Development of guidelines for the design and construction of structures with steel, timber and concrete

The idea of using Steel, Timber and Concrete in structures can work, under scientifically tested guidelines and the application of core engineering principles. Ultimately the responsibility for a safe design lies with the professionals who are engaged by developers to realise their dream projects. They must satisfy themselves that the proposed design works at both the ultimate and serviceability limit states and issue a certificate of good structural practice.

Supervision is as crucial as design and therefore construction should be supervised by professionals, and preferably by the same person that designed.

The onus is on the developer to submit safe designs to the planning authority for approval. Developments that have no approved plans are illegal.

The Technical committee proposed guidelines on:

a) Structural Timber: *Sourcing of good quality timber, selection & handling, keeping timber free from moisture content, Durability Treatment against termite attack, Grading & marking, and quality assurance.*

b) Structural Steel: *Section sizing, Durability of Structural Steel- Protection against Corrosion, Durability of Structural Steel-Protection against Fire, Standardization and Labelling/Marking, Quality assurance.*

c) Concrete in Construction: *Sourcing, selection & storage of materials, Concrete Mix Design and batching, Quality control, reinforcing steel quality* *assurance*

d) Connections: *Welded connections, Bolted Connections, Timber to timber connections, Steel to Concrete connections, Concrete to Timber connections, Steel to Steel Connections*

e) Reinforcement of the concrete slab: *anti-cracking and flexural reinforcement Design & location, Concrete cover requirement.*

f) Mechanical and Plumbing installations: *Mechanical Ventilation and Air Conditioning Installation, Plumbing Installations, Fire Fighting Installations.*

g) Electrical Installations: *Proficiency level of technicians, Conduiting, protective devices, cabling.*

h) Fire resistance: *The Minimum fire rating should be at least 1 hour, corresponding to a minimum slab thickness of 100mm.*

i) Minimum Design guidelines: *Design by professionals, Checks for Stability, robustness, Durability & Strength at the very least, detail allowing for inspection & maintenance & buildability.*

j) Post Construction and Maintenance: *As-built drawings, occupation permits, maintenance needs.*

vi. Guidelines for welders of structural steel works in the building construction sector.

Welding practitioners should be required to undertake training in welding offered at various UBTEB accredited training institutions. Welding practitioners who have learnt on the job can undertake training through the Directorate of Industrial Training (under Uganda Vocational Qualifications Framework), which offers certification from Level 1 up to Level 5. The minimum level of certification for welders for structural construction works should be Level 4.

9.2 Recommendations

9.2.1 To Building Committees

i. To refrain from approving building plans without a design report, drawings showing connection details, sizing of structural elements, specifications for materials and geotechnical investigation reports.

ii. Should request developers of already existing STC structures to submit detailed Structural Integrity Assessment reports which should be carried out by registered engineers before an occupation permit is issued.

iii. Ensure that the STC structures which can be retrofitted, are strengthened and brought into compliance with the Building Control Regulatory Framework.

9.2.2 To Developers

i. Engage qualified professional engineers to carry out structural integrity assessments for existing STC constructions and make proposals for retrofitting.

ii. Apply for building permits for any proposed developments (including retrofits) and only start after issuance of the same from the appropriate building committees

iii. Engage qualified professionals to undertake all building operations, including design, construction and quality control.

iv. Apply for occupation permits from the appropriate building committees before occupying the buildings.

9.2.3 To UNBS

i. Require steel suppliers to indicate grade and manufacturer on the structural steel sections on sale.

ii. Enforce the requirement for all timber producers to mark their products as required by the standards.

iii. Formulate timber standards for engineering purposes, and make them mandatory.

9.2.4 To ERB

i. To discipline professionals who negligently stamp drawings for which a design basis is not available and who fail in their duty of care to clients.

ii. Arrange for Continuous Professional Development courses for welding engineers, instructors and inspectors.

iii. Require each Professional Engineer to file projects that they have certified to ERB as pre-condition for renewal of practising license.

9.2.5 To NBRB

i. Review the National Building (Structural Design) Code, 2019 to align with the current limit state design (Load and resistance factor design) philosophy.

ii. Plan to sensitise and roll out these guidelines with a timeline on review of the same.

iii. To consider the proposed guidelines as basis for a statutory instrument.

iv. Review the BCR, 2020 to include Welding Procedure Specification as a mandatory submission for steel structures for issuance of building permit.

9.2.6 To URSB

i. To require that Engineering and Construction firms should have at least one of the directors as a professional in a construction field.

ii. To consult the Minister in charge of works on any applications for patents or utility models that are of an engineering nature.

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Appendix 1: Proposals raised in the public consultative workshop held on 14th March 2024

	Suggestion/Comment from the Consultations	Status/Comment from Technical Committee
1	Be clear on whether the prohibition should be lifted after the guidelines or not	Technical Committee has clarified the position in the conclusion-the Prohibition of the STC building method should not be lifted.
2	Clarify on whether the guidelines are for the banned Steel-Timber-Concrete 'Composite' method or for use of a combination of the three materials in construction	Technical Committee has clarified in the text that the guidelines apply to use of 2 or 3 of the materials in combination, not necessarily the building method as banned.
3	Guiding the public on the cost of the novel scheme vis-a-vis the cost of the traditional methods of floor slab construction would be crucial on whether and when the scheme should be embraced. Point out that the cost of the scheme one adopts could depend much on the architectural plans and layouts of the building facility. The case in point is that the public takes it for granted that a maxpan slab is cheaper than a solid slab. But, obviously, this is not necessarily the case; it depends on the use of the building, the architectural layout, etc.	Cost comparison was out of scope of the work of the technical committee, given the different configurations that could exist even for the conventional methods. To be proposed to Universities for project research work by students
4	The guidelines are Ok but to be redetermined after further studies and cost implications	
5	Look at cost comparison with the known methods	
6	Talk about the issue of 'carbon footprint'	Out of scope of the Technical

	Suggestion/Comment from the Consultations	Status/Comment from Technical Committee
	considerations in the design and construction industries. How would this new scheme compare with the traditional methods?	Committee. The work of the technical committee was restricted to only safety aspects.
7	The challenges that the draft report highlighted; the lack of expertise and artisans for the scheme, etc. You will appreciate that not many people (including designers and supervisors) are conversant with the composite actions for structural elements.	Observations noted and agreed to. NBRB should work with UNABSCEC and DIT on skilling at all levels
8	More time is requested to skill the implementers	
9	The guidelines are generally fine but they should extend to the qualifications of carpenters and masons handling the timber and concrete respectively	
	Apart from the Engineer, the Mason/Crafts person should also have minimum qualifications	
	Propose regulations specifying minimum standards of builders/tradesmen on sites to ensure proper construction standards	
	Training a pool of certified builders for such facilities	
10	On the electrical guidelines, we could consider including a statement that target allowing for changing the cables installation in a building after a period of about 25 years after installation. This should be to reduce incidences of possible fire outbreaks arising	NBRB should work with UNBS to consider life span of electrical consumables and possible labelling of the same on the products.

	Suggestion/Comment from the Consultations	Status/Comment from Technical Committee
	out of old and overused/tired/damaged cables.	
11	No guidelines for safety and serviceability	The existing design standards and codes are sufficient for designers to account for all limit states
12	Implementers must be regulated to achieve the goals	Noted. NBRB as regulator to hand-hold Building Committees in district and urban authorities to carry out their statutory duty
13	Propose the new guidelines to be based on quality function deployment (House of Quality) to bring out: a) customer requirements b) design requirements c) competitive assessment d) correlation amongst design requirements e) relationship customer requirements with design requirements f) technical attributes devaluation	The work of the technical committee was restricted to only safety aspects
14	There should be improvement in connections	The guideline that connections MUST be designed by competent engineers has been included
15	Hollow sections should not be used but rather employ use of I-sections bolted on base plates	TC recommended preference for bolted connections and use of pedestals. Section selection is an output of engineering design
16	The way the slab is done is suspect; have gone through such structures and the floors vibrate as one walks. This should be looked into.	Vibration is a serviceability limit state. Design engineers should satisfy themselves that vibrations are within comfort levels of the users

	Suggestion/Comment from the Consultations	Status/Comment from Technical Committee
17	The guidelines should start with the design and analysis procedures. What you have presented are construction and implementation guidelines	Design is the preserve of design engineers. The method as marketed was not designed by professionals. Most safety risks arise in implementation
18	The proposed guidelines for the use of STC in construction is great, however, the guidelines should also consider proposing a collaboration and close interaction with the promoters to ensure realization of a middle ground so that the innovation is not shot down	NBRB being a public institution is open to any consultation, discussion aimed at making the built environment safer
19	All the guidelines are OK apart from that of developers getting timber from known and registered places showing grade. At least allow developers to have dry timber of at least 100mm x 100mm. Timbers direct from forests and being provided from forests less than a month should not be allowed.	For quality assurance, timber should be traceable from the forest to final use.
20	Additional items like the decking sheet for the slab.	Choice of material and configuration are the preserve of design engineers and functional requirements
21	We have to think why we are to move from steel. It is economically better and reliable	
22	Steel structures and buildings are OK and are constructed globally. The challenge with the composite is welded joints and use of wood	
23	Proper use of joints with bolts and decking sheets should be encouraged	
24	The guidelines have highlighted the key areas to be considered and followed in plan scrutinizing process	The guidelines can help Building Control officers and Building committees while considering

	Suggestion/Comment from the Consultations	Status/Comment from Technical Committee
		applications for building permits
25	In the presented guidelines, maintenance of structures should also be given a plan/ attention	Technical committee has included guidelines on post-construction and Maintenance
26	They should be further developed into a section of the code and regulations.	NBRB to consider using these guidelines as basis for a statutory instrument
27	Make some guidelines to become regulations otherwise, it can be misinterpreted.	
28	The guidelines should be stated as compulsory requirements so that they can easily be enforced	
29	The guidelines are good and will help improve performance, but we need to put up a team of experts, do pilot project before people are cleared to continue with the construction.	The aim of the TC is not to promote any particular method. The current method has been found non-viable and remains prohibited.
	The guidelines should be enforceable to avoid any gaps in the implementation. More studies should be conducted before this technology is opened.	
30	Punitive steps should be taken on culprits who endanger lives of innocent souls	It is an offence under s.42 of the Building Control Act,2013 to use a prohibited method or materials
31	Guidelines on quality assurance are captured, but not quality control especially for timber and steel	TC guided on the need of supervision of construction by professionals, whose core role at site is quality control
32	Propose minimum requirements for reinforced areas and or spacings as well as their detailing; including possible curtailment requirements	This a design output by the design engineer. Output depends on several factors, making it difficult for general guidelines such as

	Suggestion/Comment from the Consultations	Status/Comment from Technical Committee
33	Connections appear to be among the most troublesome challenges. Provide more specific guidelines e.g. beam to beam, beam to column, etc.	these to be specific to the requested for level
34	Minimum requirements for e.g. No. of and spacing of shear connectors missing	
35	Testing prototypes to verify simulated computer models	Out of scope for the TC. The onus is on proponents of novel methods to prove viability of concept
36	Development of standard building operation procedures by the STC promoters, subject to review by professional bodies to allow for safe construction.	
37	Guidelines should point designers to specific clauses of the existing modern design codes/standards	Design codes are reviewed from time to time and design engineers are expected to use UpToDate standards. The Guidelines are not a replacement of the existing codes and standards
38	Design guidelines for acoustics, ventilation and lateral load resistance seem to be captured quite narrowly	All these are limit states that Design engineers should consider. The TC guidelines are the absolute minimum for safety reasons
39	UNBS should come in and regulate the timbers as well as give specific grades for both hard and soft woods	TC has made recommendations to UNBS
40	NBRB (STC Committee) should give a guideline on testing steel because there are few testing laboratories for steel in Uganda.	TC has visited some laboratories and confirmed capacity locally to test
41	These guidelines need to be popularized through engineering schools/Universities and Colleges.	NBRB to carry out massive sensitization

	Suggestion/Comment from the Consultations	Status/Comment from Technical Committee
42	Engage the professional bodies such UIPE for workshops and sensitization	
43	We need the technicians and technologists on board as well	
44	The outcomes of the report should be shared with professional bodies (UIPE) to share with a wider scope of members and profile CPD	
45	Safety issues should also be captured	Safety was the premier consideration in drafting the guidelines
46	Cement classes are about % cement content more than strength and it is therefore better to specify cement by class and not by strength in the guidelines	Cement is specified by strength-32.5,42.5,52.5 (in MPa) and content of clinker e.g. CEM 1, CEMII, etc
47	The guidelines assume all basic principles of design and construction have been adhered to. Can we have some general remarks/statement that preliminaries have been done i.e. geotechnical investigations, survey, etc.	The assumption was based on the legal requirement that developers should engage professionals for design and construction. A professional would have covered all these bases
48	The Committee should document any successful project they met if any	TC did not encounter any satisfactory STC project
49	Proposal to have builders and suppliers' database for compliant companies/persons in order to properly guide developers.	UNBASCEC currently registers genuine contractors. This is work in progress to have this category regulated
50	With the adoption of these guidelines, do you think STC is safe?	It is not wise to sell a method of construction off-the-shelf. The TC has been clear that the STC building method as marketed is unsafe and not technically viable

	Suggestion/Comment from the Consultations	**Status/Comment from Technical Committee**
51	Can we have more specific conclusion?	Conclusions have been amended since the consultative workshop to answer specifically the objectives
52	The paper by Zziwa et al (2010) on strength properties of selected Uganda timber would provide some additional details	Both the paper and the PhD thesis were reviewed. Data given is based on allowable stress design which is now obsolete. The assignment of grades to species of timber is also risky since strength parameters even for the same species can vary significantly
53	Strength classification of structural steel is well assessed, but provide section classification as well, so as to offer guidelines on local buckling requirements.	These parameters can either be generated by the steel manufacturers or computed by design engineers once the yield strength and dimensions are available
54	Does the composite action assessment consider slip resistance?	A guideline has been provided on the necessity of designed shear connectors for any composite action to be assumed
55	Provide appropriate guidance on composite construction system	
56	Remedial measures for existing buildings should be indicated	TC has included recommendations to the Building Committees and Developers on the fate of pre-existing buildings
57	Pronounce on the existing sites and recommend way forward	
58	The QSM aspects are missing in the guidelines, especially; i) as built drawings ii) way forward on the already constructed buildings. What next?	
59	There is need to inspect existing STC buildings to assess their safety and viability	
60	I suggest the committee should not overlook	TC reviewed Timber-Concrete and

	Suggestion/Comment from the Consultations	**Status/Comment from Technical Committee**
	the issue of benchmarking not only regarding the built structure but the standard	Steel-Timber composite constructions
61	Propose bolting for use as connections in all structures due to limited skill set in the industry	TC has recommended bolted connections as preferred over welding
62	Recommend further research in this study	NBRB Research and Standards Department shall consider research in areas that still have knowledge gaps
63	At post construction aspect, it is important to clearly indicate in the guidelines as-built drawings should be produced	Guidelines on Post construction and maintenance have been included
64	As-built drawings must be provided and endorsed by the supervising structural engineer	
65	Professional bodies like UIPE, ERB & ARB should carry out trainings always to its members. Most professionals lack skills	Recommendation to ERB made on Continuous Professional Development sessions
66	The maximum W/C ratio of 0.5 for dry aggregates is misleading if the grade of concrete is lower. Therefore, I recommend that this W/C should have a grade of concrete limit	The aggregates should be in a Saturated Surface Dry Condition and the Minimum Concrete Grade for structural applications is C25
67	Issues of environment sustainability as far as timber is concerned should be included in the guidelines	Out of Scope of the TC. FSC certified producers would have sustainability aspects in their production chain

Appendix 2: Attendance list at the public consultative workshop held on 14th March 2024

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LOCATION: SILVER SPRINGS HOTEL, BUGOLOBI

DATE: 14TH MARCH 2024

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