FOUR-DIMENSIONAL DESIGN, ANALYSIS AND CONSTRUCTION OF A PRECAST CONCRETE PILOT HOUSE

By:

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DECLARATION

STUDENT'S DECLARATION

This thesis is my original work and has not been presented for a degree in any other university. No part of it may be reproduced without prior written permission from the author and/or Moi University.

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DEDICATION

This thesis work is dedicated to my new family; my fiancé *Fridah* and our lovely son *Alvin*.

ABSTRACT

This research entails the construction and overall structural analysis of a precast concrete pilot house. The house is constructed under the principles of *Four-Dimensional (4D)* design proposed by Hendrickx and Vanwalleghem (Hendrickx, 2002), in which basic elements are developed and an assembly criterion is adopted for construction. This work seeks to introduce the principles of 4D design into concrete construction to realize sustainable use of resources and qualitative use of the concrete constructions. The principles include; *design for dismantling* where materials are made to be technically separable, *design for deconstruction* where the components of the construction are made to be such that they can be disassembled and, *design for adaptability* in which the construction is made such that it is capable of adapting to emerging changes in user requirements (Debacker et al, 2007). The basic concrete elements developed in this research include columns and beams both of dimensions 80mm*80mm*1024mm, slab panels of dimensions 512mm*1024mm*24mm and connector cubes as cubic elements of sides 80mm. The assembly criteria adopted involve *dry connection* via bolts and nuts and embedded steel plates. Columns and beams are connected via the connector cubes to form a framework onto which double slab panels are anchored from the interior and the exterior wall elements respectively and connected by bolting via anchor plates embedded in the slab panels. Structural analysis of the construction was done at two levels akin to the construction stages; first upon assembly of elements to the window level and second, upon complete assembly of elements to the lintel level and the roof. The analyses were done under respective loads acting on the structure primarily dead loads and wind loads at each construction stage. The structure was modelled and analysed using SCIA Engineer software to simulate the behaviour of the structure under loading, to perform checks and to design the elements. The results of the analyses done and the completion of the pilot house construction demonstrated that the designed elements could satisfactorily be used to develop single storey precast concrete houses of up to 3m in span without partition walls while providing numerous configuration possibilities where partition walls are introduced. The cost analysis was done and showed that the pilot house costs about 30% higher than a typical masonry brick and mortar house of equal dimensions but the fact about cost saving by mass production, the advantages of disassembly, reconfiguration possibilities and demolition without wastes makes the construction technology more beneficial. Further to this, a more detailed design and analysis of the connections is being done by a colleague student *Kipkemoi Ng'eno* to assess the internal forces developed, determine optimal sizes and remove redundancies in the connection provisions. It is recommended that the possible use of single panel system for internal walls be investigated to further ease assembly process and ultimately lower the cost of construction under the 4D design technology. Finally, more research needs to be done on the development of alternative and cheaper connection systems.

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ACRONYMS

4D Design – Four-Dimensional Design

BS – British Standards

SCC – Self-Compacting Concrete

SAR – Stichting Architecten Research

H-V-D-A – Hendrickx-Vanwalleghem-Design-Approach

CHAPTER 1: INTRODUCTION

This chapter gives an overview of the 4D design concept and its application in construction. This concept was developed by Prof. H. Hendrickx and Dr. H. Vanwalleghem, (Hendrickx, 2002) as a design guide to introduce a dynamic view on the built environment. By designing adaptable construction systems, which are compatible with each other, a dynamic – and hence a sustainable – answer can be given to an unexpected and unpredictable future of a construction, (Debacker, 2006). These construction systems are made of a minimum number of basic elements and a set of combination rules. They allow development of various configurations by means of adding or removing elements and have potential for recycling and reuse. Further, this chapter contains the research statement, the research objectives, a summary of the research methodology and finally the limitations of this research.

1.1 Life cycle of construction materials

Three life cycles of the built environment can be discerned in a bid to study the wastes and energy flow. They are described in the *figure 1.1*, (Debacker et al, 2007) as: the cycle of the *building*, the cycle of the *components* and the cycle of the *materials* used to manufacture the components. From this figure, seven different paths can be followed in ascertaining the total energy and cost input, material uptake and the wastes so generated. These can further be categorised as follows:

a) Paths in which the building is partially or totally disassembled or demolished:
   In path I, land filling – all or a part of the sorted components is disposed by burying, and path II, combustion, all or part of the sorted components is burned to recover energy.

In path III, feedstock recycling, the sorted components are processed into feedstock to make building material, path IV, material recycling – where the sorted components are directly processed into a building material and path V, reuse of components – where sorted components are, after a short maintenance procedure taken back as the same components to perform similar or other purposes.

b) Paths in which the building remains standing:

In path VI, renovation and restoration of artefacts – the building remains standing while some parts are removed and new ones replaced in order to reinstate the artefact.

In path VII, reuse of the artefact, where only minor maintenance is required and the artefact remains fully useful.

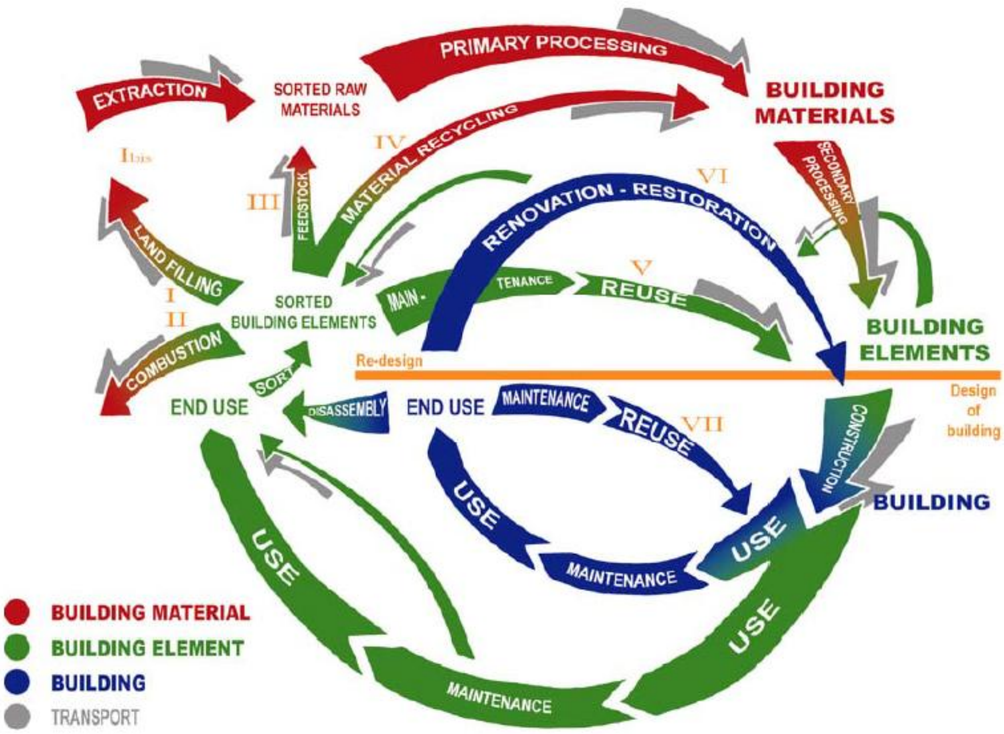


Figure 1.1: Incorporated life cycle model (Debacker et al, 2007)

1.2 The 4D Design Concept

1.2.1 Definition

4D Design refers to an attitude of the designer to consider the time dimension at the first stages of conception of an artifact. Whereas time is related to wear and tear of the artifact, there may also arise need for the artifacts to adapt themselves in spatial as well as structural terms to the changing and evolving circumstances. 4D strategies conceive building artifacts that support evolving processes in life and society instead of pre-designing for a specific situation, (Debacker et al, 2007).

1.2.2 The 4D Design Strategies

Three strategies are described in the 4D design systems as follow: (descriptions refer to *figure 1.1*)

a) **Design for Dismantling** – where the building material is made to be technically easy to separate according to their waste or re-investment treatment. Technical materials are recycled as feedstock or used in a lower grade function and biodegradables are returned into the natural cycle, paths III and IV, (Paduart, 2009).

b) **Design for Deconstruction** – here the building components are technically made easy to disassemble and to reuse multiple times, path V. Ten principles of the design for deconstruction process are described as follows, (Pulaski, 2005):

   i. Design for prefabrication, preassembly and stage construction
   ii. Simplify and standardize connection details
   iii. Simplify and separate building systems
   iv. Consider worker safety during deconstruction and construction

v. Minimize different components and materials

vi. Select fittings, fasteners, adhesives and sealants that allow for quicker disassembly and facilitate removal of reusable materials

vii. Design to accommodate deconstruction logistics

viii. Design to reduce building complexity

ix. Design for reusable materials

c) **Design for Adaptability/Flexibility** – buildings are technically easy to adapt to changing constraints. The design and re-design are flexible and can undergo inexpensive refurbishment procedure, path VI and path VII where the building is reused for the same or other preferred functions.

The use of open-space offices with modular wall panel systems enhances flexibility and adaptability of office spaces. While some aspects of flexibility may increase first costs, other aspects, such as open layouts and reduction of interior partitions, can help to improve constructability and reduce costs (Pulaski, 2005).

Possible changes over time are being incorporated from the first stages of conception of a building and its components. These design strategies take into account buildings life cycle in three different levels – the material, the component and the artifact while introducing adaptability, reuse and recycling, (Debacker et al, 2007)

This design vision enables different possibilities for reuse of materials and components on different levels of the built environment:

a. Relocation and/or reuse of an entire building,
b. Reuse of major parts of a building – renovation of a building,
c. Reuse of components in a building or elsewhere in the same building,

d. Reprocessing of components and materials into new components – recycling of materials into new materials.

1.3 4D Design and Construction in Moi University

The 4D design concept has been applied in two main construction materials in *Moi University* namely steel and precast concrete. In both cases basic elements were developed, assembly criteria adopted and the actual assembly (and disassembly) demonstrated by construction of models at the Civil & Structural Engineering Department Laboratory. The research on precast concrete particularly forms the background and the formative stage of this work.

1.3.1 4D Concept in steel construction

A study has been conducted on the application of the 4D concepts in steel constructions, (Okumu, 2010). This work focused on design and analysis of steel elements and their connections. Based on various design options, structural analysis checks and calculations of the proposed sections and elements were evaluated in accordance to the British Standards. Steel hollow sections and steel sheets sandwich panels were used as structural and walling elements in the model housing unit, a section of which was constructed as shown in *figure 1.2*. The results of the design and the analysis indicated that applying the 4D design principles, a housing unit can be constructed using only one type of structural section; the square hollow section of size 40mm*40mm*2mm was adopted as further discussed in *chapter 2*.

Figure 1.2: Picture of steel wall constructed under the 4D concepts, (Okumu, 2010)

1.3.2 Construction of a precast concrete wall using 4D concepts

The use of Self-Compacting-Concrete (SCC) in Moi University was initiated by a research group in which a preliminary research was done on mix designs where a good flow for the concrete was achieved although the strengths were low, (S. M. Shitote, 2007). This was attributed to the high water/cement ratio used. A further research to refine the mix design procedures was done, (Njeru, 2011), in which improved mix design ratios and procedures discussed in *chapter 2* were developed and as such adopted in the manufacture of precast concrete elements for construction of the 4D models. Previous work done on application of 4D concepts in precast concrete forms the predecessor of this research in which an assembled precast concrete wall was made from concrete slab panels and column elements, *figure 1.3*, (Sugut V. & Kipkemoi N., 2011). The dimensions considered were as follows: slab panels of 512mm*512mm*24mm and columns of 80mm x 80mm in cross-sections and length of 1536mm. The slab panels were developed to have two embedded steel plates on each vertical edge with provisions for bolting to the columns. The columns have in-cast threaded rods protruding on the face at points of connection. The technique was successful to the extent of practical assembly and disassembly but could only permit linear development. A further research – the construction of a precast concrete assembled cube – was therefore necessary to device a connection that makes it possible to build around corners and introduce partitioning, that is, T-junctions.

Figure 1.3: Picture of the wall constructed as part of the 4D Concrete research

1.3.3 Development of an assembled precast concrete cube using 4D concepts

An assembled cube was constructed using slab panels of 512mm*512mm*24mm, beams of 80mm*80mm*512mm and connector cube elements of 80mm*80mm*80mm. The connector cube element, *figure 1.4a,* sufficiently provided for connection to the six-face directions which facilitated development of the assembly to various configurations shown in *figure 1.4b.* To the four vertical faces, beams can be connected and on the top and bottom faces columns can be connected. The connection method entails introducing cast-in nuts on the connector cube on all faces, *figure 1.4a*, through which studs are fastened to protrude and upon attachment of the column or beam element via endplates, nuts are fastened. Slab panels are connected to the beams and columns via anchor plates embedded as illustrated in *figure 1.4c*. The beams and columns are designed to have steel endplates which have bolt holes to provide for fastening as illustrated in *Figure 1.4d.*

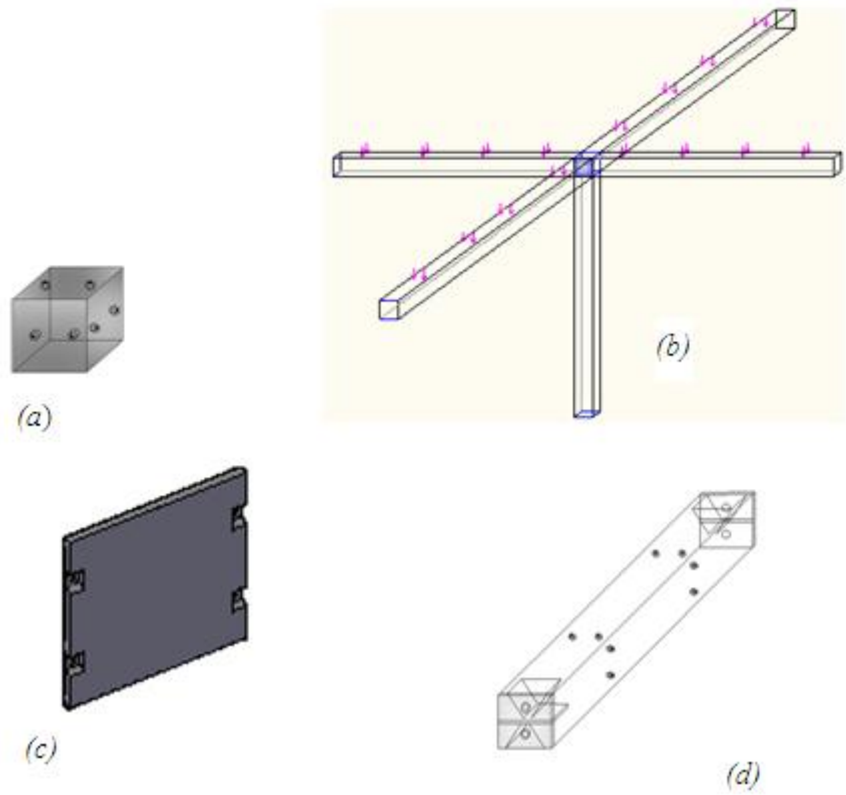


Figure 1.4: Illustrations of (a) Connector cube, (b) Connections to connector cube, (c) Slab panel and (d) Beam or column (not drawn to scale)

The assembled cube so constructed is illustrated in *figure 1.5*. It demonstrates the possibility of using the connector cube to assemble elements around corners and hence form a closed cube.

Figure 1.5: Picture of the assembled cube

1.4 Research statement

The success of the preceding research in which a precast concrete wall and an assembled cube were developed indicated a possibility of applying the 4D concepts in precast concrete works. This research therefore endeavours to apply the concepts and conceptional designs as preliminary basis to demonstrate applicability of the 4D concepts by building a model pilot house. To achieve this, basic elements need to be designed, manufactured and with an assembly criterion and procedure adopted, the house can be constructed. Further, structural analysis is required to ascertain the strength capacities of the components and the structure, and the behaviour of the structure under various loading conditions and thereby drawing conclusions and providing recommendations on the construction technology employed.

1.5 Objectives

1.5.1 Main objective

To carry out the design, construction and overall structural analysis of the 4D precast concrete pilot house.

1.5.2 Specific objectives

To attain the above objective the following sub-objectives have to be satisfied:

1. To design and fabricate the precast concrete elements,
2. To assemble the precast concrete pilot house,
3. To perform structural analysis of the structure,
4. To assess the cost implications of the construction technique

1.6 Research methodology Summary

The dimensions of the elements, the components and hence of the pilot house are derived from the H-V-D-A dimensioning system, (Hendrickx, 2002) as discussed in *chapter 2* and applied in *chapter 3* with reference to the preceding researches that form the preliminary design. With these dimensions, steel moulds are designed for each element incorporating the different provisions for connections. The moulds and the connection components are fabricated in the *Mechanical and Production Workshop*, where the main activities include marking, cutting, welding and grinding steel plates, steel rods, steel bars, bolts and nuts as per the respective design requirements, *chapters 3.3* and *3.4*, here the hand-services of workshop technicians is sought from time to time.

The process of manufacture of the precast concrete elements begin in the *Concrete Laboratory* by preparation and proportioning of the concrete materials including sand, coarse aggregates, cement, lime filler, water and steel bars or mesh reinforcements. The moulds are then assembled and the required connection attachments and inserts and reinforcements are positioned. The mixing and testing machines and equipment set up and the process of mixing and placing of concrete in the moulds is done, *chapter 3.5.*

Construction of the pilot house is done in two construction stages, *chapter 4*. Since two column elements are connected via connector cube element to reach the roof level, the first construction stage constitute elements assembled up to the level of the first column (or window level) and includes column anchorage to the floor, the columns, beams, connector cubes and slab panels. The second construction stage constitutes the second column element and the slab panels. The roof, window/door fittings and finishes are then added. The construction is done in Moi University at the Department of Civil and Structural Engineering.

Analysis of the structure is done according to the *British Standards* (BS), first by hand where the loads are derived including the roof loads, the dead loads and the wind loads. SCIA Engineer Software is used to model the structure and with the derived loads and constraints, the behaviour of the structure is simulated, the design for reinforcement of elements is done and checks on deflection are done to determine the allowable free wall span for this construction technique, *chapter 5*.

The cost implication of this construction system is done in *chapter 6* and a comparison is made with conventional construction techniques. From the structural analyses carried out, the complete construction of the assembled pilot house and the cost implication determined, conclusions are made as to the practicability and viability of the system and recommendations drawn for furtherance of this work, *chapter 7.*

1.7 Limitations of the research

The 4D principles are applied only to the wall elements including slab panels, beams/columns and the connector cubes including the window/door openings and the partitions but not to the roof and the floor. The roof structure is conventionally made from steel lintel beams, steel trusses and galvanised iron sheets which can be disassembled, whereas for the floor, a typical concrete floor was adopted. This research is undertaken jointly with a colleague student and this report is limited to the overall analysis while detailed local analysis of the connections is done and reported by the other student in a separate report.

CHAPTER 2: LITERATURE REVIEW

2.1 Introduction

This chapter highlights the works previously done and other literature behind the various aspects of this research. Important aspects include the 4D design approaches and applications, the background of SCC, its application in research and in the in-situ and precast constructions, and the connection methods and their requirements. The aspects of shrinkage in concrete and tolerances in prefabrication construction are also discussed.

2.2 4D Design Approaches

2.2.1 The SAR Approach

The SAR, STICHTING ARCHITECTEN RESEARCH, of 1965 developed a design approach for adaptable dwellings wherein users could choose their own living environment and support the changing needs and requirements. In 1965 the initiative lead by J. Habraken, was geared toward intensive mass construction of dwellings as a result of the housing shortage in the Netherlands after the World War II. In that period prefabricated dwellings were produced on a massive scale to provide dwelling comfort at a reasonable price. The SAR 65 approach is based on three main pillars, (Debacker, 2006):

a) The concept of 'support and infill' – where the infills are separated from the supports for ease of adjustments, *figure 2.1.*

b) Modular coordination system – in which dimensional compatibility is allowed to permit use of different infill materials as one may wish, and,

c) The use of zones and margins – where a series of variants in which the various infill options for different supports are compared. A methodology based on the concepts of zones and margins, has been developed to facilitate the design of the supports. This methodology proposes four different types of zones, characterizing the structure of the housing:

   i. **Alfa-zone**: the area intended for private use, situated inside the construction and having a relationship with the outside;

   ii. **Beta-zone**: the area intended for private use, situated inside the construction and having no relationship with the outside;

   iii. **Gamma-zone**: the area intended for private use, situated at the inside or the outside;

   iv. **Delta-zone**: the area intended for private use, situated at the outside.

This system of construction for temporary dwellings and adoption of separated supports and infills and the zonings that enable configuration and reconfiguration forms part of the background basis of the 4D concepts as adopted in this research.

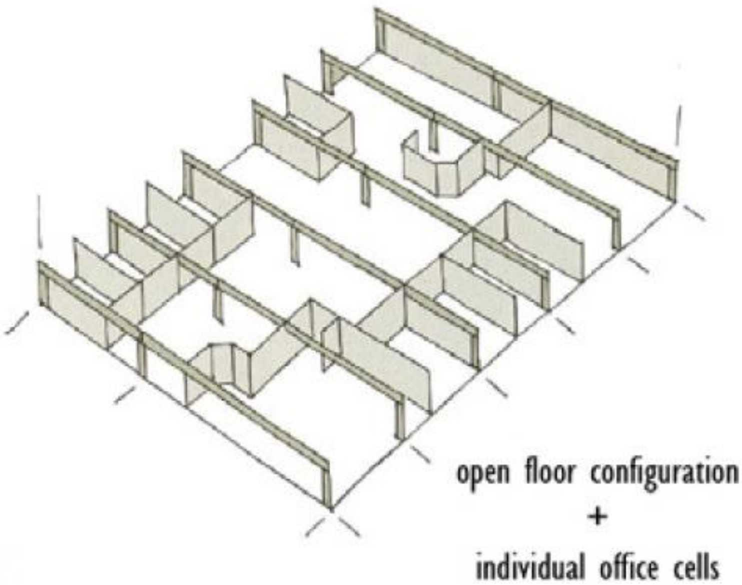


Figure 2.1: Support and Infill system of open floor and offices

2.2.2 The HENDRICKX–VANWALLEGHEM Design Approach (H-V-D-A)

This approach recognizes that there are only three basic dimensions by which any physical object can be defined; the length *L*, its breadth *B*, and its height *H*. From these, three basic forms are derived, the cube or cuboid, the sphere and the prism. All these have the *L*B*H* dimensions.

Three types of basic elements can be realized on combination of the basic forms:

I. Line elements (one-dimensional)

II. Plane elements (two-dimensional)

III. Volume elements (three-dimensional)

The H-V-D-A provides two design tools: a generating form and dimensioning system; and a method for developing design catalogues. The former case has been adopted in this research.

Design tool 1: A Generating Form and Dimensioning System

The ‘generating form and dimensioning’ system is core to the H-V-D-A concept. It includes a set of standardization rules through which forms and dimensions of all basic elements are regulated. H-V-D-A presumes that any tangible basic element, in any construction phase, can be approximated with a minimal diversity of basic forms. The concept considers the square, its diagonals and the inscribed circle, *figure 2.2*. This makes sense since right angles are found in many material solutions and certainly in the area of construction, (Debacker, 2006). To make effective use of the proposed system, the set of basic forms should be provided with basic dimensions. In order to achieve optimal flexibility and combination, the basic elements should have the same dimensions. Because dimensional differences will be unavoidable, Hendrickx and Vanwalleghem propose to solve the problem using the rules of either halving or doubling. Both are the result of an easy mathematical function and create a geometrical series. Starting with a square with side ‘*x*’ one finds: *x*, *2x*, *4x*, *8x*..., *Figure 2.2*, (Debacker et al, 2007).

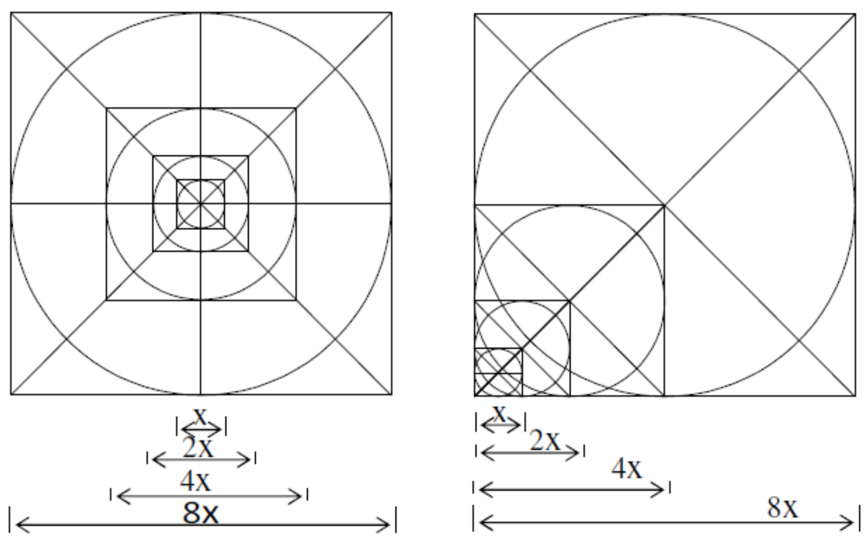


Figure 2.2: The generating form and dimensioning system (Debacker et al, 2007)

Dimensioning starts with the smallest visible unit and is followed by basic operations such as addition, subtraction, multiplication or division. Hendrickx and Vanwalleghem propose to take 1mm as the start point, whereas other start points could be 1cm, 1 dm, 1 inch, or 1 foot. Using the 1mm start point the following metric system can be developed by multiplying the predecessor by 2 to form a sequence:

1 – 2 – 4 – 8 – 16 – 32 – 64 – 128 – 256 – 512 – 1024 – 2048 – 4096 – ... mm

0.1 – 0.2 – 0.4 – 0.8 – 1.6 – 3.2 – 6.4 – 12.8 – 25.6 – 51.2 – 102.4 – 204.8... cm

0.01 – 0.02 – 0.04 – 0.08 – 0.16 – 0.32 – 0.64 – 1.28 – 2.56 – 5.12 – 10.24 – 20.48 – 40.96 –. dm

Dimensioning of elements is then done by:

- Picking a value from the sequence,
- Adding or subtracting values picked from the sequence,
- Projection over given scales on a fractal model, considering the form and constitutive material. This also defines how the elements should be joined and combined into a building component, *figure 2.3*

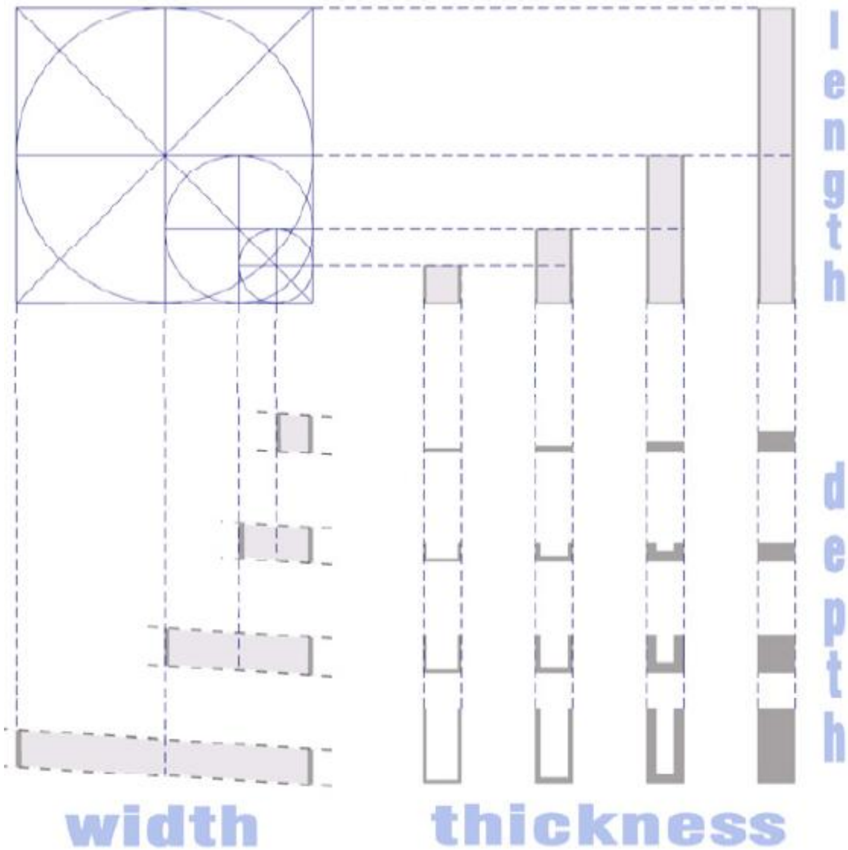


Figure 2.3: Dimensioning basic elements through a fractal model of the generating system (Debacker et al, 2007)

Design tool 2: Developing Design Catalogues

Theoretical design catalogues are developed and used as an aid to the development of construction systems. In a first step each material solution or, more precisely, each of its construction elements – in whichever phase – is objectively and verbally described, based on characteristics, strengths and weaknesses. Each characteristic has one or more parameters as a counterpart, all bracketed between predefined limits. This delimitation, for each parameter, is done at the level of the entities (Henrotay, 2008).

Since all artifacts are measurable, they can therefore be depicted in the catalogue. Variants of different sizes are attainable through interpolation and combination of outer elements in the series. Using arithmetic and geometric calculation rules, changes in each series can be described; every value of the parameter thus gets its place within the series. To achieve the goal of using a minimal number of basic elements, stepwise variations in each series are preferred. The adopted geometrical standardization corresponds with the fractal model in the generating system, (Debacker et al, 2007), (Henrotay, 2008). A theoretical design catalogue can thus be established for combining elements such as the one for corrugated plate given in *figure 2.4*.

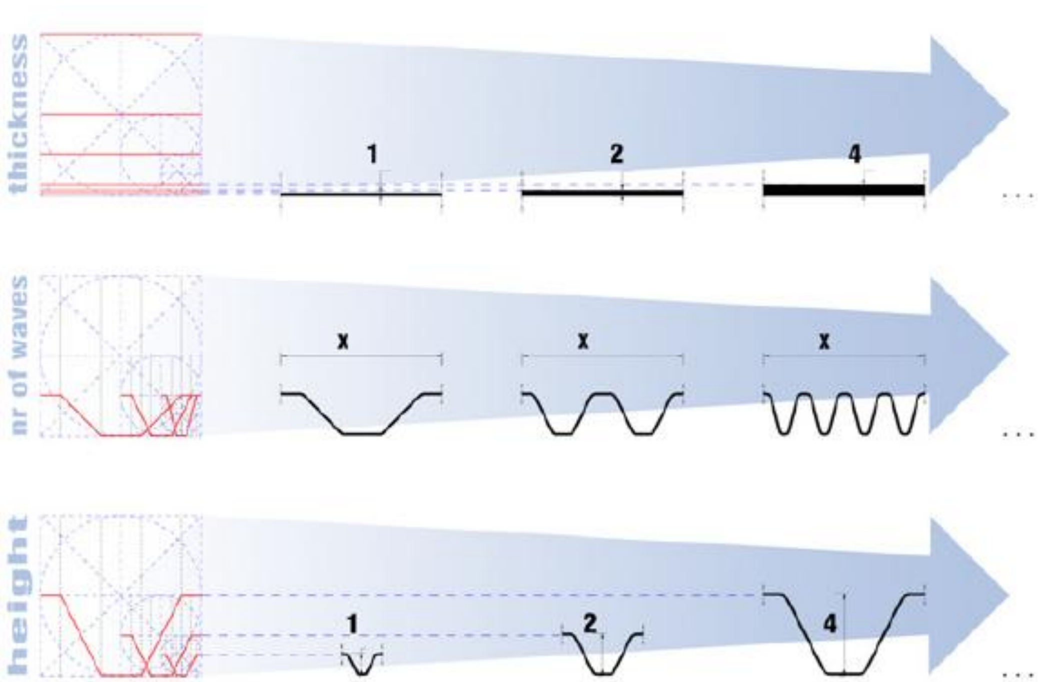


Figure 2.4: Theoretical design catalogue for a corrugated plate.

2.2.3 Cradle-to-Cradle Approach

Arch. W. McDonough and chemist M. Braungart criticize the way the material culture is designed and created through their manifesto 'Cradle to Cradle: Remaking the Way we Make Things' (McDonough W., 2002), they argue that many buildings and daily products are made on the basis of linear, cradle-to-grave flow, (Debacker et al, 2007), *figure 2.5*. The building materials such as concrete and masonry are often down-cycled in a bid to extend the lifespan although this process reduces the quality of the materials, making them suitable for use only in the lower value application such as sub-base in road construction. Gradually some of these materials end up as wastes.

McDonough and Braungart propose to use the concept of eco-effectiveness, that is, the transformation of products and their associated material flows such that they form a supportive relationship with the ecological system and future economic growth. They propose to use a cradle-to-cradle system. In short, they imply that a designer should design for disassembly and design for re-use as envisaged in this particular research.

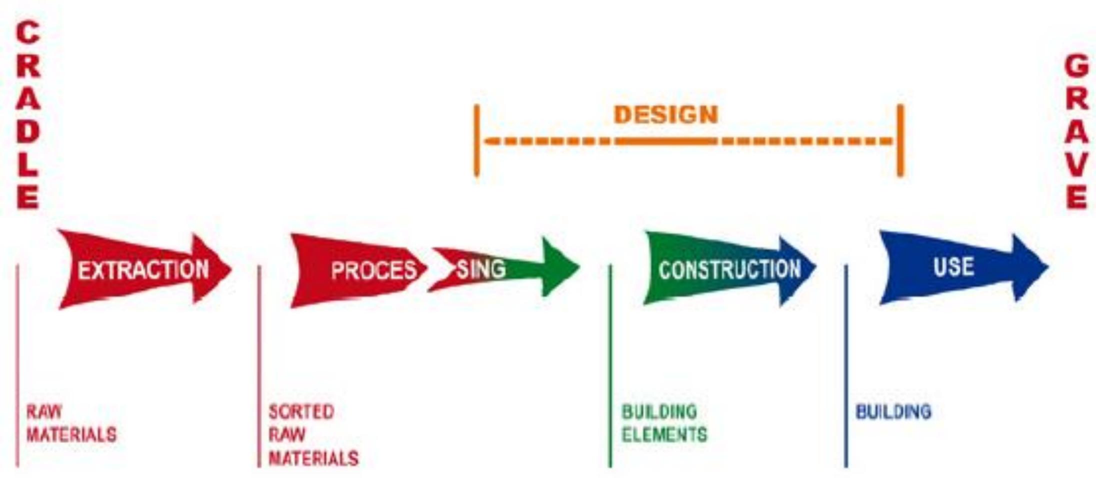


Figure 2.5: Linear model flow of materials in the built environment; Cradle-to-Grave, (Debacker et al, 2007)

2.3 Use of Self-Compacting Concrete (SCC)

2.3.1 Background of SCC

Self-compacting concrete (SCC) is an innovative concrete that does not require vibration during placing and compaction. It is able to flow under its own weight, completely filling formwork and achieving full compaction, even in the presence of congested reinforcement. The hardened concrete is dense, homogeneous and has the same engineering properties and durability as traditional vibrated concrete. Self-compacting concrete was initially developed in Japan in 1988 at the University of Tokyo by Prof. Okamura and was made available for use in 1989. However, a proper mix design and procedure was not available until 1992.

SCC has been applied in many structures since early 1990's and continues to gain popularity especially in the developed world. By the year 2000, the amount of SCC used for prefabricated products and ready-mixed concrete in Japan alone was about 400,000m^3 (Takada, 2004).

SCC offers a rapid rate of concrete placement, with faster construction times and ease of flow around congested reinforcement. The fluidity and segregation resistance of SCC ensures a high level of homogeneity, minimal concrete voids and uniform concrete strength, providing the potential for a superior finishes and durability of the structure. SCC is often produced with low water-cement ratio providing the potential for high early strength, earlier demoulding and faster use of elements and structures, (Nakamura, 2003).

2.3.2 Application of SCC in the Precast Concrete Industry

SCC is sensitive to variations in the condition and proportioning of materials and the environment, (Ouchi, 2003). This is however not a considerable challenge for the precast concrete industry since the precasting is done under controlled factory conditions and has the following advantages in addition to the advantages of SCC mentioned above:-

- Substantial reduction in on-site labour and noise levels

- The reduction of dust in the air due to vibration

- The reduction of wear to the formwork
- The use of less robust formwork/moulds with simpler connections
- The possibility to produce elements with various architectural designs and quality, *figure 2.6.*
- Very high strength concrete can be manufactured, up to 60MPa without difficulty, (Nakamura, 2003). With further treatment more superior strengths can be achieved.

Figure 2.6: Unique architectural designs and qualities of SCC, (Walraven J. , 2009)

2.3.3 Application of SCC in In-situ Constructions

The introduction of SCC in in-situ constructions is slower than in the precast concrete industry. There are a number of reasons for this, (Walraven J. C., 2001):

- In case of failure the consequences for an in-situ application are much more severe than in the precast concrete industry. In the latter case the unsuitable elements can simply be rejected, whereas in the first case demolition might be the ultimate consequence.

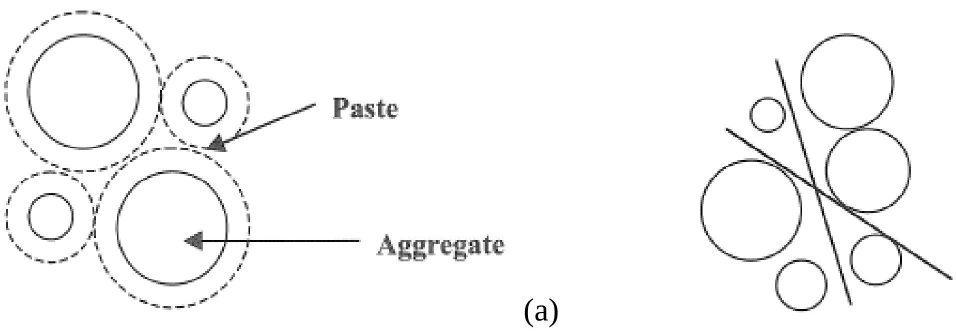
- There was often no agreement on the way in which the properties at the building site have to be controlled.

- Self compacting properties can be more easily reached with higher strength than with lower concrete strength. In a number of practical applications the concrete strength was therefore higher than actually necessary, which has cost consequences.

2.3.4 Studies on SCC

a. Fibre reinforcement of SCC

Fibres are available in a wide range of materials, at different shapes and with different properties concerning their affinity to water or cement paste. Some types of fibres are fragile, flexible or stiff, cylindrical, rectangular or irregular. Anyhow, they are known to improve the hardened properties of SCC. On the contrary they tend to lower the workability and flow characteristics of fresh state SCC, (Walraven J. C., 2001). In fact long fibres tend to entangle and build 'nests' of fibres which further counteracts the flow. To compensate this, more fines are added, *figure 2.7.*



(a) (b)

Figure 2.71: Illustration of concrete mix, (a) aggregates surrounded by paste, (b) Aggregates with fibres added

Fibres bridge cracks in concrete and retard their propagation. They contribute to an increased energy absorption compared to plain concrete and hence in SCC it allows exploration of even more special applications, (Grunewald, 2004). For a very long period it was noted that the addition of fibres to concrete decreased the workability. However, in his PhD-thesis Grünewald, (Grunewald, 2004), showed that this is not necessary at all. He proved that self-compacting fibre concretes are very well possible, even up to fibre contents of 140 kg/m^3, if the right combination of fibres and mixture composition is chosen. *Figure 2.8* shows the maximum possible fibre content for which mixtures are still self-compacting (defined as having a flow circle with a diameter of at least 600mm, a round shape and a homogeneous fibre distribution). At the vertical axis the fibre content in kg/m^3 is given. At the horizontal axes the fibre type (aspect ratio/length) and the mixture type (with the sand/gravel volume ratio) are given. This aspect of fibres in SCC can be of important input in precast concrete such as in this research to the extent of the ease of moulds assembly with reduced fixtures such as the reinforcements.

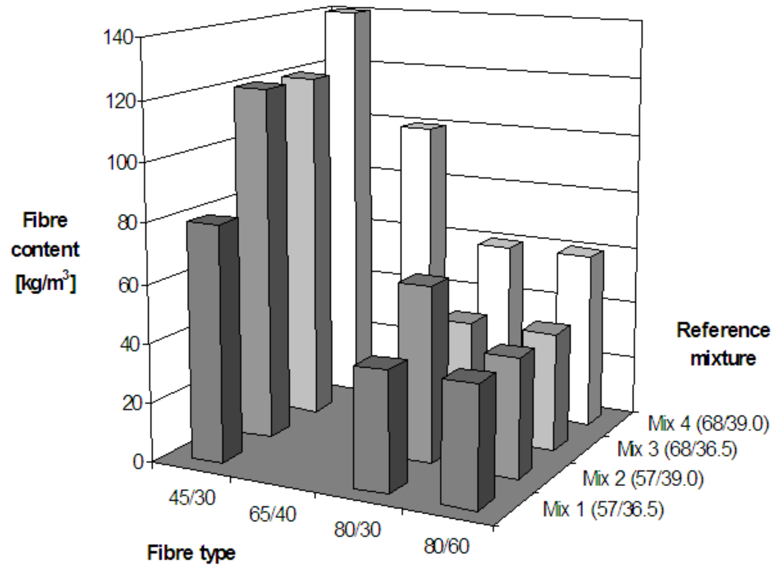


Figure 2.82: Fibre content/type to produce SCC, (Grunewald, 2004)

b. Mix design procedures for SCC in Moi University

Experiments were conducted with the objective of producing mix design procedures for SCC using crushed stone aggregates as the coarse material, (Njeru, 2011). The arrangement was to vary the water/cement ratio by weight from 0.35 - 0.65, *table 2.1*, at intervals of 0.05 for a given concrete mix. In this way, a relationship between strength and water/cement ratio could be observed. Further, the super-plasticizer requirement for the various water/cement ratios was done while the other concrete ingredients were kept at a constant. Particularly, coarse aggregates were kept at 30%, fine aggregates (sand) at 40% and filler (lime) at 20%. The results for the strengths and super-plasticizer requirements for the water/cement ratios are as in *table 2.2*.

Table 2.1: Range of constituent materials for SCC

Constituent	**Range**
Water/cement ratio (by weight)	0.35 – 0.65

Coarse aggregates	30% - 40% of total volume
Super-plasticizer dosage	0.8 – 1.5 litres/100Kg of cement
Filler	20% of mortar volume
Fine aggregates	40% – 50% of mortar volume
Powder (cement and filler)	500Kg/m^3 – 600Kg/m^3

Table 2.2: Relationships between water/cement ratio, super-plasticizer dose and strength

Water/cement ratios	0.35	0.40	0.45	0.50	0.55	0.60	0.65
Super-plasticizer dosage (litres/100Kg of cement)	2.40	1.50	1.20	0.95	0.90	0.85	0.80
7 Day strengths (N/mm^2)	31	32	29	23	28	15	10

The procedure was repeated at all desired water/cement ratios and the same for a varied coarse aggregate content including 35% and 40%. A statistical technique for modelling the mix design data was adopted, that is, the central composite design (CCD) technique. The CCD technique is an experimental design tool for building a second order (quadratic) model for the response variable without need for a complete three-level factorial experiment.

The research was able to come up with the following findings:-

a. Concrete mixes that were highly homogeneous,
b. Concrete test cubes had smooth finishes,
c. There was neither bleeding nor segregation when the right super-plasticizer was used,

d. The retention for the concrete flow-ability was longer for the chosen lime content, and

e. From the CCD analysis, a tabulation of the optimum volume proportions for specific SCC classes was compiled and is excerpted in *table 2.3* for a SCC class 20 that yielded 28-day strength of 30.6N/mm^2, (Njeru, 2011).

Table 2.3: Optimum volume proportions for specific SCC classes: run 15, (Njeru, 2011)

Run	**Water/cem-ent ratio (by volume)**	**28-day strength (N/mm^2)**	**Fine Agg.**	**Coarse Agg.**	**Super-plastic-izer**	**Lime**	**Water**	**Cement**
15	1.503	30.60	0.26	0.35	0.0041	0.041 2	0.2069	0.1377

c. Strength Development in SCC

In concrete technology and indeed precast concrete construction, the knowledge of early strength development of concrete and the strength of the fresh concrete is important in the practice of demoulding of the precast elements, pre-stressing and lifting or transportation of the precast elements. Strength is – among other factors – a function of time and temperature. There are several methods of determining the maturity of concrete, important of which is the '*weighted maturity method*' developed in the Netherlands by Ronald de Vree, (Tegelaar, 2000).

Weighted maturity, *Rg*, is the sum of the products of concrete curing times and temperatures on fixed measurement intervals, taking into account the influence of the temperature sensitivity of the binder. It is calculated as follows, (Krikhaar, 2005):

$$R_g = \frac{10(C^{0,1T-1.245} - C^{-2.245})}{ln\ C}$$

Where; R_g is the weighted maturity of concrete after 1 hour,

T is the mean temperature of the concrete in that hour, and

C is the C-value of the cement or the binder used. The C-value is the number representing the temperature sensitivity of the binder and is obtained from the supplier of the binder or developed as a standard.

With experimental data of temperature and time during production and curing of concrete, two graphs can be developed as shown in *figures 2.9* and *2.10.*

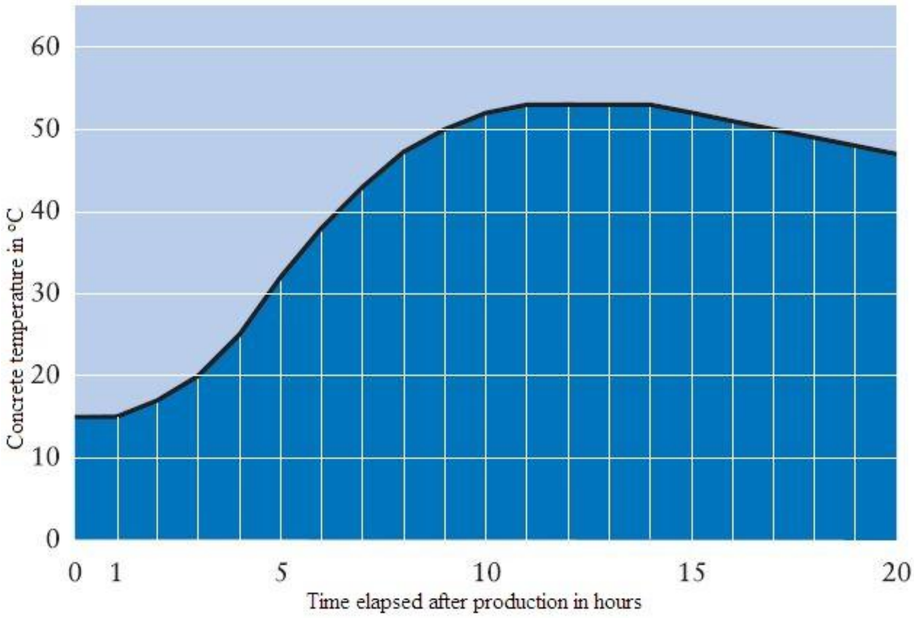


Figure 2.9: Concrete temperature (^oC) versus time (hours), (Krikhaar, 2005)

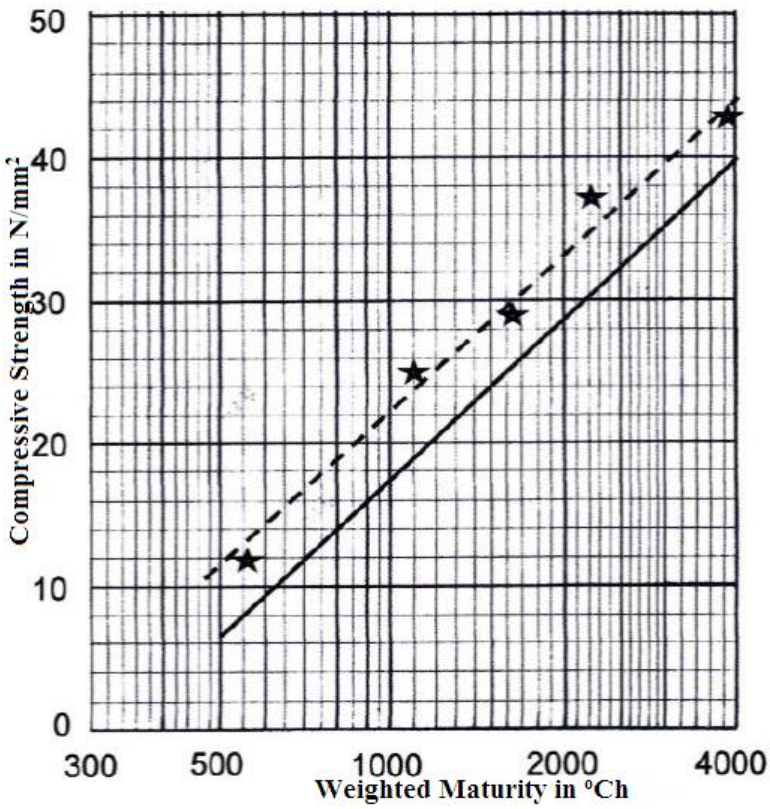


Figure 2.10: Compressive strength in N/mm^2 versus weighted maturity in ^{o}Ch, (Krikhaar, 2005)

For the first 20 hours of production, for instance, at a temperature of 30^{o}C, the weighted maturity from the *figure 2.9* is about 550 (approximately 55 rectangular units of 1hour *10^{o}C, hence 1hour*10^{o}C*55units = 550). With this value, and *figure 2.10*, the compressive strength attained after 20 hours of production of the concrete can be deduced to be about 8N/mm^2.

This method relies on the principle that the achieved concrete strength for a concrete mixture with a given composition at each time point has a solid relationship with the weighted maturity at that time, regardless of the temperature gradient along which this weighted maturity was reached. In practice, this method for fresh concrete is reliable.

2.4 Connections in precast elements

2.4.1 Selection of connection methods

Connections are very important in any assembly construction since they are the weak points in the construction process hence their strength capacities ought to be assured. In general, efficient connections have the lowest detailing/complexity, low labour in fabrication and erection and are durable, (BCSA, 2011). The factors considered when selecting a connection detail may include: connection type, steel grades and sizes of fitting, bolt grades/sizes and lengths, weld types and sizes, and the geometry to be adopted. Particularly for purposes of disassembly, additional factors to be considered include: the speed of construction anticipated, the reuse of the connection and the ease and potential of disassembly, (Henrotay, 2008).

For effective and efficient connection options, the following are some considerations and their benefits, (BCSA, 2011):-

a. A reduction in number of different connection types allows better understanding of their costs and performance,
b. Use of locally available materials improves availability, reliability, and saves on costs,
c. Use of one grade and diameter of bolts if possible and in a limited range of lengths saves time in changing drills and hand tools, quickens erection and reduces potential errors on site.

A criterion developed for selection of preferred connections, specifically with the potential of disassembly and reuse of the construction elements, connections and the connection components illustrated by *Henrotay*, (Henrotay, 2008) is as in the summary *table 2.4*. As in the table, the connection by bolting and utilization of friction are highly preferable. It has widely been used in steel frame and structures construction requiring assembly and disassembly and also in 4D designs application in steel, (Okumu, 2010).

Table 2.4: Criterion for selection of a connection system with disassembly potential (Henrotay, 2008)

	type	**speed of construction**	**strength of connection**	**reuse of connection**	**deconstruction potential**
mortar bonding	*infilled*	-	- *to* +	- - *to* -	+/-
adhesive bonding	*infilled*	+/-	- *to* ++	- -	- -
welding	*infilled*	+/-	++	-	- -
resin bonding	*infilled*	+/-	++	-	- -
nail fixing	*direct*	+/-	+/-	+/-	+/-
riveted fixing	*direct*	+	+	+/-	-
bolt fixing	*indirect*	+	+	+ +	+ +
screw fixing	*indirect*	+	+/-	+	+
friction	*indirect*	+	-	+ +	+ +

- -, *none;* -, *limited;* +/-, *average;* +, *substantial;* + +, *extensive.*

2.4.2 Information on bolt connection systems

a) Typical precision bolt/nut system

An illustration of a typical hexagonal head bolt/nut system is given in the *figure 2.11*, with an elaboration of the details and abbreviations given in the following *table 2.5*.

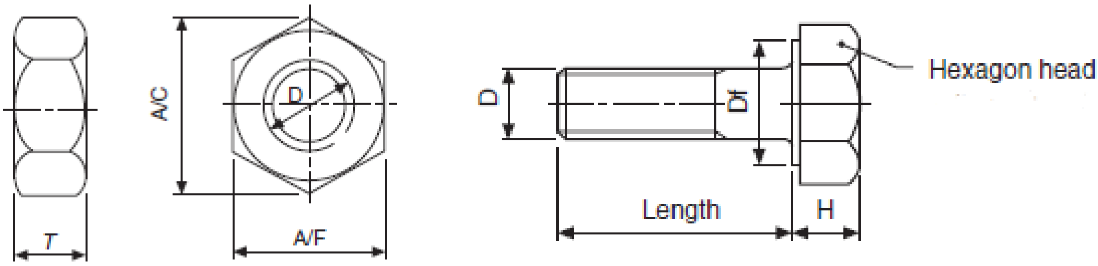


Figure 2.11: Illustration of typical bolt/nut system

b) Table of dimension details

Table 2.5: Table of dimension details of ISO Metric bolt/nut systems,

Nominal size thread diameter D	Thread pitch	Minor diameter of thread	Width across corners A/C	Width across flats A/F	Diameter of washer face Df	Height of bolt head H	Thickness of normal nut T	Thickness of thin nut t	Washer inside diameter	Washer outside diameter	Washer thickness Form A	Washer thickness Form B
M1.6	0.35	1.1	3.5	3.0		1.0	1.25		1.7	4.0	0.3	
M2	0.4	1.4	4.5	4.0		1.5	1.5		2.2	5.0	0.3	
M2.5	0.45	1.9	5.5	5.0		1.75	2.0		2.7	6.5	0.5	
M3	0.5	2.3	6.0	5.5	5.0	2.0	2.25		3.2	7.0	0.5	
M4	0.7	3.0	8.0	7.0	6.5	2.75	3.0		4.3	9.0	0.8	
M5	0.8	3.9	9.0	8.0	7.5	3.5	4.0		5.3	10.0	1.0	
M6	1.0	4.7	11.5	10.0	9.0	4.0	5.0		6.4	12.5	1.6	0.8
M8	1.25	6.4	15.0	13.0	12.0	5.5	6.5	5.0	8.4	17	1.6	1.0
M10	1.5	8.1	19.5	17.0	16.0	7.0	8.0	6.0	10.5	21	2.0	1.25
M12	1.75	9.7	21.5	19.0	18.0	8.0	10.0	7.0	13.0	24	2.5	1.6
M16	2.0	13.5	27.0	24.0	23.0	10.0	13.0	8.0	17.0	30	3.0	2.0
M20	2.5	16.7	34.0	30.0	29.0	13.0	16.0	9.0	21.0	37	3.0	2.0
M24	3.0	20.0	41.5	36.0	34.5	15.0	19.0	10.0	25.0	44	4.0	2.5
M30	3.5	25.5	52.0	46.0	44.5	19.0	24.0	12.0	31.0	56	4.0	2.5
M36	4.0	31.0	62.5	55.0	53.5	23.0	29.0	14.0	37.0	66	5.0	3.0

c) Post-installed anchor systems

This type of connection is installed in a hole that is drilled in hardened concrete. There are two main types: the mechanical expansion anchors (MEA) and bonded anchors, (Caltrans, 2011)

i. Mechanical Expansion Anchors (MEA)

These anchors are inserted in pre-drilled holes and are expanded to bear against the concrete surface and via any of the following techniques:

- Hammering the anchor (deformation controlled)
- Tightening a nut (torque controlled)
- Expanding into an undercut (notching)

The MEAs have the following advantages: they are inexpensive, are quick and easy to install, can be installed in any orientation and the loads can be applied immediately after installation. They however have a limited tension load carrying capacity and are unsuitable where vibratory loads are exhibited. The two common types of MEAs used include the following and are illustrated in *figure 2.12* respectively:

- the shell expansion anchor with internal threads (shell type) – requires independent stud, nut and washer and are stronger in tension
- The integral stud anchors (stud-type or wedge type and external plug) – is furnished with nuts and washers, is easy to install in a multi-hole base plate and is stronger in shear.

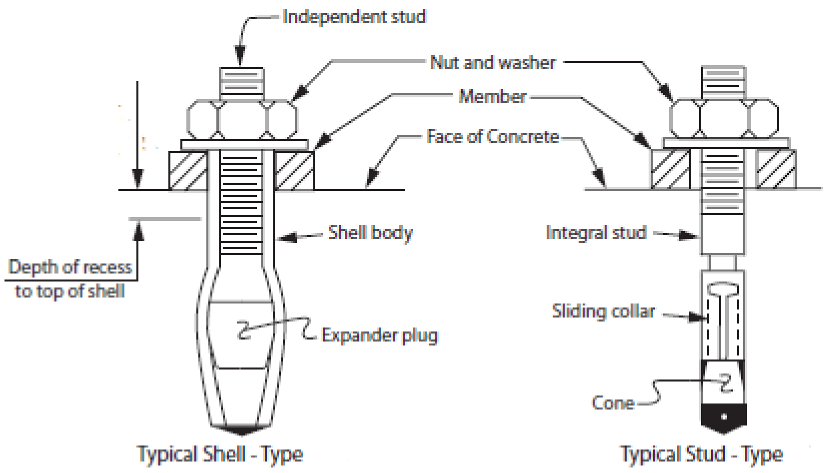


Figure 2.123: Typical shell and stud type post installed anchors, (Caltrans, 2011)

ii. Bonded Anchors

These are systems that use bonding agents such as cement paste (grout), epoxy or other chemical adhesives. The bonding agent is smeared on to the surfaces to be bonded or pressed into holes that receive the protruding edges. These systems provide simple, effective, economical and preferred bonds except where vibratory or seismic ductility is critical. This system is however not important for this research.

d) Selection of anchorage systems

The critical items to be considered in selection of the anchorage system include the following, (Cheok, 1998):

1. Base material in which the anchor or fastener will be installed.
2. Loads applied by the fixture or material to be fastened.
3. Anchor or fastener and bolts types.
4. Installation procedures including the method of drilling or the installation tool used.

5. Effects of corrosion.
6. Dimensions of the base material including the plate thickness, anchor or fastener spacing, and edge distance.

2.4.3 Tolerance

Tolerance represents the acceptable variations that can be expected with regard to size and position of components in a construction. It should not however cause impaired performance or appearance. In buildings, each stage construction is often adapted to the variations in the preceding stages and therefore the need for planning for tolerances at the design stage. It is nearly impossible to make two elements absolutely identical and the fact that two elements can be made to be interchangeable doesn't mean that they are identical rather that their variations are co-ordinated with the variations of the parts they fit in, (CMHC, 2004).

In assembly constructions, tolerances allows for ease of fitting and removal of components without extensive adjustment of connections, eliminates risks of damaging components in an attempt to force them into position and reduces chances of lose of alignment. Typical generally accepted standards in the absence of specifications are given in *table 2.6.*

Table 2.6: Typical tolerances for alignment of concrete members, (CMHC, 2004)

Dimension, mm	**Tolerance, +/-mm**
0 – 2400	5
2400 – 4800	8
4800 – 9600	12
9600 – 14400	20

14400 – 19200	30
19200 – 57600	50
Above 57600	Specified by designer

Whereas a tolerance of +/-5mm is recommended for concrete elements of lengths between 0-2400mm shown in *table 2.6*, this value seems large for the designs of elements developed in this research. The value can be reduced considerably for more controlled precasting environment.

Besides the tolerances for alignment above, the precast elements are manufactured with inherent tolerances in the specific components, position of lifting attachments and attachment of cast-in items or ferrules as illustrated in *table 2.7*, (MRT, 2012).

Table 2.74: Tolerances per 2000mm length of precast elements, (MRT, 2012)

Precast concrete element	Tolerance (mm)				
	Thickness of any section	Length or width	Internal dimensions	Straightness†	Squareness
Slabs and panels, including RSS wall panels	± 3	± 3	-	3	± 3 in 2000 mm
Pits, gullies and manholes	+ 5, - 3	± 5	± 5	-	± 5 in 2000 mm
Kerbs, channels and blocks	+ 5, - 3	± 5	-	5	± 3 in 2000 mm
Retaining walls	± 5	± 10	-	3	± 5 in 2000 mm
Traffic barriers	± 5	± 10	-	5	± 5 in 2000 mm
Piles	+ 10, - 5	± 20	-	10	-
Boat ramp planks	± 3	± 3	-	5	± 3 in 2000 mm
All other products	+ 5, - 3	± 5	± 5	3	± 5 in 2000 mm

† Deviation from a 1 metre long straight-edge except for piles the deviation is over the length of the pile.

2.4.4 Shrinkage

During the hardening process of concrete due to the effects of cement hydration and concrete drying process due to water loss, there is a reduction of the concrete volume.

Concrete shrinkage is a characteristic where the concrete changes its volume over time in a way that it decreases dimensions of an unloaded concrete structure proportionately in all directions. As a porous hard material, concrete starts deforming as soon as it is exposed to mechanical, thermal and hydrologic effects. Hydrologic effects as well as the thermal effects are a result of environmental condition (temperature and humidity) changes or changes in the concrete mass (heat of cement hydration and self-desiccation), (Isovic, 2012).

There are various types of shrinkage mechanisms namely:-

- Plastic shrinkage – which develops on the surface of fresh concrete exposed to drying
- Chemical shrinkage – which is consequence of chemical binding, that is using of water in concrete in the cement hydration process,
- Autogenous shrinkage – also called hydration shrinkage. It is closely connected to chemical shrinkage. It is a consequence of self-desiccation in pores of cement paste in the process of hydration of non-hydrated cement,
- Drying shrinkage – when the concrete is exposed to the environment which causes evaporation of water from capillaries,
- Thermal shrinkage – a result of temperature changes in concrete in the process of hydration,
- Carbonation shrinkage – where carbon dioxide from the environment causes chemical reactions in cement concrete

All these shrinkage mechanisms are bound to be exhibited. In precast constructions where the elements are manufactured in a factory setting with minimal variations in environmental conditions and where all processes are monitored, the effects are minimal. Alternatively, parameters that influence the shrinkage can be checked regularly for instance, for drying shrinkage, the parameters may include; thickness of elements, porosity or content of free water in concrete, paste volume, binder fineness, temperature and relative humidity, (Isovic, 2012). The use of SCC also reduces the total shrinkage as illustrated in the comparison of *figures 2.13* and *2.14* for shrinkage in ordinary concrete and high-strength concrete (SCC) respectively.

The investigation of shrinkage and its significance in precast concrete cannot be underestimated but is however not practically done in this research at this stage but an assumption is made based on literature that in SCC the total shrinkage is reduced.

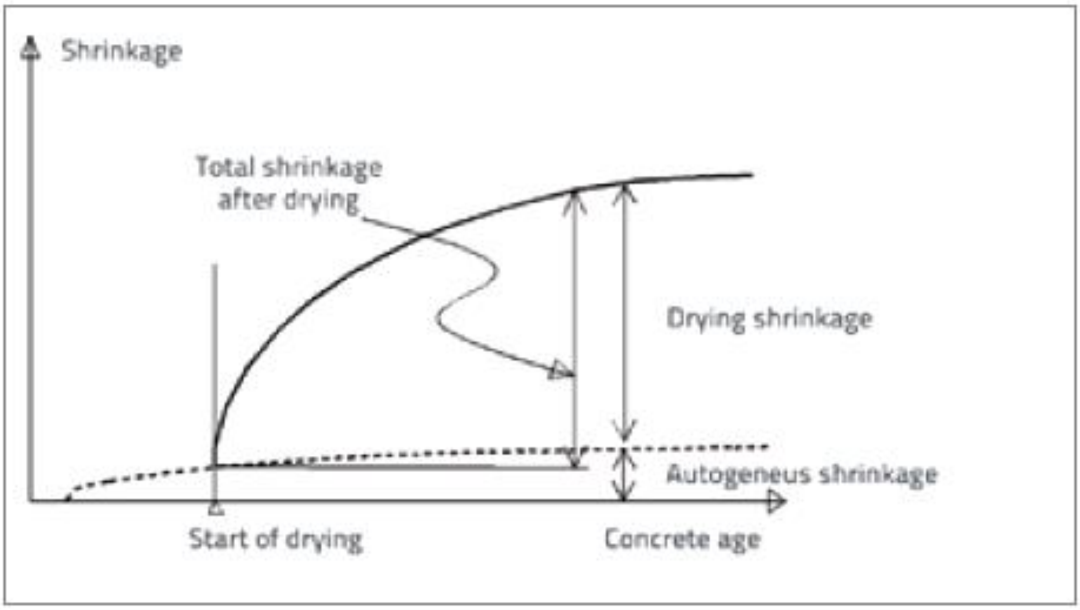


Figure 2.13: Shrinkage in ordinary concrete, (Isovic, 2012)

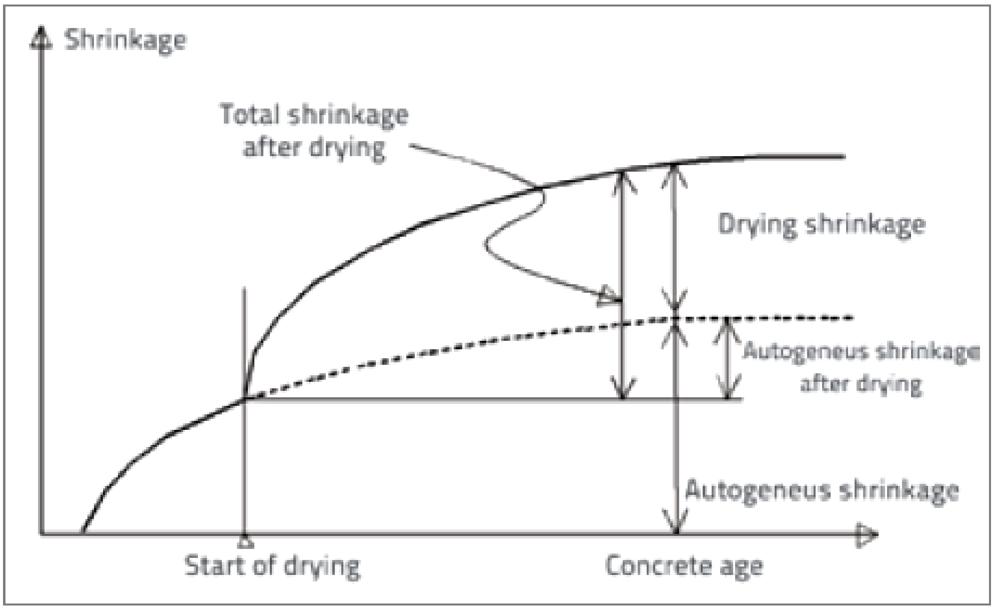


Figure 2.145: Shrinkage in High-Strength Concrete (SCC), (Isovic, 2012)

2.5 Monitoring and corrosion control

2.5.1 Monitoring of precast constructions

Monitoring of structures involves recording of time dependent parameters during certain periods. These parameters can be physical, mechanical, chemical or others which are usually present in a structure. Two levels of monitoring are usually done; material monitoring in which the behaviour of the materials are investigated and, structural monitoring in which the structural behaviour of the construction is investigated. The monitoring can be done at different phases of the life of the structure including the following, (Samuel Vurpillot, 2002) :

1. **At the construction stage**

At the construction stage, the monitoring tries to investigate the following:

- Whether or not the intended values are achieved and maintained,

- Whether defects (such as premature cracking) are exhibited, and their severity,
- Understand the real behaviour of the structure and hence estimate performance and remedial actions if necessary.

At this stage, the following information is sought:

- Strength parameters such as crushing strength,
- Hardening time of concrete to estimate when shrinkage stresses begin to be generated,
- Deformation measurements during early age of concrete to estimate the self-stressing and risk of cracking,
- Quality of prefabricated elements, and behaviour on first loading,
- Damage on prefabricated elements due to unusual loads such as thunderstorm and earthquakes,
- Ease of connection with regard to location and precision.

2. At refurbishing, strengthening or enlargement

Material degradation and/or damage are usually the reasons for refurbishing existing structures. Also, new functional needs (or enlargement) call for reconfiguration and at times strengthening. For prefabricated concrete elements, a procedure for removal, addition or replacement of components should be envisaged at the early stages of construction.

3. Monitoring during service

The service phase is the most important period of the life of a structure. During this phase, the construction materials are subject to degradation by aging. Concrete cracks and creeps and reinforcements oxidises. The degradation of materials is caused by mechanical (loads higher than the theoretically assumed) and physico-chemical factors (corrosion of steel, penetration of salts and chlorides into concrete, freezing of concrete) and as a result, the strength capacity, durability and safety of a structure decrease.

Monitoring during service provides information on structural behaviour under predicted and repeated loads, and also registers effect of unpredicted loads. The data are useful for damage detection, evaluation of safety and determination of residual capacity of structures. Early damage detections are important because they lead to appropriate and timely interventions. Late damage detection may lead to costly refurbishment or even dismantling of structures.

4. Monitoring during dismantling

When the structure or parts of the structure, or vital components in the structure do no longer respond to the required performances, and/or the cost of their replacement or reparation is excessively high, the ultimate life-span of the construction, the part or the component has been attained and should as such be dismantled, or replaced. Monitoring helps to detect necessity of dismantling or replacement of components and enabling the exercise to be done safely and successfully.

2.5.2 Corrosion control

Corrosion-induced deterioration of reinforced concrete elements (usually chloride-induced corrosion of reinforcement or steel components) in a structure occurs when the environmental loading on the structure is greater than the ability of the structure or elements to resist the environment (or environmental factors). One can either decrease the loading or increase the resistance or do a combination of both, (Virmani, 2000).

Corrosion can occur when the exposed steel components are in contact with the environment, or when the structure (or elements) is subject to freeze-thaw cycles, expansive reactions, excessive deflections or fatigue. These processes cause the concrete to crack which subsequently allows water and chlorides easy access to the interior of the concrete and get into contact with the steel reinforcements. Corrosion control methods are either mechanical or electrochemical.

Mechanical methods are physical barriers that prevent contact or delay the ingress of chlorides, oxygen, and moisture through the concrete cover to the reinforcement. They include admixtures, sealers and membranes, overlays, and coatings on the steel components. Sealers or overlays commonly used are: Portland cement concrete, low-slump dense concrete, latex-modified concrete, silica fume-modified concrete and polymer concrete. Coatings used are either organic or metallic: organic coatings include the non-metallic fusion-bonded epoxy coatings while the metallic coatings include materials such as nickel, stainless steel and zinc.

Four general categories for corrosion control are given as: design, concrete, corrosion inhibitors and steel/reinforcement type, (Virmani, 2000).

a) Design: while designing for corrosion control, attention is paid on; concrete cover, maximum allowable crack widths in service, reinforcement distribution and exposure and designing rigid overlays (such as silica fume concrete, latex-modified concrete, dense concrete and polymer concrete).
b) Concrete: properties of the concrete considered include; water-cement ratios, pozzolans (silica fume, fly ash, slag), latex, epoxy and polymer admixtures, cement type and aggregated gradation.
c) Corrosion inhibitors include organic, inorganic and combinations

d) Steel/reinforcement considered may include; epoxy-coated, galvanized, nickel-clad, copper-clad, stainless steel-clad, stainless steel or non-metallic steel/reinforcement bars.

2.6 Infill Walls/Panels Systems

Panels assembled to fit frameworks of beams and columns/posts tend to restrain lateral in-plane displacements. They act as shear walls whether they are a full wall or partial, which is one method of stabilizing assembled frames, *figure 2.15*. The other method is introduction of a diagonal member to the structure so that the structure cannot translate as in a 'parallelogram'. Alternatively a rigid frame so suffices (Schodek, 2008).

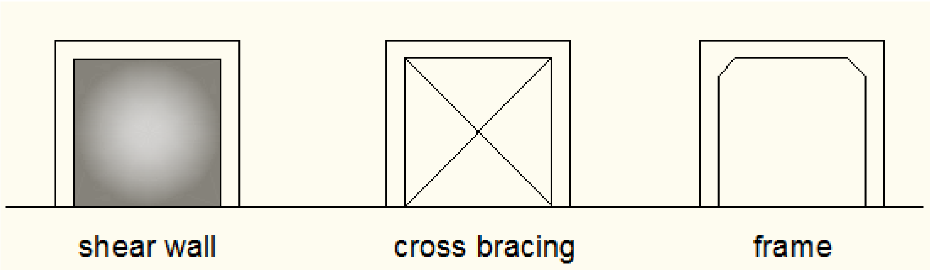


Figure 2.156: Illustration of the three methods of stabilizing assembled frames

The research on development of the assembled cube, (Sugut V. & Kipkemoi N., 2011) applies the use of panels as shear walls given that they are attached so as to fit in the framework of beams and columns, *figure 2.16* to provide restraint against in-plane lateral deformation.

Figure 2.167: Picture of assembled concrete cube (Sugut V. & Kipkemoi N., 2011)

CHAPTER 3: DEVELOPMENT OF PRECAST ELEMENTS

3.1 Introduction

This chapter describes the process of dimensioning the elements, the components and the pilot house. It also describes the procedures followed in fabricating the various steel components in the steel workshop and to manufacture the precast elements in the concrete laboratory. Steel works entail development of the steel moulds for the elements and the elements connection details. The works mainly involve plate and steel rods cutting, drilling, welding and grinding to give the desired shapes and finishes. Concrete works on the other hand include preparation of the steel moulds for concrete placing, positioning of reinforcements and inserts in the moulds, concrete proportioning, mixing, placing and curing of the precast elements.

3.2 Geometrical design of elements

The dimensions of the basic elements, the connection components and the pilot house adhere to the 4D system proposed by Hendrickx and Vanwalleghem as discussed in *chapter 2* and a reference made to the preliminary researches. From a generated sequence such as the one below, dimensions are either picked from the sequence or are obtained by adding and/or subtracting values in the sequence:

1 – 2 – 4 – 8 – 16 – 32 – 64 – 128 – 256 – 512 – 1024 – 2048 – 4096 – ... mm

3.2.1 Elements/components dimensions derivation

i. Bolt sizes: Diameter: 2 + 8 = 10mm

Length: 8 + 32 = 40mm

ii. Studs sizes: Diameter: 2 + 8 = 10mm

Length: 8 + 32 = 40mm

iii. Slab panel anchor plates: Thickness: 1 + 2 =3mm *figures 3.6*

Square sides: 16 + 64 = 80mm

Welded rods length: 16 + 64 = 80mm

iv. Endplates: Thickness: 1 + 2 =3mm *figures 3.7*

Square sides: 16 + 64 = 80mm

Tongue length: 4 + 32 + 64 = 100mm

v. Beams/columns: Square Cross-section: 16 + 64 = 80mm *figures 3.3* and *3.4*

Length: 1024mm

vi. Slab panels: Thickness: 8 + 16 = 24mm *figures 3.1* and *3.2*

Width: 512mm

Length: 1024mm

vii. Connector cube: Square sides: 16 + 64 = 80mm *figure 3.5*

viii. Connections: Drill holes on endplates/anchor plates: 4 + 8= 12mm

Connection points on slab panels: intervals of 256mm *figure 3.1*

Connection points on beams/columns: intervals of 256mm *figure 3.3*

Illustrations:

a) Slab panels

Figure 3.1 shows the top view of the slab panel showing the dimension details. The slab panel being 1024mm*512mm with connection points at two short edges and one long edge at 256mm apart and 128mm from the edges. The embedded steel anchor plate is 80mm*80mm and has a 12mm bolt hole as illustrated. The 3D visualization of the slab panel thickness of 24mm is on *figure 3.2*.

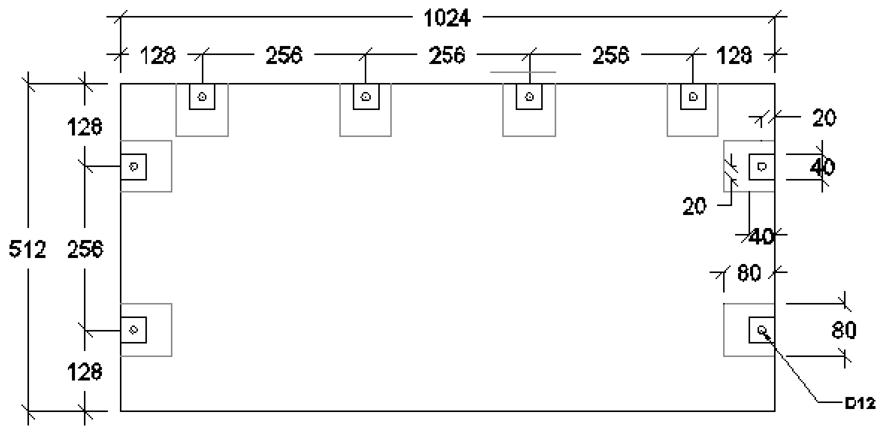


Figure 3.1: Illustration of the slab panel dimensions: length, width and anchorage details

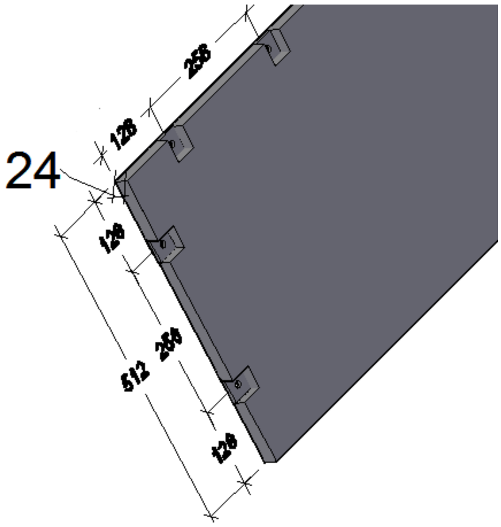


Figure 3.28: Illustration of slab panel dimensions: thickness

b) Beams/columns

Figure 3.3 illustrates the 3D visualization of the beams/columns showing the dimensions being square cross-sections of 80mm*80mm and a length of 1024mm. The connection points are provided at 128mm from one end and 256mm centre to centre. The end details are zoomed-in in the figure. Further details and sectional visualization are on *figure 3.4*.

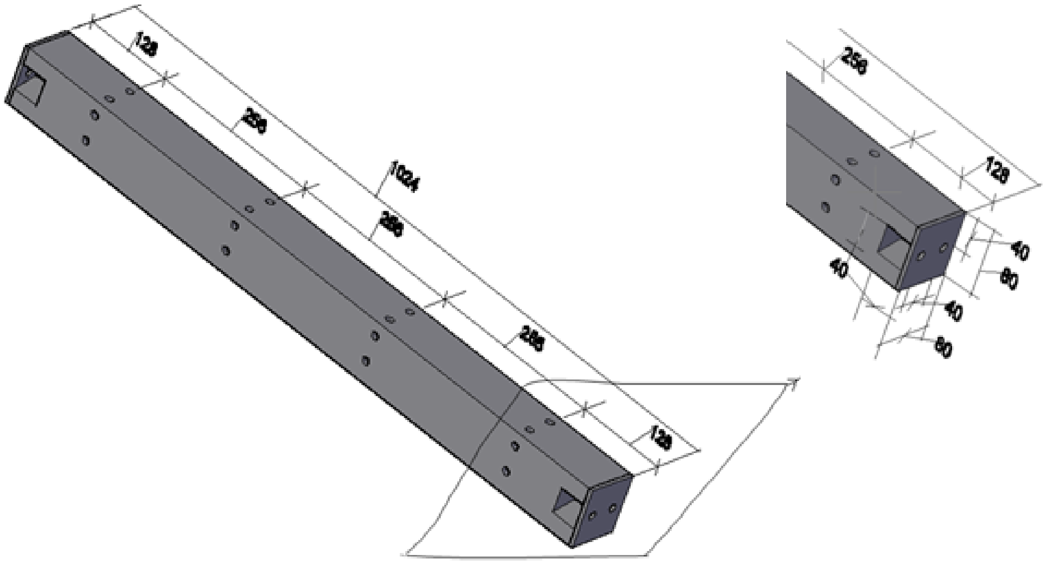


Figure 3.3: Beams/columns dimension details, (zoomed section alongside)

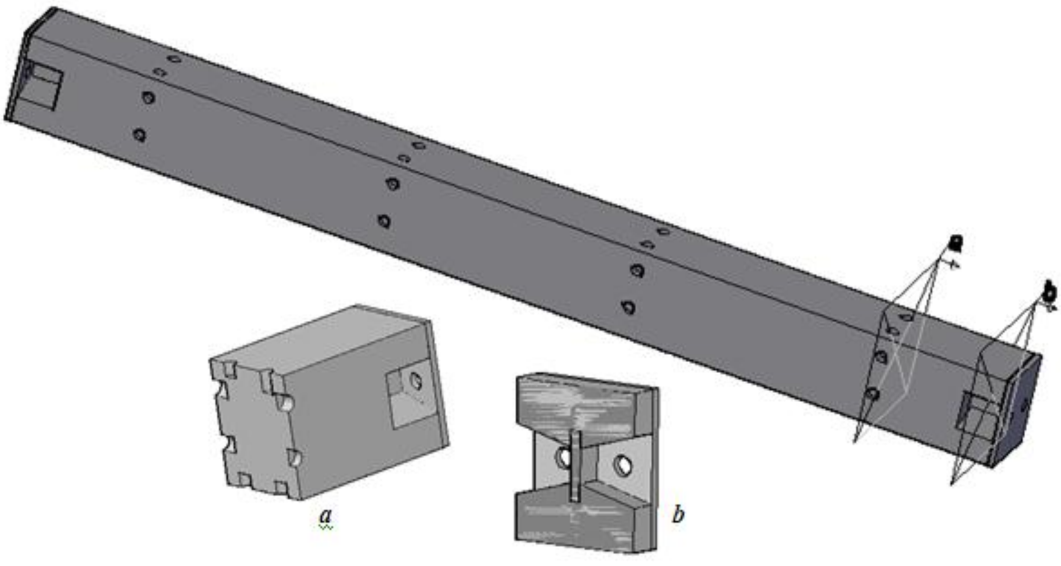


Figure 3.49: Illustration of beams/columns sections a and b

c) Connector cube

A 3D visualization of the connector cube is shown in *figure 3.5*. The dimension details are as illustrated. The 10mm diameter bolt hole is provided by the in-cast nuts.

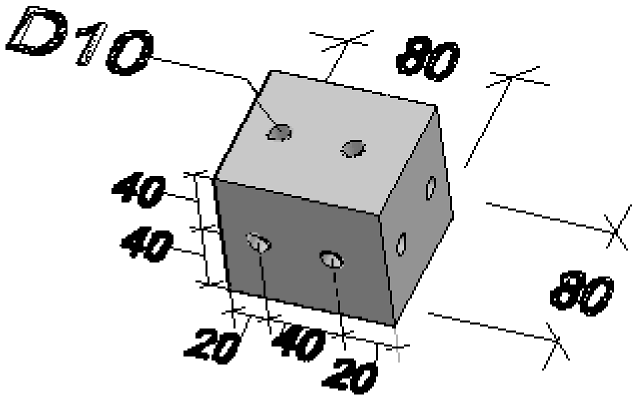


Figure 3.510: Connector cube dimensions

3.3 Fabrication of components in the steel workshop

Steel components are fabricated in the steel workshops and include the following; slab panel anchor plates, beams/columns end plates, studs, hooked nuts, nuts with studs and the steel moulds including slab panel moulds, beams/columns moulds and the connector cube moulds.

3.3.1 Slab panel anchor plates

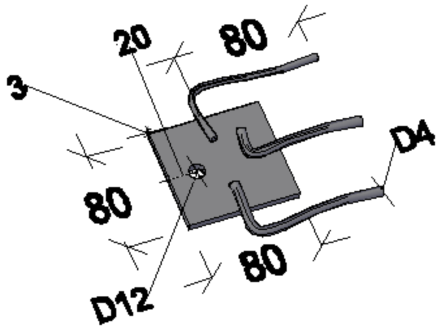


Figure 3.611: Illustration of the slab panel anchor plate and dimensions

The anchor plates are made from 3mm thick steel plates of size 80mm square. These plates have two main features: a bolt hole for anchoring slab panel by bolting and welded hook rods for attaching these plates onto concrete. The bolt hole is 12mm in diameter while the hooks are steel rods of 4mm in diameter and 80mm in length. The rods are welded onto the plates and bent specially so that when placed in the mould to firmly anchor into the concrete, *figure 3.6*.

3.3.2 Beams/columns endplates

The endplates are made from 3mm steel plates with the plate at the end being square of sides 80mm. They have two features; two bolt holes for connection to the connector cubes and a tongue-like hooking plate to attach the plate into the concrete, *figure 3.7*. The bolt holes are 12mm in diameter and are designed so as to coincide with studs emanating from the connector cubes, that is, the measurements are replicated. The hooking plate is 40mm in width, 100mm in length and 3mm in thickness with the shorter end welded onto the endplate as illustrated. The other end is cleaved and curled in the manner sown on the figure so as to provide firm attachment of the endplates to the concrete.

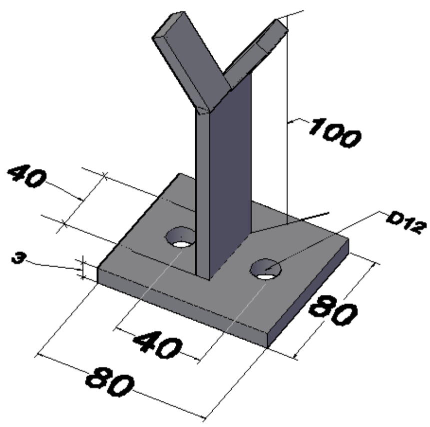


Figure 3.7: Illustration of the beam/column endplate and dimensions

3.3.3 Studs, hooked nuts and nuts with studs

THE STUDS

The studs are threaded rods, *figure 3.8*, that allow fastening of nuts from either ends. They are used in the connection of beams and columns onto the connector cubes since the connector cube has a provision for bolting and the beams/columns endplates provide for fastening nuts from the other end. The studs are 40mm in length and 10 mm in diameter.

Figure 3.8: The studs

HOOKED NUTS

These are nuts with steel rods welded onto them to appear winged-like so as to anchor into concrete. They have two steel rods of 3mm in diameter and 40mm in length welded onto two of the nut edges and extending to the rear as in the illustration in *figure 3.9*. These modified nuts are cast-in concrete to enable studs to be fastened on the faces of the elements for further attachment of other elements as discussed in *chapter 3.5*.

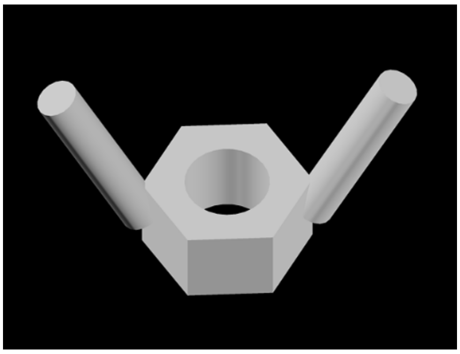


Figure 3.9: Illustration of the hooked nuts (Nuts with hooks welded on sides)

NUTS WITH STUDS

Figure 3.10: Modified nuts with studs for slab panel assembly

These are studs used in the anchorage of slab panels onto the framework of beams and columns during assembly. They are made by welding a nut onto the stud: one edge of the nut is welded onto one end of the stud in the manner illustrated in *figure 3.10*. During assembly, the studs are first screwed into the nuts provided on the face of the columns/beams and then the panels are attached so that the bolt hole provided on the anchor plate of the panel coincide with the nut welded on the stud to allow fastening of a bolt to attach the slab panel to the beams/columns framework.

3.3.4 Steel moulds

Moulds define the shape and dimensions of the precast elements being cast and as such need to be accurately fabricated with regard to dimensions, precise provisions for positioning of the attachments or inserts and should be stable in shape during use. Two assumptions are made in the design of moulds:

- The choise of using of 3mm thick steel plate for fabrication of moulds is not a result of design but rather the consideration that the sizes of elements manufactured and the manual nature of the mould assembly cannot subject the plate to undue deformation or damage. Stiffening of edges and the bases of moulds is also done to stiffen and to maintain the shape. The capacity of the steel workshop to conveniently work on thicker plates is also limited.

- The provisions for tolerance as discussed in *chapter 2.4.3* which would require about 2mm – 5mm allowance on mould dimensions is not provided for in this pilot stage of the research so as to assess the challenges associated with lack of fit and make reccommendations for the subsequent stages of this research.

The steel moulds developed include: connector cube mould, slab panel mould and beam/column mould. Two moulds are developed for each ofthese elements and note that the beams and columns elements are made from the same kind of mould.

5. Connector cube mould

Illustration of the initial process of mould development, that is, dimensioning and drawing, is shown the *figure 3.11* for the connector cube element of dimensions 80mm*80mm*80mm, a 3D visualization of the mould in *figure 3.12*, and a ready developed mould in *figure 3.13*.

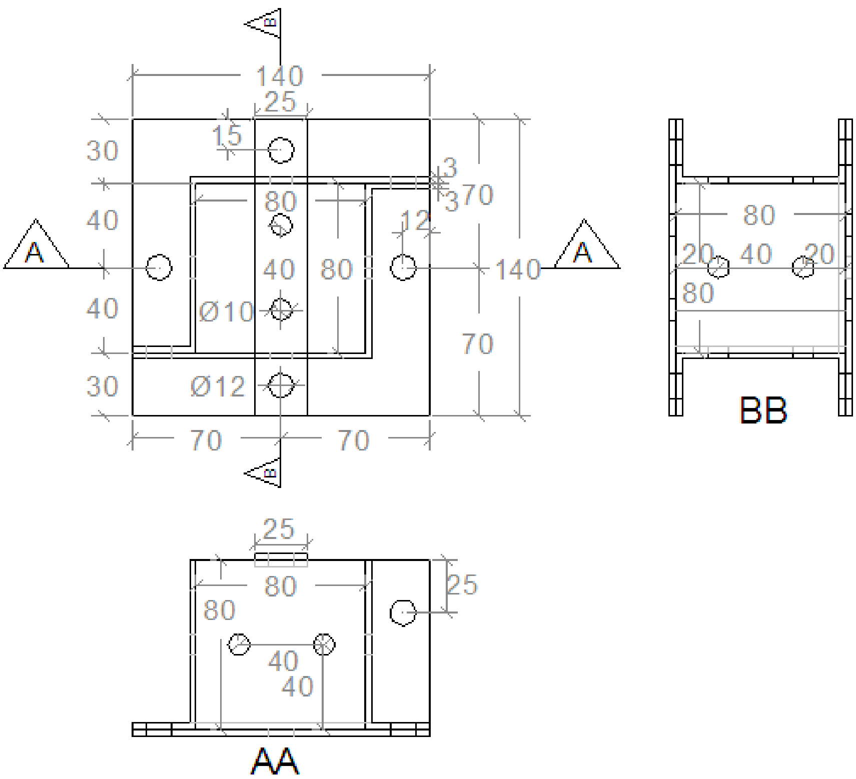


Figure 3.1112: Dimensions of the connector cube mould

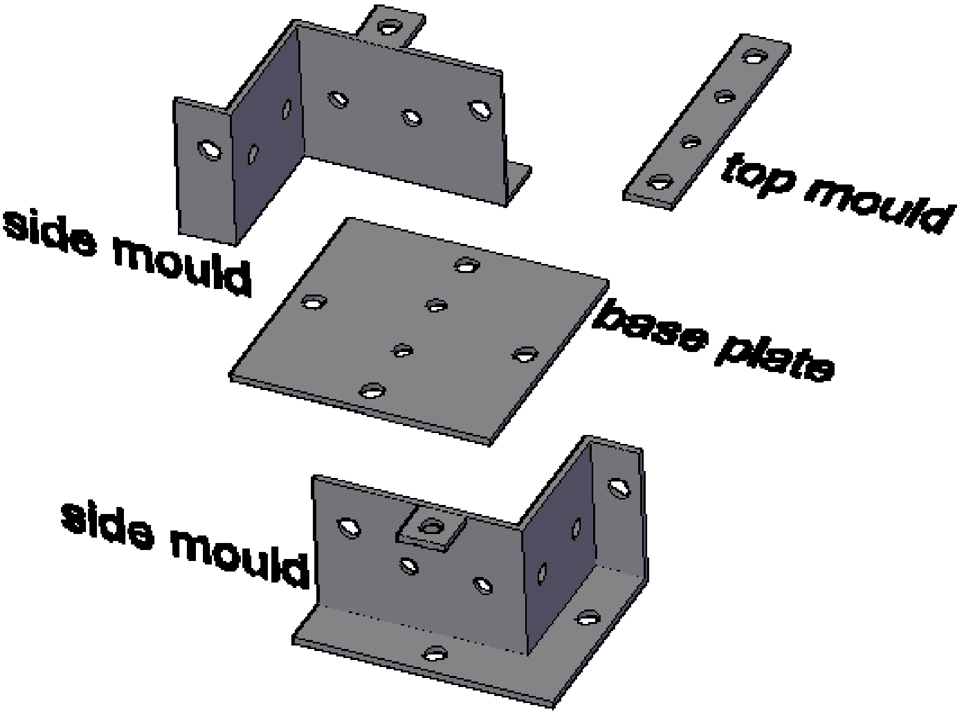


Figure 3.1213: 3D visualization of the connector cube mould

Figure 3.13: Picture of connector cube mould

6. Slab panel mould

The slab panel mould dimensions are illustrated in *figure 3.14*. It is made from steel plate forming the base and angle-line fabricated to form the side moulds, *figure 3.15*. The side moulds are held onto the base plate by bolting via the 12mm diameter bolt-hole provisions as illustrated: two provisions at each short edge and three provisions at the long edges. The anchor plates are held in position by bolting via the 10mm diameter bolt-hole provisions.

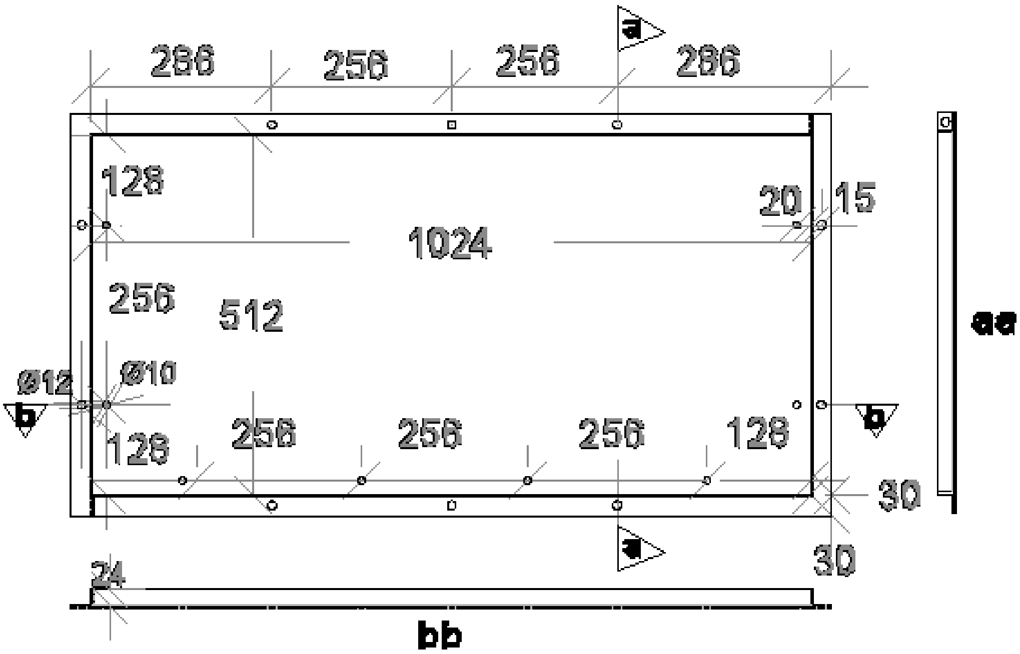


Figure 3.1414: Slab panel mould dimensions

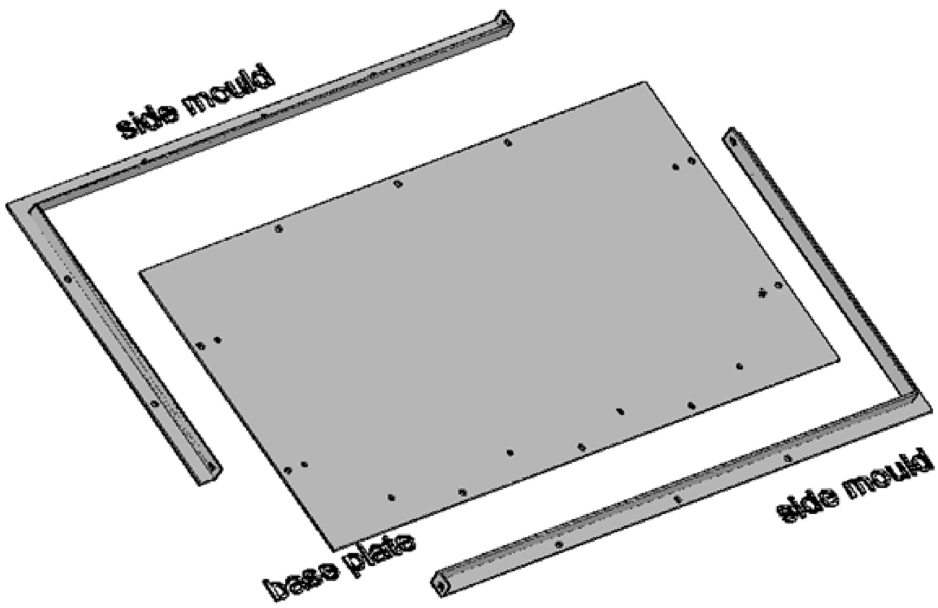


Figure 3.15: 3D visualization of the slab panel mould

7. Beams/columns moulds

Beams/columns moulds are fabricated in the like manner as the slab panels and connector cube moulds. The special features are the provisions for attaching the nut with hooks made on the sides moulds, top and base plates in form of 10mm bolt-holes. The endplates are also fastened at the inside-ends of the mould via bolt holes provided as shown in *figures 3.16* and *3.17* and more clearly in the 3D visualization in *figure 3.18*.

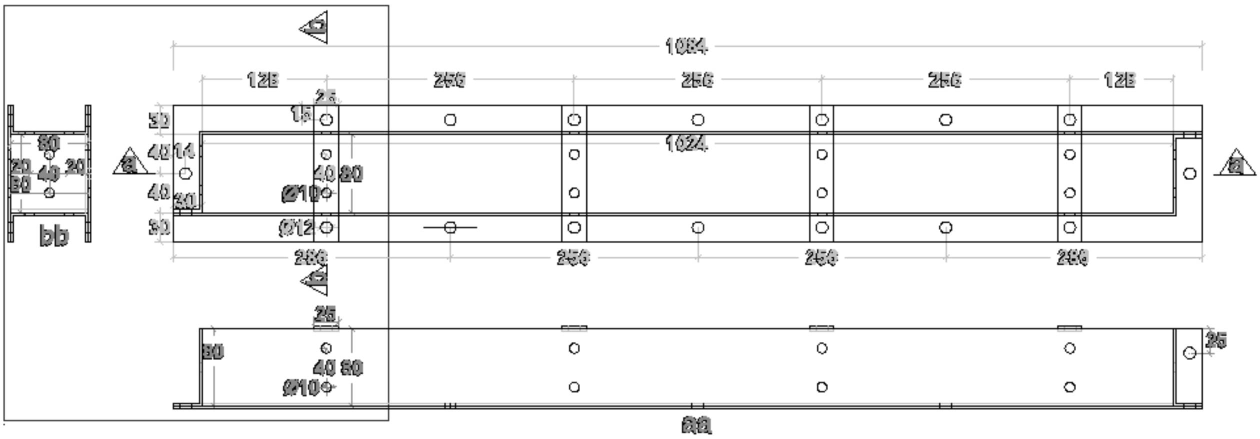


Figure 3.16: Dimensions of the beams/columns mould (the highlighted part is zoomed in figure 3.17)

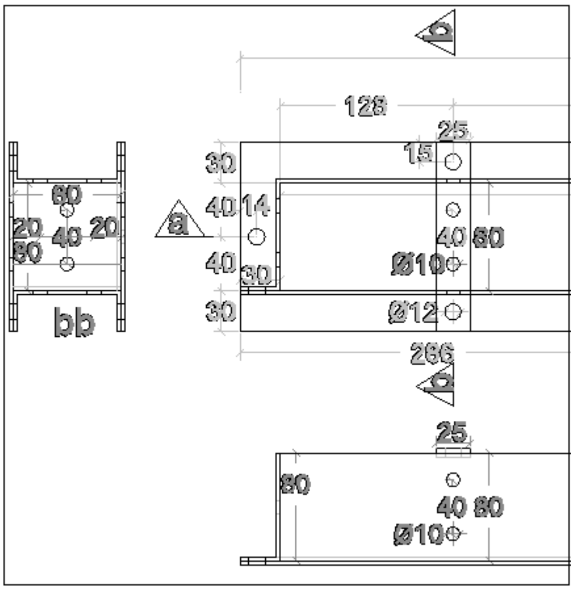


Figure 3.1715: Zoomed part of figure 3.16

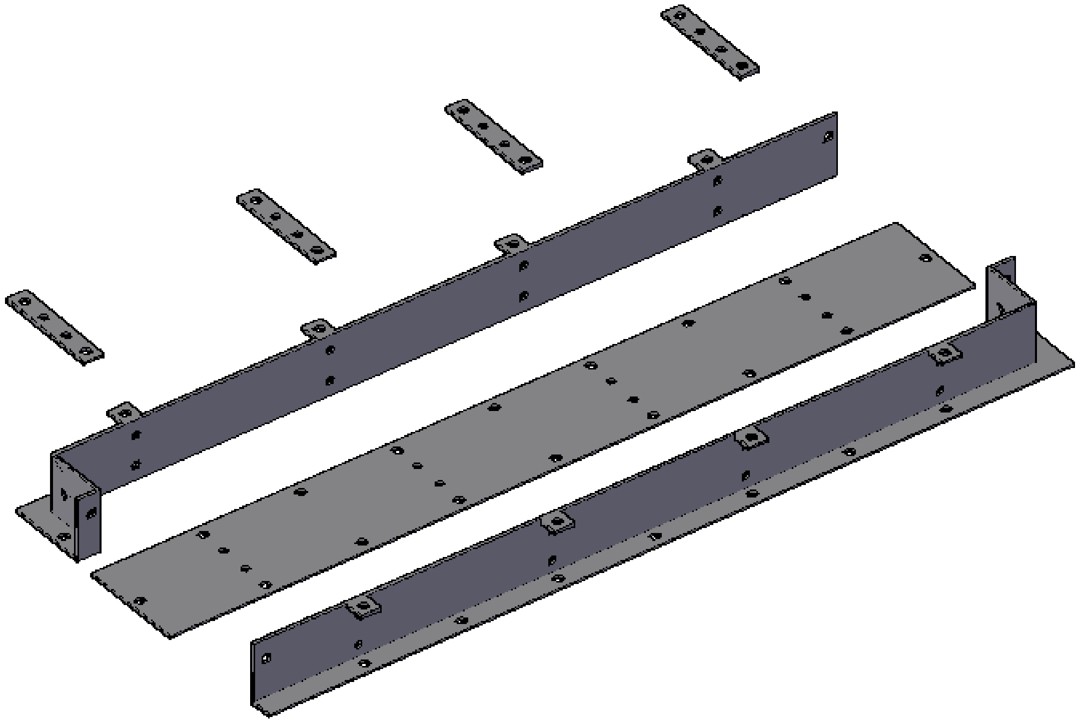


Figure 3.18: 3D visualization of the beams/columns mould

3.4 Pilot House dimensions and the assembly procedure

3.4.1 Dimensions of the pilot house

The dimensions of the pilot house starts from the consideration that the smallest useful space/room in a building, such as a toilet, could be 1000mm*1000mm, which in the dimensioning system adopted translates to 1024mm*1024mm. This can be propagated to a basic housing unit of 4096mm*4096mm, proposed by Okumu in (Okumu, 2010), and further developed to a typical one-bedroom dwelling (with kitchen, living room and toilet) of about $4.8x10^7mm^2$ ($=48m^2$) in floor area or about 6000mm*8000mm in dimensions. This translates to 6144mm*8192mm, or an area of about $5.0x10^7mm^2$ ($=50m^2$) in the dimension system. Several partition configurations are possible as illustrated in the *figure 3.19* (*a-f*). In the figure, *a* represents a floor plan without partitions, *b* represents a mesh of possible partition configurations, *c*, *d*, *e* and *f* represent various other configurations.

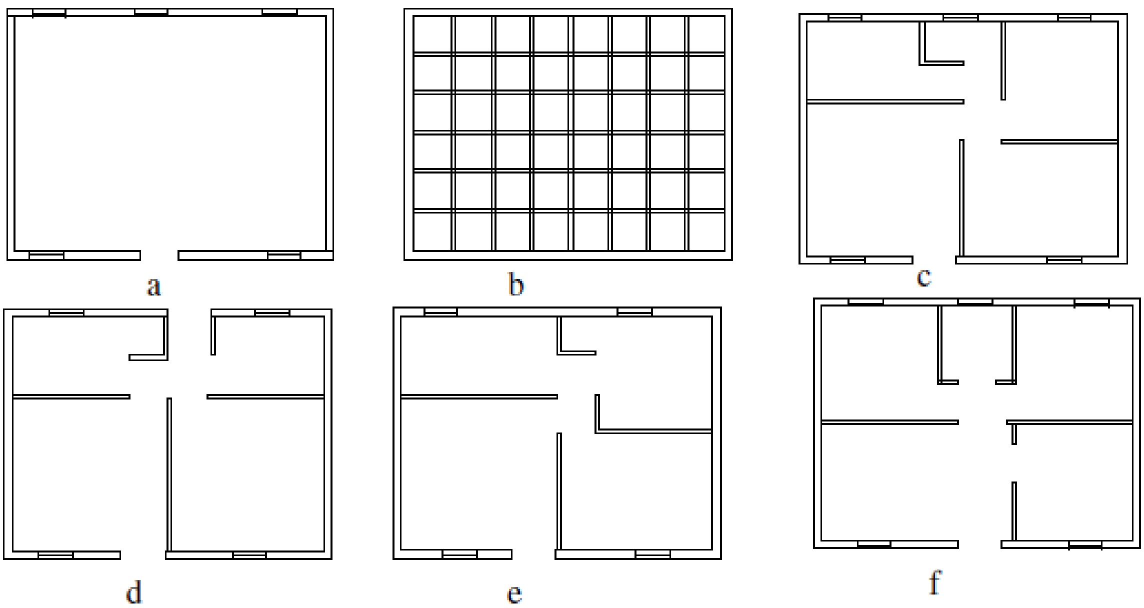


Figure 3.19: Illustrations – a to f – of the configuration development (not to scale)

In this research a floor plan of 2288mm*4496mm, with an area of about $1.0\text{x}10^7\text{mm}^2$ and a height of 2128mm to the lintel level is designed and constructed. This is presumably one of the four rooms proposed in *figure 3.19 (c-f)*. The dimensions are derived by adding the lengths of elements that constitute the respective span, that is, two slab panels and three columns for the shorter side, four slab panels and five columns for the longer side and two columns and a connector cube for the height, *figure 3.20*. Since the elements dimensions are derived from the dimension system, the results of addition of these dimensions also obey the dimension system.

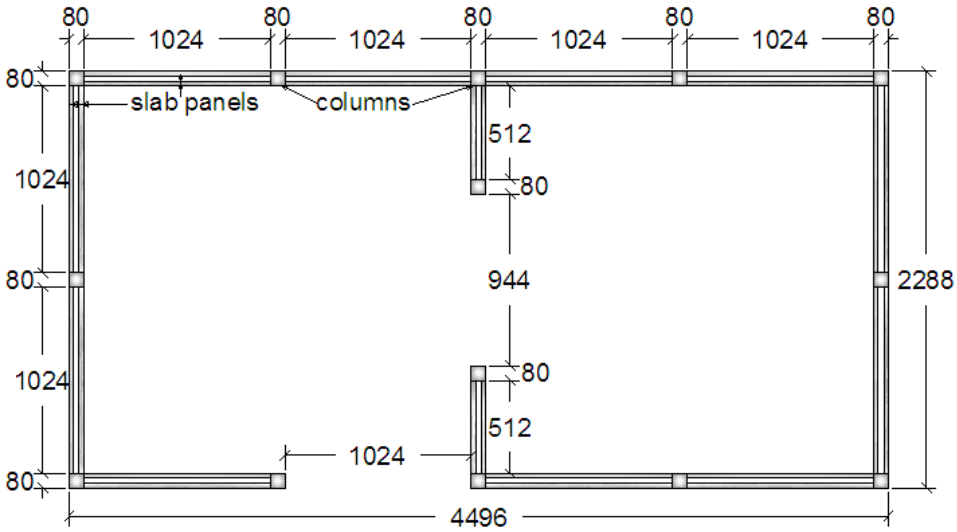


Figure 3.20: Illustration of the plan dimensions of the pilot house (units is mm)

3.4.2 Architectural view of the proposed pilot house

The architectural drawing of the proposed pilot house is shown in *figure 3.21*,

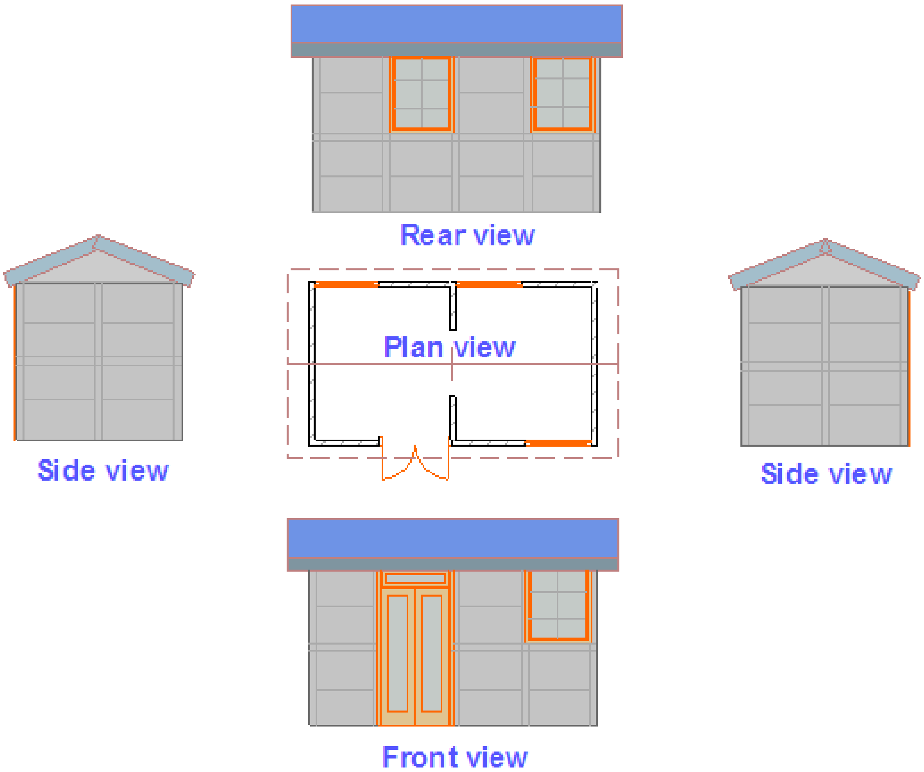


Figure 3.2116: Architectural drawing of the proposed model house (not scaled refer to figure 3.20)

3.4.3 The assembly procedure

The procedure of assembly of elements follows the steps 1-6 demonstrated in *figure 3.22*. Further elaboration and actual assembly is in *chapter 4.* The steps are as follows:

i. Drilling and dropping the holding down post-installed bolts visualized as *step 1*
ii. Position the columns so as to fasten nuts via provisions on the end plate *step 2*
iii. Prepare the connector cube by fastening studs and drop it into the column top end, *step 3*
iv. Insert beam to attach connector cube by dragging horizontally, and fasten nuts, *step 4*
v. The nuts-with-studs are screwed onto columns and beams to attach the slab panels, *steps 5* and *6*
vi. The procedure is then repeated for the entire construction.

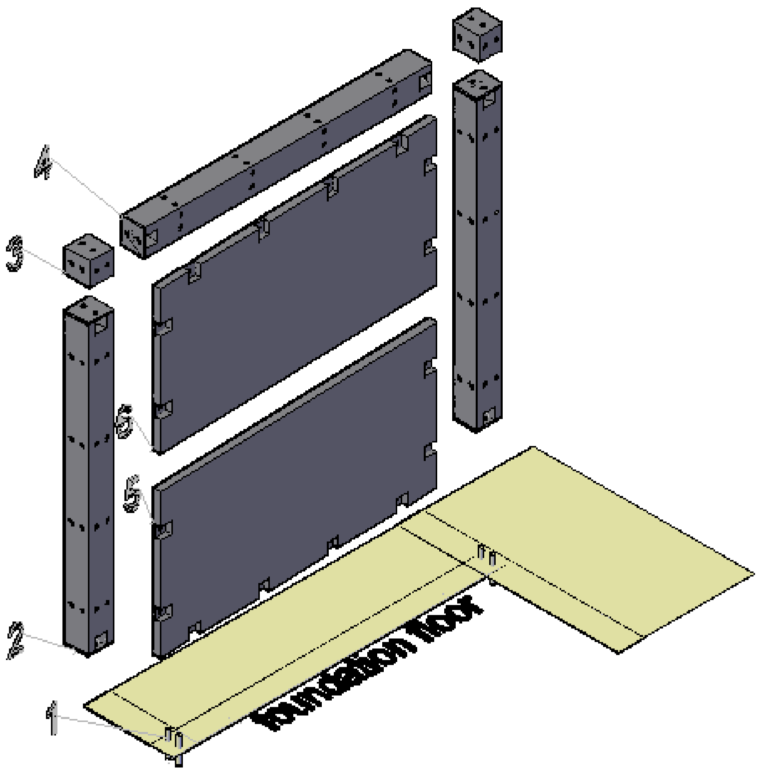


Figure 3.2217: Demonstration of the assembly process

3.5 Concrete Works

For the construction of the pilot house, the following are the number of precast elements required:

a) 40 beams/columns elements

b) 84 slab panels elements

c) 14 connector cube elements

Fabrication of the precast concrete elements is done in the concrete laboratory where the concrete materials are prepared. The preparation for precasting involves;-

I. Tools and equipment preparation – weighing machine, concrete mixer, and hand tools including spades and trowels, and concrete test equipment for slump flow and cube strength tests

II. Mould preparation – mould assembly and positioning of reinforcements, inserts and attachments,

III. Concrete materials proportioning and placing – coarse and fine aggregates, cement, lime, water and super-plasticizer,

IV. Demoulding exercise, and

V. Water pond for curing elements

3.5.1 Concrete materials and proportioning

The Self-compacting concrete (SCC) used in this research is made from a proportionate mix of half-inch coarse aggregates, river sand, cement (32.5 N/mm^2), lime filler, water and admixture (super-plasticizer – Glenium 110). This has been applied in preceding work as discussed in *chapter 2*.

The concrete used is class 20, hence the use of 32.5N/mm^2 cement and the adoption of the mix proportions which attained target strength of 30.6N/mm^2 as given in *table 3.1*. The densities given on the table are the typically accepted values for the materials.

Table 3.1: Mix Design for SCC (chapter 2)

MATERIAL	**VOLUME (per m^3)**	**DENSITIES (Kg/m^3)**
$\frac{1}{2}$ inch coarse aggregate	0.35	2500
Fine aggregate	0.26	2200
Super plasticizer	0.0041	1000
Filler (lime)	0.0412	384
Water	0.2070	1000
Cement	0.1377	3240
Steel	-	7700

3.5.2 Preparation of moulds for casting concrete

Mould preparation involves positioning of the inserts, reinforcements and attachments and firmly bolting up the assembly in readiness for pouring concrete. This to a far extent determines the ease of demoulding and assembly of the elements since the precision of positioning of the endplates and inserts is reflected in the ease of the actual construction of the pilot house.

The different elements and their moulds require different attachments, inserts and reinforcement as described as follows:

a) Connector cubes mould preparation

The attachments placed in the connector cube moulds are the hooked nuts. Twelve hooked nuts (two on each face of the cube) are attached to the interior of the mould and held in position via studs as illustrated in *figure 3.23*. The side moulds and the top moulds are then assembled onto the base plate with the hooks allowed to inter-twine as shown in *figure 3.24.*

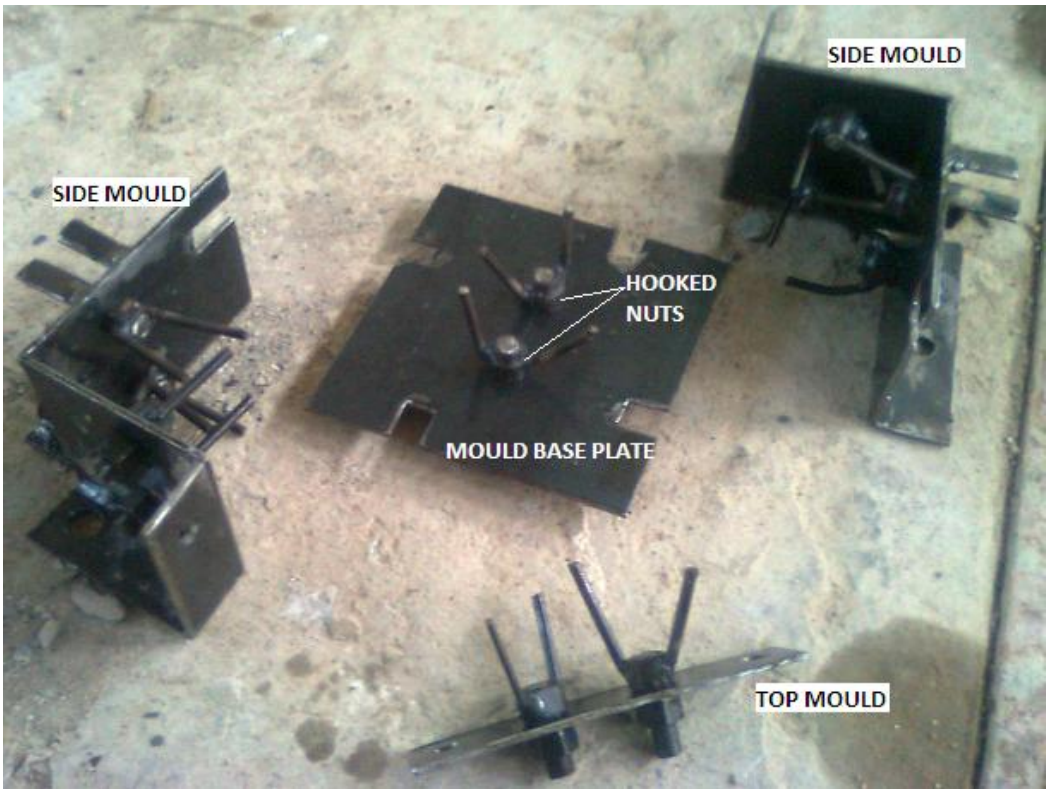


Figure 3.23: Illustration of connector cube mould assembly; side mould, top mould base plate and hooked nuts

Figure 3.2418: Top view of assembled connector cube mould

b) Slab panels mould preparation

The attachments placed in the slab panel moulds include: the reinforcement mesh, wooden spacers and anchor plates. The wooden spacers are 40mm*40mm at the bottom and 30mm*35mm at the top with three slanting edges, *figure 3.25*, and a bolt hole of 10mm in diameter at the centre. This wooden spacer has two purposes: one, to lift the anchor plate to flush with the top surface of the slab panel as shown in *figure 3.26*, and two, to leave a cavity in the concrete once it is removed.

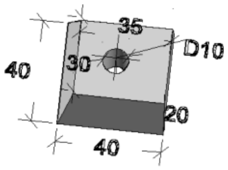


Figure 3.25: Wooden spacers for the slab panels

Figure 3.2619: Description of components of the slab panel mould assembly

a) Beams/columns mould preparation

The attachments placed in the beams/columns moulds include; the reinforcement bars, inserts, hooked nuts and endplates. The inserts provide a void at the ends of the elements for inserting and fastening nuts. They are cut out of *Styrofoam* material to the shape of a wedge as shown in *figure 3.27*. The choice of the material is guided by the fact that the depth of the insert being about 40mm does not permit easy withdrawal and hence the preferred material is that which can be crushed out easily upon hardening of concrete. The hooked nuts are placed in a similar manner as demonstrated earlier in *figure 3.23* for the connector cube.

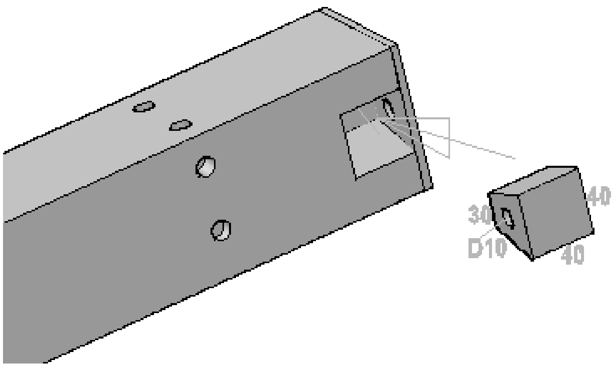


Figure 3.2720: Illustration of a Styrofoam insert for the beams/columns

3.5.3 Concrete placing

SCC requires short mixing and placing times owing to the fact that it is a quick setting concrete type. This also enables quicker demoulding and hence the repetitive use of the moulds is hastened. The process followed in the introduction of materials into the mixer is summarised as follow:

i. The mixer is prepared and cleaned and proportioned materials placed within reach,

ii. Powder material, that is, the sand, cement and lime, were first dry mixed to uniformity,

iii. Water mixed with super-plasticizer was then added while mixer was running,

iv. The paste was allowed two minutes to uniformly form,

v. Coarse aggregates were then introduced and the mixture allowed to mix adequately to uniform consistency,

vi. The ready mix was then poured into the prepared moulds.

Concrete was placed by pouring from the mixer directly into the moulds or by aid of a scooping device (such as a spade). In the mould the concrete flow uniformly to cover reinforcements and inserts/attachments and where necessary slight assistance to the flow was done by use of a steel rod. A trowel was then used to ensure surface is smooth. *Figure 3.28* shows a freshly placed slab panel and the surface smoothened.

Figure 3.28: Freshly placed slab panel elements

3.5.4 Demoulding and curing

The concrete in the mould was allowed about 24 hours to set and gain strength before the moulds were removed. The maturity method discussed in *chapter 2* permits removal of elements from the moulds upon attaining about 8N/mm^2 strengths. This was however not ascertained since 1-day concrete strength was not measured but rather a trial and error method was applied in addition to the experience from the previous researches which showed that 1-day old elements could be handled without undue damage. For the beams/columns and slab panels, a further 12 hours was allowed before the inserts (Styrofoam and wooden spacers respectively) were removed. The elements were cured for at least 7 days by dipping in a pond of water and thereafter watered in the air for a further 21 days. The elements were then stacked and stored awaiting construction, *figures 3.29* and *3.30.*

Figure 3.2921: Slab panels transported and stacked

Figure 3.30: Beams/columns and the connector cubes stacked

3.5.5 Concrete tests

Two tests were done, the slump flow test on fresh concrete and cube crushing strength on the hardened concrete.

1. **Slump flow**

Slump flow test was done on fresh SCC to measure the plastic properties or the horizontal free flow. It was done using the cone and base setup. The cone was placed on the base, filled completely with the concrete without consolidation, and then the cone was lifted. The spread of the concrete was measured. For SCC the spread can measure between 455mm and 810mm, (Ouchi, 2003). The observations made on this single test can include:

i. Flow – by measuring the spread
ii. Resistance to segregation – by observation
iii. Bleeding – by observation
iv. Air popping – by observation
v. Slump flow time of mix – by measuring the time taken to spread

Observations:

The test was done as a check to the materials and procedure of mixing and was found to be satisfactory with an average flow of 650mm. Other observations on air entrainment and bleeding were notably acceptable which was further evidenced by the quality of the precast elements made and the smooth finishes on the surfaces. An average spreading time of 7 seconds was measured. This compares well with the allowable time of between 3.5 – 14 seconds for slump flows of 600mm – 750mm. In the actual placing of mix in moulds where reinforcements are present a slight assistance of flow to remote corners of mould and to completely cover of reinforcement is done.

2. Cube crushing strength

Crushing strength test is done on hardened concrete at 7 days and 28 days after casting to give an idea of the strength gain over the time of hardening. During placing of concrete, part of the mix was poured onto six prepared standard test cubes of dimensions 150mm*150mm*150mm alongside the elements being manufactured. These cubes were allowed 24 hours in the moulds after which time they were demoulded and placed in a water bath to harden for the prescribed period of test, that is, three test cubes for 7 days and the other three test cubes for 28 days before they were removed from the water bath, and crushed in a compressive strength test machine. The results are listed in *table 3.2*.

Observations:

Table 3.2: Test cubes crushing strengths

Test days	Cube No.	Cube Cross-sectional Area (mm^2)	Crushing force (kN)	Crushing Strength (N/mm^2)	Average Crushing Strength (N/mm^2)
7 Days	4D001	22500	440	19.56	18.81
	4D002	22500	430	19.11	
	4D003	22500	400	17.78	
28 Days	4D004	22500	680	30.22	29.78
	4D005	22500	670	29.78	
	4D006	22500	660	29.33	

From the results *table 3.2*, the mean cube strengths are observed to have average strengths of 18.81N/mm^2 at 7 days and 29.78N/mm^2 at 28 days. The characteristic cube strength of the concrete can therefore be determined as the mean 28-day cube strength of 29.78N/mm^2 less 8.2 (assumed deviation factor) giving a value of 21.58N/mm^2. This is considered to be satisfactory for the SCC mix of class 20. For purposes of this research, the manufactured elements are not tested for strength but the cube strength test results are taken to reflect the properties of the concrete and hence the elements.

CHAPTER 4: CONSTRUCTION OF THE PILOT HOUSE

4.1 Introduction

This chapter brings into focus the actual steps and procedures that have been used to construct the pilot house. The steps are simple as is the objective of the research to allow easy adoption and implementation by non-skilled and semi-skilled workers. Three steps are followed namely:-

a) Construction and preparation of the foundation,

b) Assembly of the concrete elements; done in two construction stages.

c) Assembly of the roof, windows/doors and finishing.

4.2 The Assembly Process

4.2.1 Construction of the foundation and the floor slab

This research adopted a typical foundation in which a concrete slab, *figure 4.1*, is laid on a strip foundation and a sub-base of compacted hardcore and murram. This is done because of two considerations: the construction technique is also suitable for partitioning already existing structures in which a foundation and floor exists and the fact that the focus for this research is on the superstructure and as such any suitable foundation should suffice.

The strip foundation is made by digging a trench of 600mm in depth and 300mm in width to a stable base. Murram is compacted on this base up to 150mm in thickness and a 100mm thick blinding of concrete is poured. Two masonry block courses are placed along the strip. The base of the slab is excavated to remove top soil; hardcore stones arranged and compacted with murram to the level of the masonry blocks.

By aid of constructed formwork, ordinary concrete is poured throughout the slab, to a depth of 200mm. The concrete mix ratio used is the ratio for hand mixing given as ratio 1:3:5 for cement, sand, coarse aggregates respectively. A finish of 30mm thick mortar is then applied.

Figure 4.122: Image of the finished floor

4.2.2 Anchorage of the columns to the foundation

Post-installed anchors are used to firmly affix the columns onto the floor. For this purpose expansion bolts of grade 8.8 are used. They are bolts with sleeves and wedged nuts as shown in *figure 4.2*. The procedure of installation is as follows:-

i. Drill a bolt hole into the concrete that just fits the anchor bolt with its sleeves when loosened,
ii. Drop the anchor bolt into the bolt-hole allowing it to protrude sufficiently to give room for fastening,
iii. Insert the column via the end plates onto the protruding end of bolt, *figure 4.4*,
iv. Fasten a nut via the provision on the column end to force the sleeves to spread outwards and firmly fix the column onto the floor base, *figures 4.3* and *4.4*.

This system requires that the anchorage locations from the model drawings be carefully mapped out on the foundation and the correct drilling bit be used to drill into the concrete.

Figure 4.2: Image of the expansion anchor bolt

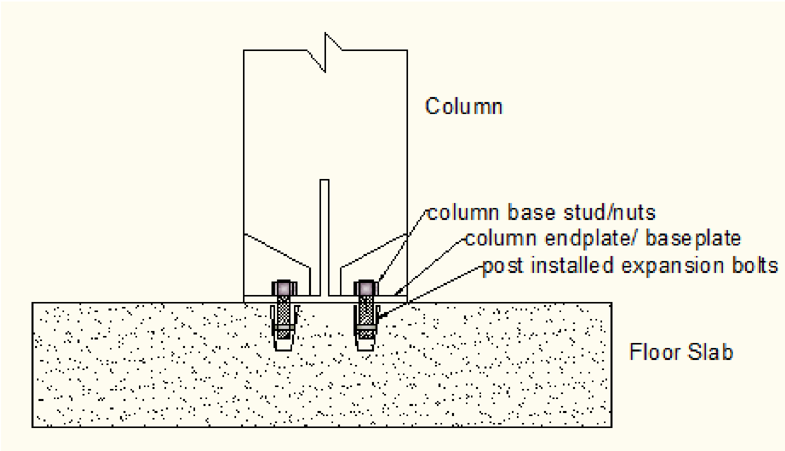


Figure 4.323: Sketch of the system of anchorage of column to floor slab

Figure 4.424: Anchor bolts dropped into bolt-hole for fixing column, and column already fixed

4.2.3 Assembly of the framework

The framework is made of the beams and the columns assembled by the connection system discussed above. The vertical elements are the columns and the horizontal elements are beams.

The process of assembly is summarised below:

i. Consecutive columns are first fixed onto the floor as described in the previous section, and illustrated in *figure 4.4,*

ii. Connector cubes – for the top of each column – are prepared by fastening studs on the required faces, as in *figure 4.5*,

iii. The prepared connector cubes are dropped on the respective column ends and nuts fastened via the provisions on the elements,

iv. Beams are then placed to be between each column and connected via the connector cubes as illustrated in *figure 4.6* and *figure 4.7*,

v. The process can be done for the entire framework at the construction stage before slab panels are mounted or alternatively slab panels can be mounted on each completed framework as illustrated in *figure 4.7*. The latter case of fixing slab panels in every complete framework before another frame work is constructed helps to build early lateral stiffness and hence preferred.

Figure 4.525: Connector cube with studs screwed for connection of beams/columns

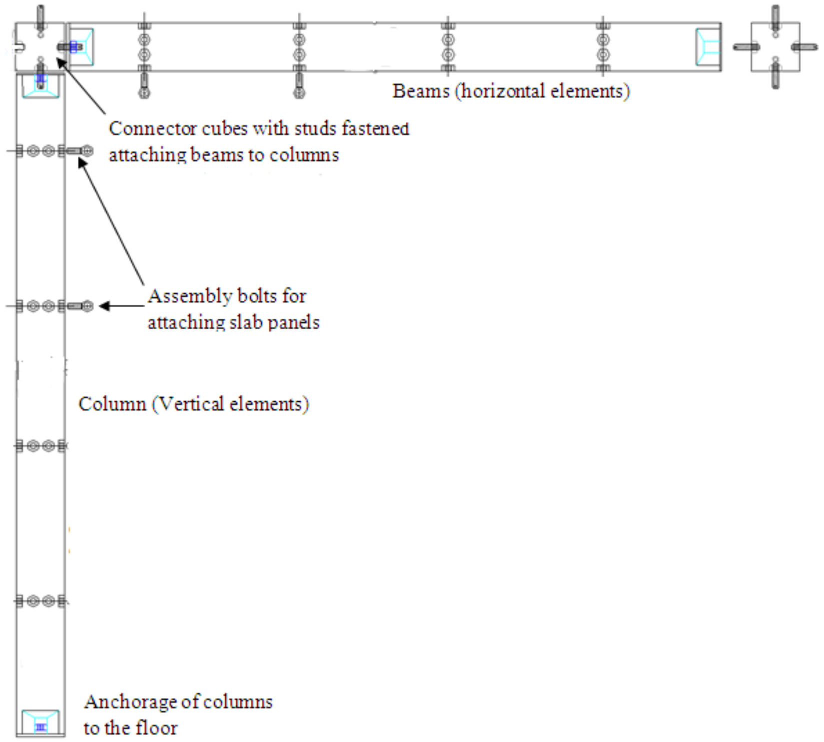


Figure 4.626: Illustration of the Assembly process of the framework (not to scale)

Figure 4.727: Picture of the development of the framework of beams and columns

4.2.4 Assembly of the slab panels

Slab panel assembly has partly been described in the preceding discussion. A system of bolting has been developed that uses a modified stud with a welded nut, *figure 4.8.* This is important such that the threaded end of this bolt is first screwed into the nuts provided on the faces of the columns allowing the nut end to protrude and appropriately position so that once slab panels are mounted the nut coincides with the bolting hole on the anchor plates of the slab panels to allow bolts to be fastened and as such affix the slab panels onto the framework; this has also been demonstrated earlier in *figure 4.7.*

Figure 4.828: Modified studs with welded nuts for slab panel assembly

4.3 The Construction Stages

4.3.1 Construction stage 1

Figure 4.9 elaborates the process of mounting slab panels and the assembly around corners. The assembly of elements to this level, which is the window level amounts to construction stage 1. The illustration also demonstrates how the interior corner of the structure is developed.

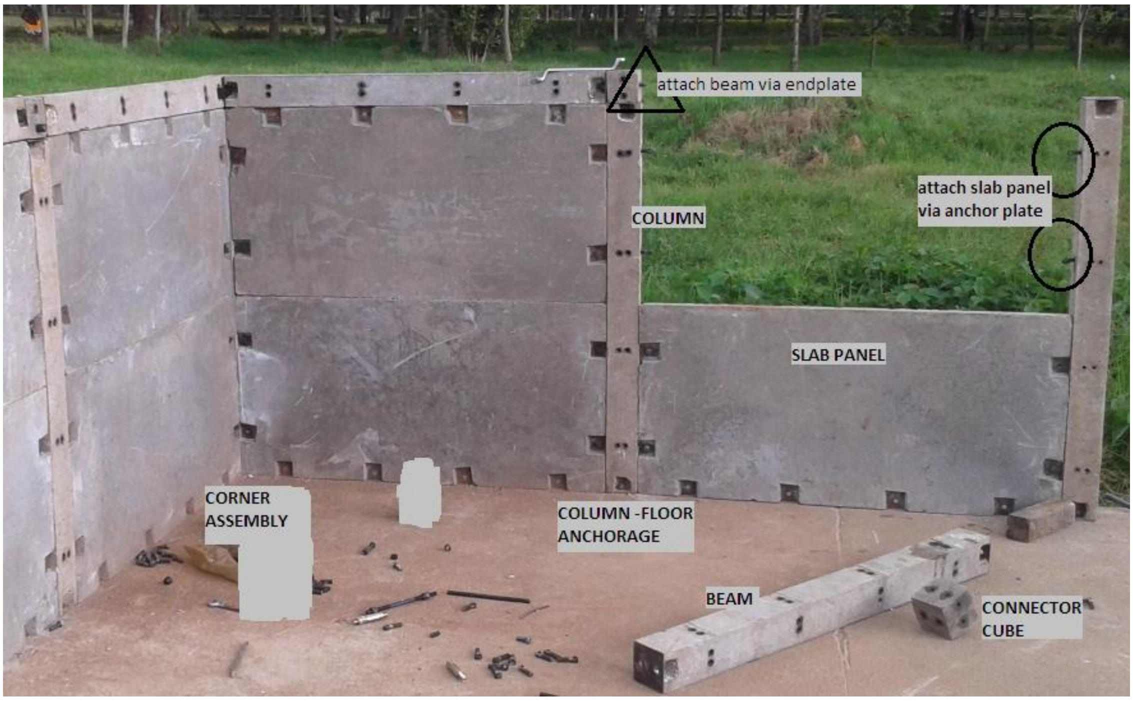


Figure 4.929: Pictorial description of elements in the assembly

Double slab panels are assembled, that is, one panel is attached from the inside and another from the outside, *figures 4.10 and 4.11*. If required an insulating material can be included between the two slab panels. This space can alternatively be used for passing building services such as network cables, electricity conduits, water pipes among others, provided prior arrangements are made to ensure services are placed as the construction progresses to minimize removal and replacing of elements for ease of assembly.

Figure 4.10: Construction stage 1 in progress, single panel assembled from the inside

Figure 4.1130: Construction stage 1 is completed

4.3.2 Construction stage 2

This stage comprises of the assembly of the second column piece and the elements to the lintel level and the roof. This includes the provisions made for the windows and the doors. The assembly process is similar to the phase one case except that columns emanate from the connector cubes at the window level as opposed to emanating from the floor. The column is attached to the connector cube in the same manner as for the beams, *figure 4.12.*

Figure 4.12: Construction stage 2 begins

Two steel square frames are prefabricated from rectangular hollow sections and used as the lintel beam, to tie all the columns round the structure. They have the provision for bolting via column ends and are also useful to seat the roofing trusses. The process is illustrated in *figure 4.13* and *figure 4.14*. Steel is preferred for the lintel beam for ease of connecting the roof trusses. The lintel beam as well as the steel roof trusses are not designed under the 4D systems as explained in *chapter 1*.

Figure 4.1331: Construction stage 2 in progress

Figure 4.14: Construction stage 2 completed with openings for doors and windows

4.3.3 Observations made during the construction stages

1. **Assembly of elements**

The following are some observations made during the process of assembly of elements at the construction stages 1 and 2:

1. The elements are shaped up by the moulds from which they are made: There are cases where an error may occur during assembly of the mould and if not corrected leads to loss of shape of the element made thus. While this is uncommon and any such element is rejected due to unconformity, there are some which end up being used such as the connector cube in *figure 4.15* highlighted in blue and labelled ‘c’. Another case of a beam or column seemingly bowed due to deformity in the mould creates or enhances gaps

between slab panels and columns such as in *figure 4.15,* highlighted in yellow and labelled ‘a’.

2. The position of the columns at the construction stage 1 must be done with very high precision since it determines the ease of assembly of the other elements including those of construction stage 2. To do this, a sketch is drawn on the floor and the location of the column anchor bolts mapped out correctly.
3. Fitting of the slab panels between the framework of beams and columns at times necessitated unscrewing of beams to create room for slab panels to be fixed. This could be an indication that the beams, columns and the connector cubes may have undergone shrinkage as discussed in *chapter 2*. In fact, *figure 4.15* pictures a scenario highlighted in red and labelled ‘b’ of a slab panel tightly fitting which is an anomaly.

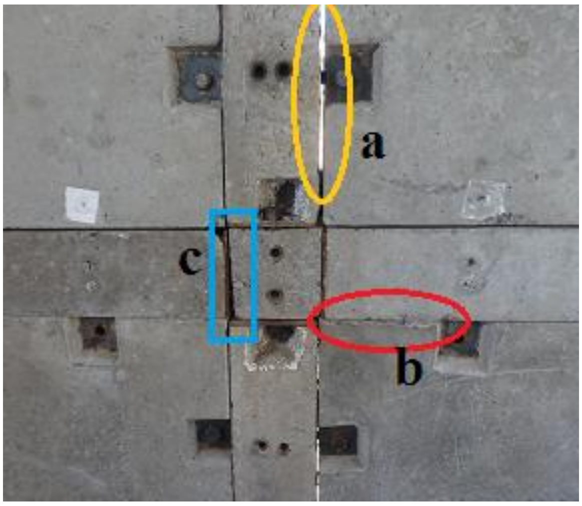


Figure 4.15: Observations made during assembly of elements

2. **Corrosion control**

The steel components exposed to the environment are prone to corrosion and they include the beams/columns endplates, the slab panel anchor plates and the bolt and nuts systems. Corrosion control is provided for two different scenarios; first, the exposed steel plates which are either covered against exposure by lean concrete as done for this research or alternatively cladding of oil-based paint (as done for roof trusses) or galvanizing the steel plates can be done. The latter case of galvanizing the components is recommended for future work. Second, the bolts and nuts system are oiled during assembly and this should be done during every such process of fastening the bolts or nuts.

3. Monitoring of construction

The process of monitoring the precast construction began upon manufacture of the elements. This process included observation of the elements to detect any cracks or deformations that may arise during manufacture or due to handling of the elements. During actual construction of the pilot house, further monitoring was done detect any cracks or deformation that may occur at first loading of the elements, or when tightening connection. There were no adverse irregularities observed besides what is discussed in*1* above.

Monitoring during service of the pilot house is recommended. This may include inspection of the structure regularly to observe the effects of long term loading, effect of the environment (corrosion and the effectiveness of the control measures undertaken) and during disassembly of the structure (or part of it) and also to detect any unprecedented changes on the construction for consideration during future application of the technology.

4.3.4 Roofing and fittings

The roofing is made from ridge type trusses and corrugated pre-coloured blue iron sheets covering. The process of erection of the roof trusses and mounting the covering is a standard method using bolts and nuts and J-hooks for the iron sheets, *figures 4.16* and *4.17*.

The fittings include three windows and one exterior door. All fittings are prefabricated in steel and adopt dimensions of the precast elements including the mode of connection. The windows and door have steel plates welded at connection points to act as anchor plates and have bolt holes through which studs protruding from columns are inserted and nuts fastened to affix, a method similar to attaching the slab panels as discussed in *chapter 4.2.4*.

Figure 4.1632: Roof trusses mounted and window frames fitted

Figure 4.1733: Roofing work in progress

4.3.5 Building services

Services are conventionally provided for in buildings mainly by concealing them within the concrete, in precast concrete however, provisions are be made during prefabrication. The services common in buildings include electric cabling and plumbing. Other services may include communication cabling and heating systems.

There are several alternatives to provide for such services in the precast constructions. Among them are: developing fixings and adhesive systems or casting a thin layer of softer material on the inside face of the panels or else incorporate knock-out panels. In this research, surface adhesives and passing services in the gap between the double panels are envisaged. Electrical wiring was done and the main method adopted is using adhesive contact glue to attach the trunking and bolting up sockets on provisions made on the columns as illustrated in the picture, *figure 4.18.*

Figure 4.1834: Electrical services incorporated on the surfaces

4.4 Conclusion: Completed model house

The activities in the *chapters 3* and *4* culminate into the constructed pilot house illustrated in *figure 4.19.* This indicates a successful completion of the assembly of the structure which is done outside the *Huisman Laboratories* in Moi University, Eldoret.

Figure 4.1935: Completed Pilot House

CHAPTER 5: STRUCTURAL ANALYSIS OF THE ASSEMBLED MODEL

5.1 Introduction

The analysis of the model is done in two steps:

I. Checking by hand calculation of the basic elements,

II. Analysing the structure as an assembled model: the analysis is done for the two construction stages

5.2 Hand calculation checks

5.2.1 Design codes

Calculations are based on the following codes:

- *BS 6399 Part 1, Loading for Buildings: Code of Practice for Dead and Imposed Load.*
- *BS 6399 Part 2, Loading for Buildings: Code of Practice for Wind loads.*
- *BS 8110 Part 1, Structural Use of Concrete: Code of Practice for Design and Construction*
- *BS 8110 Part 3, Design Charts for Singly and Doubly Reinforced Beams and Rectangular Columns*
- *BS 5950 Part 1, Structural Use of Steelwork in Buildings: Code of Practice for Design of Hot-Rolled and Welded Sections*

5.2.2 Elements dimensions

The element dimensions for design is as follows:

Columns and the beams: length 1024mm, and cross section of 80mm*80mm

Connector cube: cubic of size 80mm*80mm*80mm

Slab panel dimensions: length 1024mm, width 512mm and thickness 24mm

Structure being designed has a floor area of 4496mm*3392mm and a height of 2128mm to the lintel level unlike the prototype constructed being 2248mm*2248mm and a height of 2128mm. This was advised by preliminary design similar to *section 5.3.1*.

5.2.3 Loads

An assumption is made that horizontal loads due to wind acts on slab panels in the respective directions and vertical loads including roof loads are taken up by the columns.

Loads on the construction will be considered in different points of view including:

I. On beams: the self-weight of the beams

II. On columns: roof loads, self-weights of the slab panels, connector cubes and beams and self-weight of the column themselves

III. Wind load on the structure acting on the slab panels

5.2.4 Material Properties

Concrete strength: SCC of class 20N/mm^2

Density of concrete: 2400Kg/m^3

Steel reinforcement: mean yield strength of 460N/mm^2

(Determined by multiplying the characteristic strength of steel, about 400N/mm^2 by the partial factor of safety of 1.15)

5.2.5 Weights of elements for design

The weight of elements is determined by both calculation as detailed in *appendix 1*, and by measurement of weights of the elements. *Table 5.1* summarises the results. The difference in the calculated and measured weights may be attributed to the fact that the volume of concrete does not exclude the volume taken up by the embedded steel and that the effect of steel removed during drilling and shaping components is not factored in.

Table 5.1: Weight of elements

Element	**Weight as calculated**	**Weight as measured**
Beams/columns	166.76N	170N
Slab panels	320.52N	290N
Connector cubes	13.54N	10N

5.2.6 Design of the roof elements

The trusses are made of steel and designed according to the BS, using *BS5950* and *BS6399* for loading. The ULS load case is considered with three load combinations:

Load combination 1: 1.4*Dead load + 1.6*Live loads

Load combination 2: 1.2*(Dead load + Live load + Wind load)

Load combination 3: 1.2*Dead load + 1.4*Wind load

Permanent/dead loads;

Iron sheet 0.1kN/m^2

Ceiling 0.1kN/m^2

Purlins 0.2kN/m

Live/Imposed loads: 0.75kN/m^2

Load combination 1 is used in the design and yields a roof load of 7.4kN on the columns carrying the roof structure. This load (multiplied by a factor of safety of 1.5) is added onto the columns during analysis at the second construction stage as discussed in *appendi*ces *2* and *3*.

5.2.7 Reinforcements design for beams and columns

The concrete cover for both the columns and the beams is considered to be a minimum of 15mm since the maximum aggregated size is 12.5mm. *Table 5.2*, gives the basis that the concrete cover should be at least equal to the maximum size of aggregate in the mix. Besides, the concrete type used, that is, the self-compacting concrete attains more superior bonding and durability properties than normal concrete with respect to covering reinforcement, air entrainment and actual strength.

Table 5.2: Nominal cover to reinforcement: Table 3.3 of BS 8110

Conditions of exposure (see 3.3.4)	Nominal cover mm				
Mild	25	20	20[a]	20[a]	20[a]
Moderate	-	35	30	25	20
Severe	-	-	40	30	25
Very severe	-	-	50[b]	40[b]	30
Most severe	-	-	-	-	50
Abrasive	-	-	-	See NOTE 3	See NOTE 3
Maximum free water/cement ratio	0.65	0.60	0.55	0.50	0.45
Minimum cement content	275	300	325	350	400

(kg/m^3)					
Lowest grade of concrete	C30	C35	C40	C45	C50
NOTE 1 This table relates to normal-weight aggregates of 20mm nominal diameter size. Adjustments to cement minimum contents for aggregates other than 20mm nominal maximum size are detailed in BS 5528-1: 1997. NOTE 2 Use of sulfate resisting cement conforming to BS 4027. These cements have lower resistance to chloride ion migration. If they are used in reinforced concrete in very severe or most severe exposure conditions, the cover in table 3.3 should be increased by 10mm. NOTE 3 cover should not be less than the nominal value corresponding to the relevant environmental category plus any allowance for loss of cover due to abrasion.					
[a] These covers may be reduced to 15mm provided that the nominal maximum size of aggregate does not exceed 15mm. [b] Where concrete is subject to freezing whilst wet, air-entrainment should be used (see 6.3.3 of BS 5528-1: 1997) and a strength grade may be reduced by 5mm					

Design of the beams

The beams are designed to carry their self-weights only since no imposed load is allowed. The panels on top are carried by the columns on either side. The moment and shear from the design in *appendix 3* is illustrated in *figure 5.1*. From the design only lower reinforcements are required and therefore the 2Y8 top and bottom as illustrated in *figure 5.2* are used.

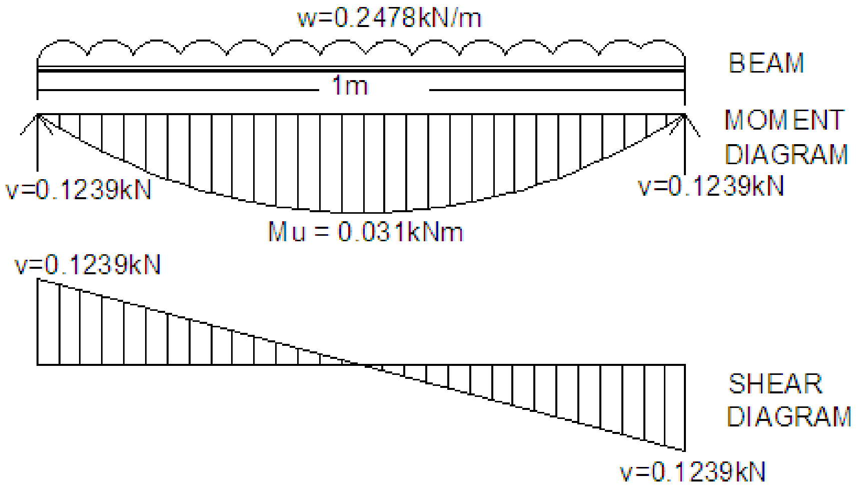


Figure 5.136: Moment and shear diagrams for the beams

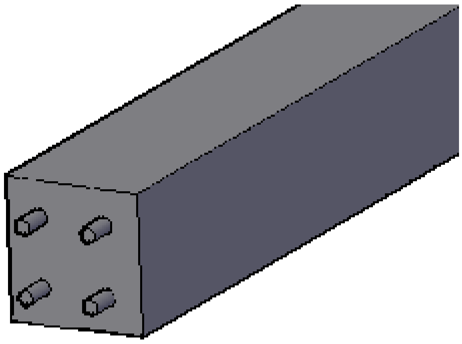


Figure 5.237: Illustration of the reinforcement in beams

Design of columns

Design of columns is in accordance with *BS 8110 part 1, section 3.8*

Columns have a cross-section of 80mm*80mm and a height of 1024mm, considered as 1000mm.

Since lateral deformation is restrained in particular planes, the columns are designed as braced and pinned connected at both ends. *Table 5.3* gives a factor for determination of the effective height as 1, that is, $\beta = 1$ in $l_e = \beta\, l_o$.

Table 5.3: Table of Nominal restraint factor in columns: Table 3.19 of BS 8110

Table 3.19 — Values of β for braced columns

End condition at top	End condition at bottom		
	1	2	3
1	0.75	0.80	0.90
2	0.80	0.85	0.95
3	0.90	0.95	1.00

Table 3.20 — Values of β for unbraced columns

End condition at top	End condition at bottom		
	1	2	3
1	1.2	1.3	1.6
2	1.3	1.5	1.8
3	1.6	1.8	—
4	2.2	—	—

3.8.1.6.2 *End conditions*

The four end conditions are as follows.

a) *Condition 1.* The end of the column is connected monolithically to beams on either side which are at least as deep as the overall dimension of the column in the plane considered. Where the column is connected to a foundation structure, this should be of a form specifically designed to carry moment.

b) *Condition 2.* The end of the column is connected monolithically to beams or slabs on either side which are shallower than the overall dimension of the column in the plane considered.

c) *Condition 3.* The end of the column is connected to members which, while not specifically designed to provide restraint to rotation of the column will, nevertheless, provide some nominal restraint.

d) *Condition 4.* The end of the column is unrestrained against both lateral movement and rotation (e.g. the free end of a cantilever column in an unbraced structure).

Short and slender columns

This check is as in *section 3.8.1.3 of the BS 8110,* for which column has l_{ex}/d = l_{ex}/b = 1000mm/80mm = 12.5 < 15 (for braced column), hence column is short.

The *section 3.8.1.7* limits the slenderness such that the clear length l_o <= 60b = 4800mm

From the analysis in *appendix 3,* 4Y8 reinforcement is sufficient while only minimum shear reinforcement is required. The shear reinforcement is provided as binding wire that ties the reinforcement bars to the endplates, *figure 5.3.*

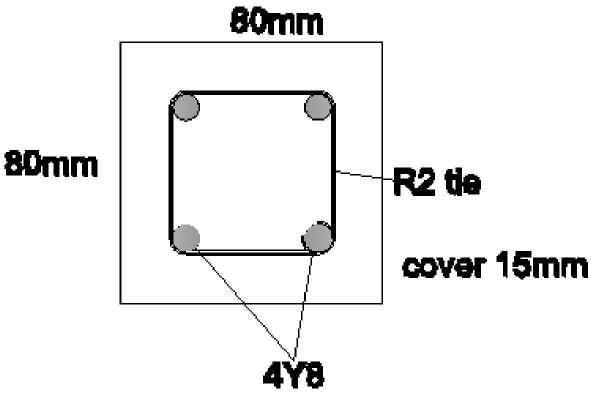


Figure 5.3: Illustration of reinforcement and tie of the columns

5.2.8 Determination of the wind loading

Wind load on the structure is considered for two orthogonal directions. The roof is duo-pitch truss, with three trusses at equidistant spanning the shorter length of the structure.

The design is based on *BS 6399 part 2, Code of Practice for Wind loads.*

The standard method is described in *section 2* of the *BS 6399* for which the wind direction is considered normal to the faces of the structure and an equal-duo-pitch roof recommended as in *section 2.1.1.1*

The dynamic pressure is determined by $q_s = 0.613\ V_e^2$ where V_e is the effective wind speed (m/s) *Section 2.1.2.1*

The effective wind speed is determined as in s*ection 2.2.3*: $V_e = V_s * S_b$

From *table 5.4,* interpolation for a height of 3.3m above ground for a case of site location exceeding 100km from the sea, the terrain and building factor is approximated as S_b =1.35

Table 5.4: Table for wind parameter S_b: Table 4 of BS6399

Site in country or up to 2 km into town					Site in town, extending ≥ 2 km upwind from the site			
Effective height H_e	Closest distance to sea upwind km				Effective height H_e	Closest distance to sea upwind km		
m	≤ 0.1	2	10	≥ 100	m	2	10	≥ 100
≤ 2	1.48	1.40	1.35	1.26	≤ 2	1.18	1.15	1.07
5	1.65	1.62	1.57	1.45	5	1.50	1.45	1.36
10	1.78	1.78	1.73	1.62	10	1.73	1.69	1.58
15	1.85	1.85	1.82	1.71	15	1.85	1.82	1.71
20	1.90	1.90	1.89	1.77	20	1.90	1.89	1.77
30	1.96	1.96	1.96	1.85	30	1.96	1.96	1.85
50	2.04	2.04	2.04	1.95	50	2.04	2.04	1.95
100	2.12	2.12	2.12	2.07	100	2.12	2.12	2.07

NOTE 1 Interpolation may be used within each table.
NOTE 2 The figures in this table have been derived from reference [5].
NOTE 3 Values assume a diagonal dimension a = 5 m.
NOTE 4 If H_e > 100 m use the directional method of Section 3.

The site wind speed is determined as per *section 2.2.2* where several parameters come into play:

$V_s = V_b . S_a . S_d . S_s . S_p$

$S_a = 1 + 0.001(\Delta)_s$ the altitude factor for this case the altitude is as 2500*masl*, hence $S_a = 3.5$

S_d = the directional factor for which a critical case of 1 is assumed

S_s = 1 for permanent building as the seasonal factor,

S_p= 1 normal design applications, the probability factor

V_b is the basic wind speed taken as 10m/s

Therefore the site wind speed is derived to be

V_s=10 ×3.5×1×1×1 = 35 m/s

And V_e = 35*1.355 = 47.425m/s

Thus, the dynamic wind speed: $q = V_e^2 \times 0.613 = 1378.717 \text{N/m}^2$ *or* 1.38 kN/m^2

External and internal pressure coefficients

The values of external and internal pressure coefficients are determined in the *appendix 4* and summarised in the *table 5.5* for evaluation of the zonal net pressures.

Table 5.5: Summary of determination of zonal net wind pressures

		15°	**22.5°**	**30°**	***Cpe - Cpi***	***Cpe – Cpi***	**Net pressure (KN/m^2)**	**Net pressure (KN/m^2)**
0°					*Cpi* = +0.2	*Cpi*= -0.3	*Cpi*=+0.2	*Cpi*= -0.3
	A	-1.1	-0.8	-0.5	-1	-0.5	-1.38	-0.69
		0.2	0.5	0.8	0.3	0.8	0.414	1.104
	B	-0.8	-0.65	-0.5	-0.85	-0.35	-1.173	-0.483
		0.2	0.35	0.5	0.15	0.65	0.207	0.897
	C	-0.4	-0.3	-0.2	-0.5	0	-0.69	0
		0.2	0.3	0.4	0.1	0.6	0.138	0.828
	D	-	-	-				
	E	-1.3	-1.1	-0.9	-1.3	-0.8	-1.794	-1.104
		-1.3	-1.1	-0.9	-1.3	-0.8	-1.794	-1.104
	F	-0.9	-0.7	-0.5	-0.9	-0.4	-1.242	-0.552
		-0.9	-0.7	-0.5	-0.9	-0.4	-1.242	-0.552
	G	-0.5	-0.5	-0.5	-0.7	-0.2	-0.966	-0.276
		-0.5	-0.5	-0.5	-0.7	-0.2	-0.966	-0.276
90°								
	A	-1.6	-1.4	-1.2	-1.6	-1.1	-2.208	-1.518
	B	-1.5	-1.3	-1.1	-1.5	-1	-2.07	-1.38
	C	-0.6	-0.6	-0.6	-0.8	-0.3	-1.104	-0.414
	D	-0.4	-0.45	-0.5	-0.65	-0.15	-0.897	-0.207

The net pressure, last two columns of the *table 5.5*, is given by *NP = (Cpe – Cpi)*dynamic pressure*C_a*, C_a is the factor for the non-simultaneous action of the gust wind across the external surface of the structure, most severe case is when $C_a = 1$

The wind pressures derived above are acting perpendicular to the roof and are so applied on the analysis of the construction introducing both effects of vertical and horizontal effects.

Wind loading on the panels/walls

Wind loads on the panels constitute the horizontal force that the structure has to withstand via the lateral stiffening of the panels and the connections.

With the dynamic pressure as determined earlier as qs = 1.38kN/m^2 and the internal and external pressure coefficients determined in *appendix 4*, the net pressures for 0^o and 90^o are:

0^o: $NP = qs\ (Cpe - Cpi)*C_a$ = 1.38kN/m^2*(+0.85 – 0.2)*1 = 0.897 kN/m^2

90^o: $NP = qs\ (Cpe - Cpi)*C_a$ = 1.38kN/m^2*(-0.5 + 0.3)*1 = -0.276 kN/ m^2

The net pressure is therefore taken to be 1 kN/m^2. Considering a factor of safety of 1.5, the design wind pressure is 1.5kN/m^2 acting on the wall elements of the structure.

5.3 Check for span/stage construction

5.3.1 Stage construction

The safety of stage construction is ascertained by the possibility of erection of columns and assembly of elements at the respective stage without risk of collapse otherwise supports will be required to enhance safety. Simple calculation of the effect of horizontal loads on the columns is done as follows:

Given the two bolts anchorage of columns as illustrated in *figure 5.4*, the moment resistance against topping of the column about the edges *A* and *B* is determined. For the M10 bolts of grade 8.8, the nominal tensile strength, R_n, is 800N/mm^2. With the factor of safety of 1.2, the design tensile strength, R_d, is R_n/1.2 = 800N/mm^2/1.2 = 666.67N/mm^2.

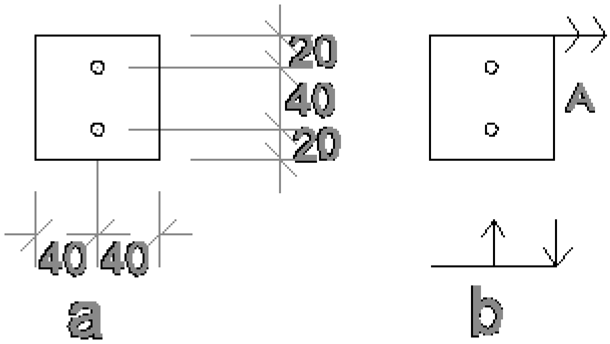


Figure 5.438: Column bolts and toppling effects: a – dimensions, b – toppling about edge A, c – toppling about edge B

a) Check for moment resistance against toppling about edge *A* of *figure 5.4*.

Given: the design tensile strength capacity of bolts: 333.33N/mm^2

The area of bolt resisting tensile forces: A_{sbolt} = π*(8.5mm/2)^2 = 56.745mm^2. (Note the diameter of bolt resisting tension is the diameter of bolt (10mm) less the thread pitch of 1.5mm from *table 4*).

Moment resistance against toppling about edge *A* is determined as follows:

M_u = (0.9*40mm*A_{sbolt}*R_d)*2

= 0.9*40mm*56.745mm^2*666.67N/mm^2*2 = 2.724x10^{6}Nmm

b) Check for moment resistance against toppling about edge *B* of *figure 5.4*.

The effect of the two bolts act at 20mm and 40mm lever-arms from the edge *B*. The moment resistance is thus:

M_u = 0.9*(20mm + 60mm)*A_{sbolt}*R_d

$= 0.9*(20mm + 60mm)*56.745mm^2*666.67N/mm^2 = 2.724x10^6Nmm$

c) Effect of wind loading

The wind load acts as a pressure determined as 1.5kN/m^2 as in *chapter 5.2*, and illustrated in *figure 5.5*.

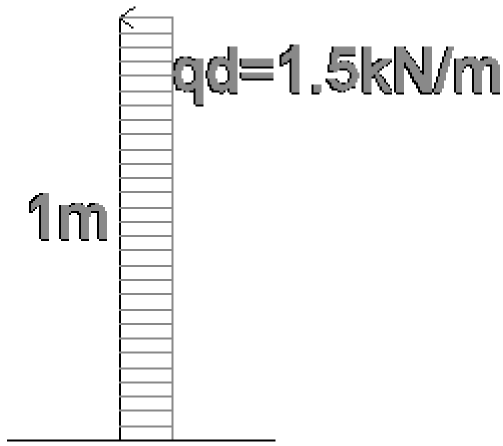


Figure 5.539: Wind load simulated to act as uniform load on columns

The design moment, M_d, due to the wind load is determined as follows:

$M_d = ½*q_d*l^2 = ½*1.5kN/m*(1m)^2 = 0.75kNm$ per 1m wide of wall that each column supports.

The moment resistance offered by the bolts in each direction of toppling illustrated above, $M_u = 2.724x10^6Nm = 2.724kNm$, which exceeds the moment due to the wind loading determined to be $M_d = 0.75kNm$. And since in each construction phases, 1m columns are assembled at a time, the toppling tendency is the same for all columns and is shown to be satisfactorily contained by the bolts.

d) Checking the free span of the wall

A simple method to check the free span length of wall that can be constructed is to determine the design moment at the top of a series of columns against the limit which is the moment resistance developed at the bottom of the columns. The design moment, M_d is determined by:

$$M_d = 1/8*qd*(nl_e)^2$$

Where q_d is the uniform load, l_e is the effective length between supports and n is the series of beam spans (equal to the length between columns).

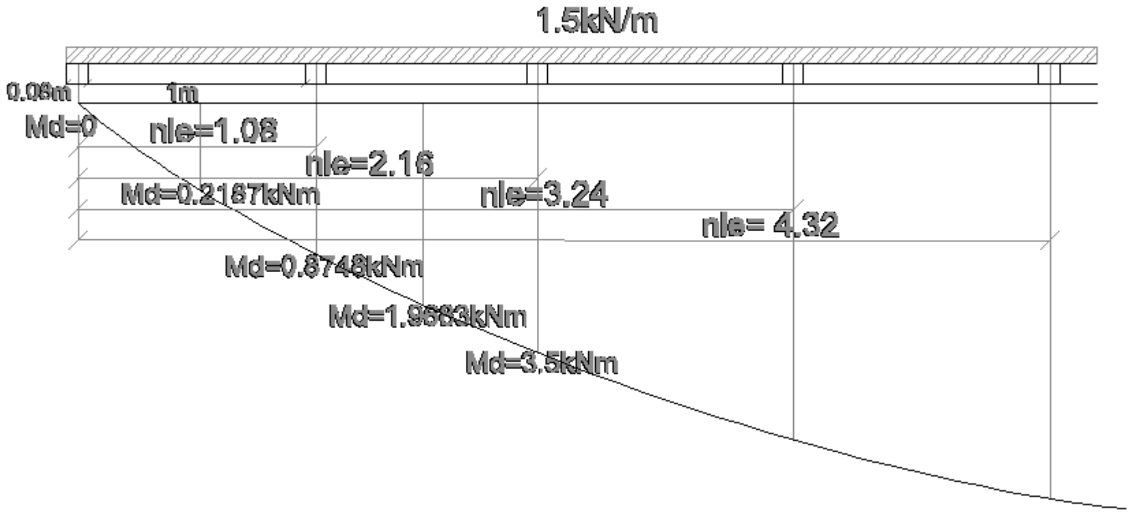


Figure 5.640: Illustration of the increase in M_d with increase in span length l_e

As the span length, l_e, increases, the mid-span design moment M_d, increases, *figure 5.6*. The requirement is that the M_d should be less than or equal to the moment capacity M_u, at the bottom of the respective column. Thus, for the grade 8.8 anchor bolts of tensile capacity of 800N/mm^2, the moment resistance as calculated earlier is 2.724kNm. This permits an assembly of up to l_e = 3m, at which point the M_d developed is 1.9683kNm. Beyond 3m span, for instance at l_e = 4m, the M_d = 3.5kNm which exceeds the capacity of the column anchorage to inhibit toppling. The maximum free span is therefore 3m (or 3 beams).

An expanded model can be generated to simulate the behaviour of a wall with free span of 4m (or 4 beams). This model exhibits a deflection that exceeds the allowable limit as discussed in *appendix 5*.

5.4 Design of the structure using SCIA Engineer Software

The analysis with *SCIA Engineer* software is done for the two construction stages: construction stage 1 – the window level assembly and construction stage 2 – the whole model assembly. The design of elements is done at the construction stage 2. The important loads are the dead loads of all elements and the wind loads on panels.

5.4.1 Modelling in Scia Engineer Software

The following are the steps and considerations in developing the model for analysis:-

i. The structure is made of 1D elements, which are, the beams and columns connected via connector cube elements and 2D elements that is, the slab panels with the respective dimensions. The connector cubes are modelled as hinged connectors.

ii. The supports are placed on the lower nodes of columns and are modelled to have reduced rigidity as per the fixity of the member,

iii. Slab panels are modelled to connect to the columns via hinges provided between the slab panels and the columns. Since this connection is done via the midline, only single panels can be modelled.

iv. Loads are introduced to the structure as dead loads, load case LC1, in all members and wind loads, load case LC2, on the panels and a load combination C1 is determined where a combination factor of 1.2 is used for both dead and wind loads, this factor is adopted for both the ULS and SLS design cases since no live loads are involved.

v. The 2D elements, that is, the slab panels are divided into a mesh of square 4-noded finite elements with an edge size of 50mm.

vi. The primary solution variables are the displacement of elements and the secondary solution variables are the moments, normal forces and shear forces.

vii. The ultimate outputs include the reinforcement requirements for the 1D and 2D elements and the displacements of the elements and of the structure.

5.4.2 Construction stage I: Phase one assembly

This constitutes assembly of elements to the window level as illustrated in *figure 5.7*. The panels are modelled as single panels as opposed to the double panels in the actual construction due to the limitation of the software.

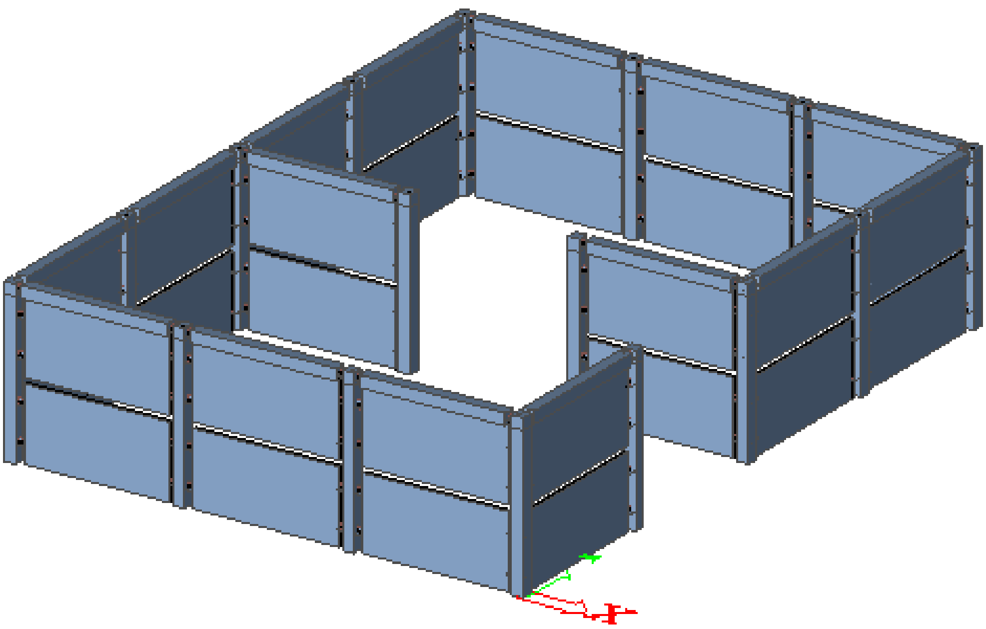


Figure 5.7: Visualization of the construction stage 1 assembly

Loading in construction stage 1

The loads include the dead loads as load case LC1 on all elements assembled and wind loading, load case LC2 on the panels. The wind load is 1.5kN/m^2 acting as horizontal loads on the panels as obtained from analysis of wind load on walls. The wind loading is modelled as surface loads on the slab panel elements as illustrated in the *figures 5.8* for wind in x-direction and for wind in the y-direction.

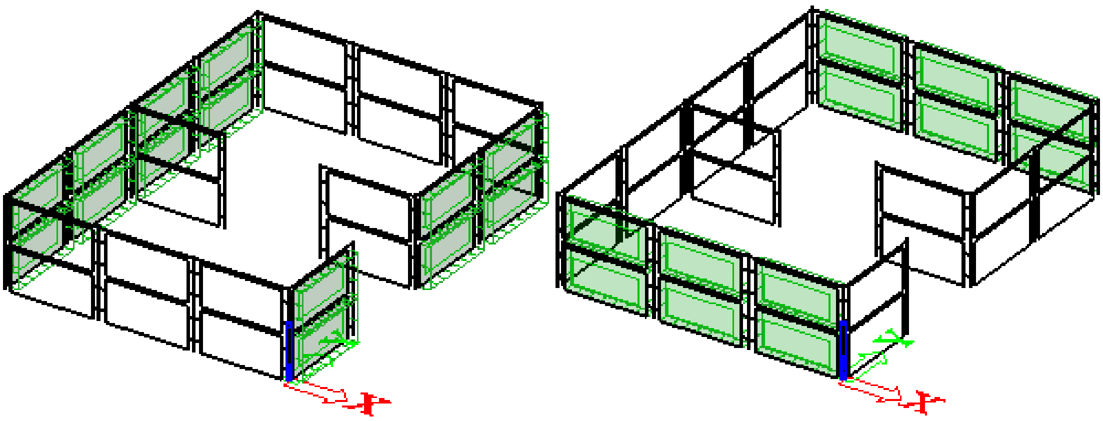


Figure 5.841: Wind loading visualization for construction stage 1

Results of analysis

Analysis was done for the LC1, LC2 and a load combination C1 for the given wind directions. The results for the combination case C1 are given below.

a) **Primary variable: Displacements**

Displacements due to wind in x-direction

Maximal displacements are observed at the middle of the free span wall facing the direction of the wind. This is ideally expected. A quick check on the deflection requirement for the 2000mm free span shows that the total deflection limit being span/500 is 2000mm/500 for this case of wind loading in the x-direction giving a deflection limit of 4mm. This has not been exceeded by the 1.8mm deflection observed in *figure 5.9* and therefore the construction process at this stage is safe for wind in the x-direction.

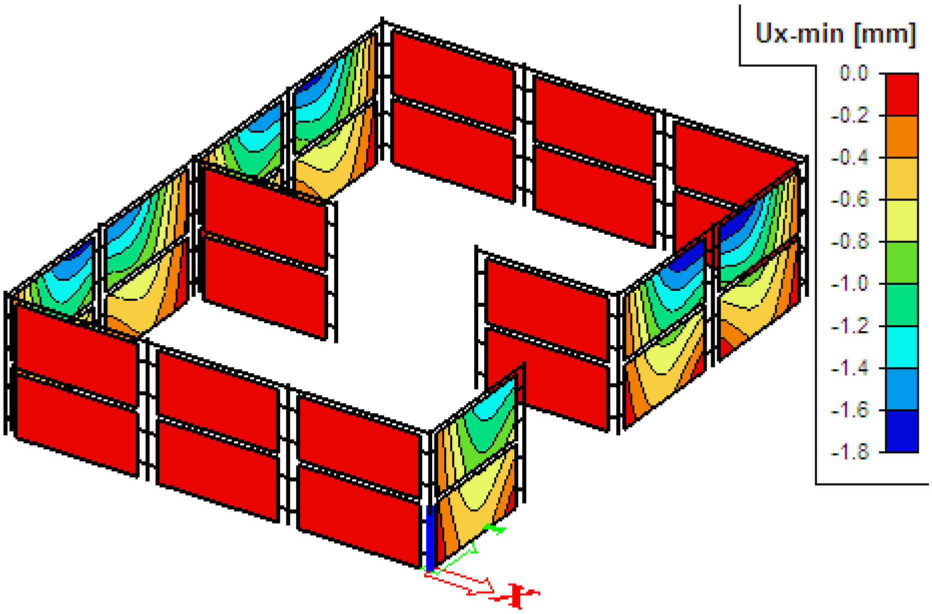


Figure 5.9: Displacement due to wind loading in x-direction

Displacement due to wind in y-direction

Deflection check for this case, with free span of 3000mm facing the wind, the total deflection limit being span/500 gives a limit of 6mm which is not exceeded by the 3.4mm deflection observed in *figure 5.10*. This shows that the construction is safe against wind in the y-direction.

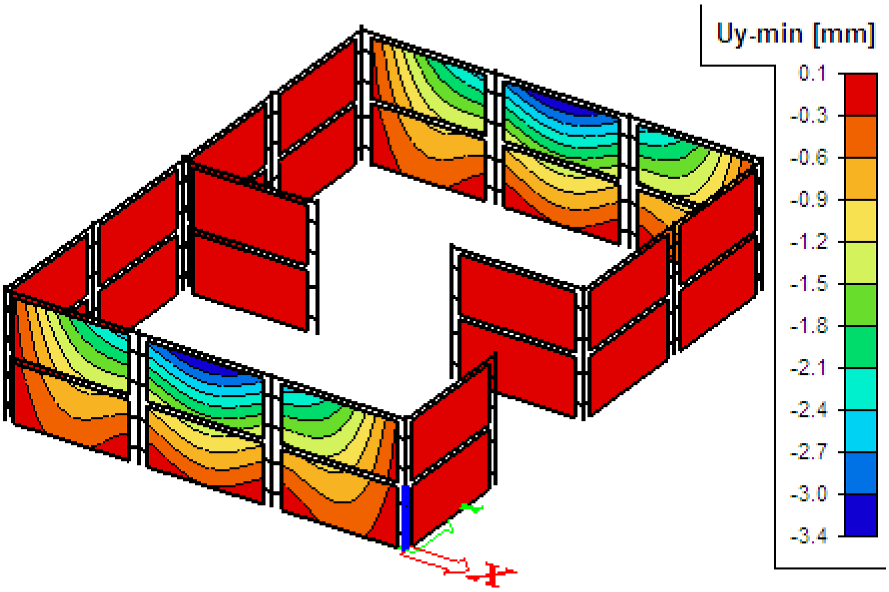


Figure 5.1042: Displacement due to wind loading in the y-direction

b) Secondary variable: Internal forces

Moments due to wind in the x-direction

The maximal negative moments are observed to be at the mid-line of the slab panels facing the wind while maximal positive moments are concentrated at the connection points between the slab panels and the columns, *figure 5.11*. This is expected since the panels are only supported at the vertical edges.

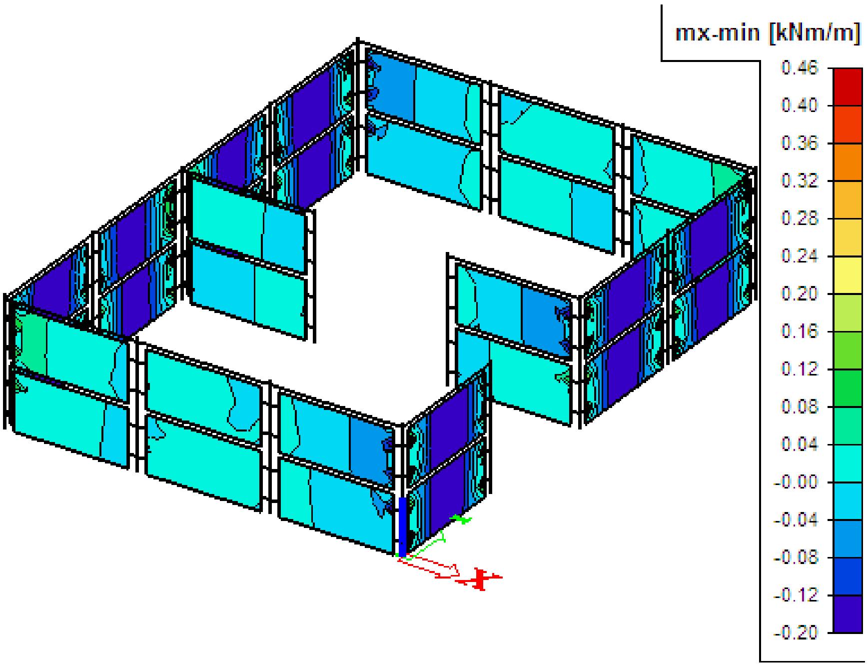


Figure 5.11: Moments due to wind loading in x-direction

Moments due to wind in the y-direction

Similar to the case above, the maximal moments are at the mid-line of slab panels and at the connection points, *figure 5.12.*

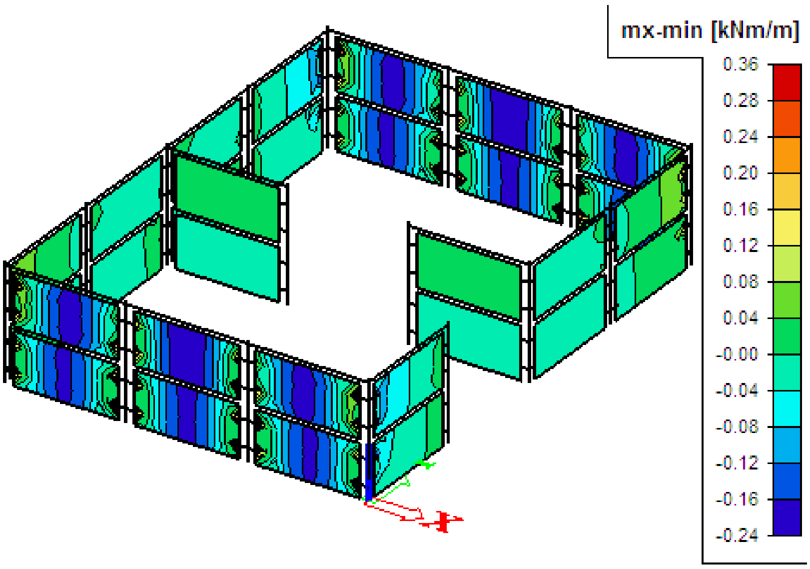


Figure 5.1243: Moments due to wind loading in the y-direction

These results of the construction stage 1 are not used in design of elements since the elements are not fully loaded. However, the deflections at this construction stage can give an indication of what is exhibited under the wind loading to show the safety of stage construction.

5.4.3 Construction stage II: Whole assembly analysis

The construction constitutes the whole structure as illustrated in *figure 5.13*. The openings for windows and door are as in the illustration.

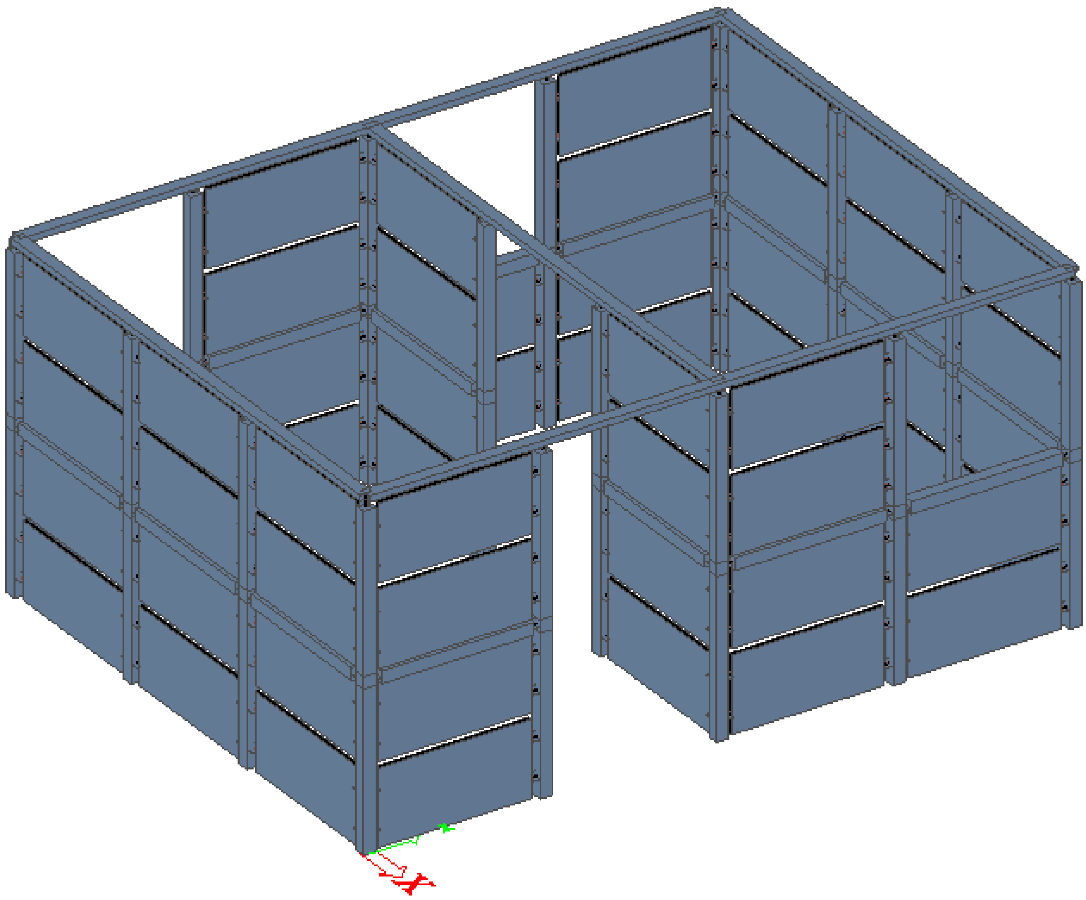


Figure 5.1344: Visualization of whole model at construction stage II

Loading of the structure at construction stage II

The loading for this construction stage constitutes self-weights of all elements as load case LC1, and load case LC2 being the roof loads derived earlier as equal to 11.1kN and the wind load on panels equal to 1.5kN/m^2. The roof loads are point loads at the top of the columns onto which roof trusses sit, as illustrated in *figure 5.14* together with wind loading in the x-direction.

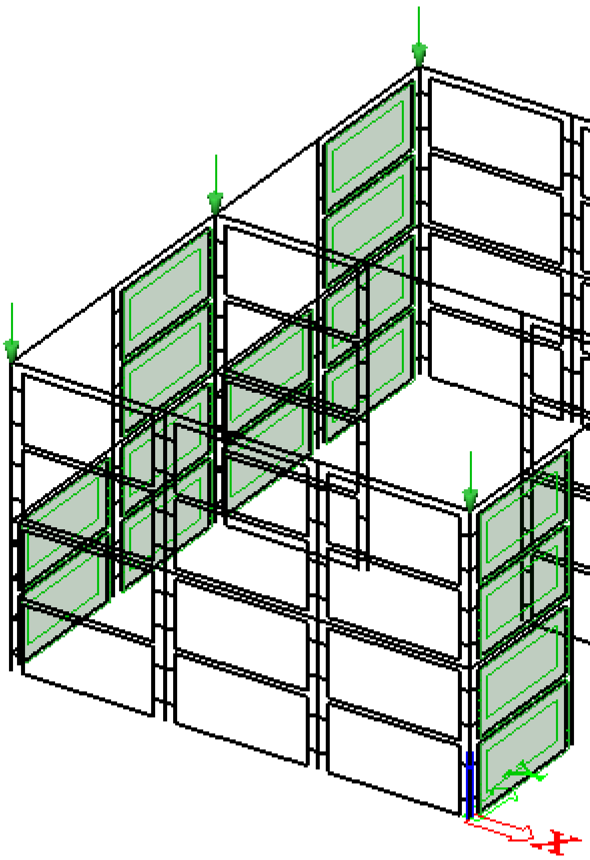


Figure 5.1445: Illustration of roof point loads and wind loading in x-direction

Results of the analysis

a) Primary variables: displacements

Displacement due to wind in the x-direction

Displacements are exhibited in the wall facing the direction of the wind. Maximal values tend to appear towards the centre of the wall ideally because of the concentration of pressure at these points. The presence of openings tend to reduce the pressure concentration and hence lesser displacements, *figure 5.15*. A maximal value of 2.4mm is observed against a limit of 4mm as discussed earlier. The structure is therefore safe against wind in the x-direction. While the total span is about 4m, the partition at the mid-span plays a role to reduce the free span and hence reduce the maximum displacement. Further illustration of this aspect is in the *appendix 5*.

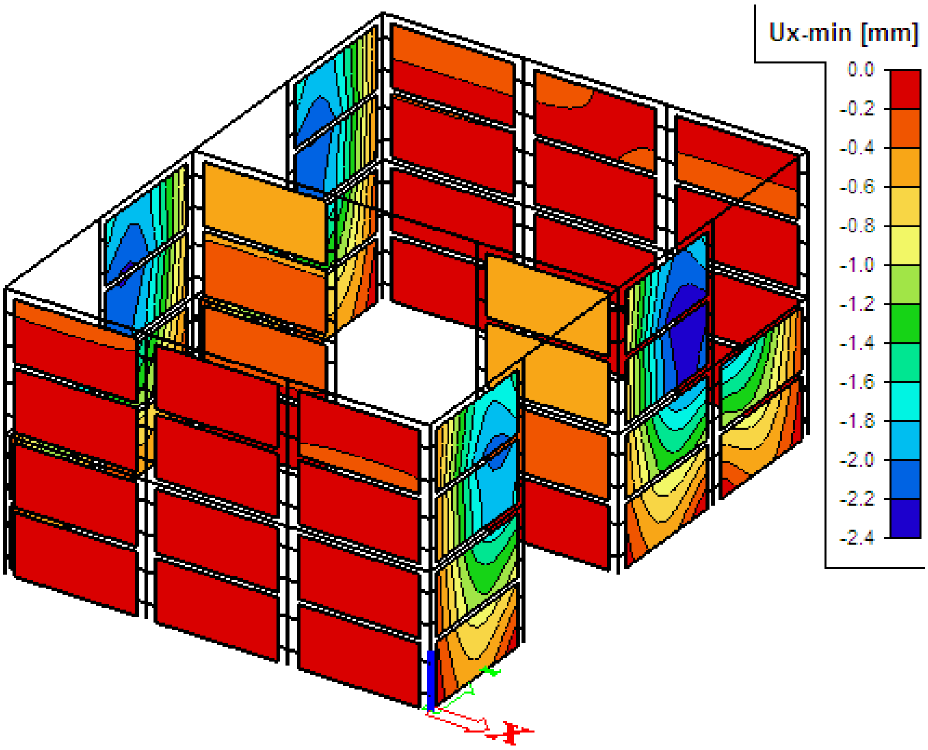


Figure 5.1546: Displacement of elements due to wind in x-direction

Displacement due to wind in the y-direction

The maximum displacements are observed at the top-centre of the free span of the wall facing the wind. A value of 7mm is exhibited which slightly exceeds the span/500 limit of the deflection given as 6mm, *figure 5.16*. The lintel can be stiffened or made out of concrete elements to reduce the deflections as shown in *figure 5.17*.

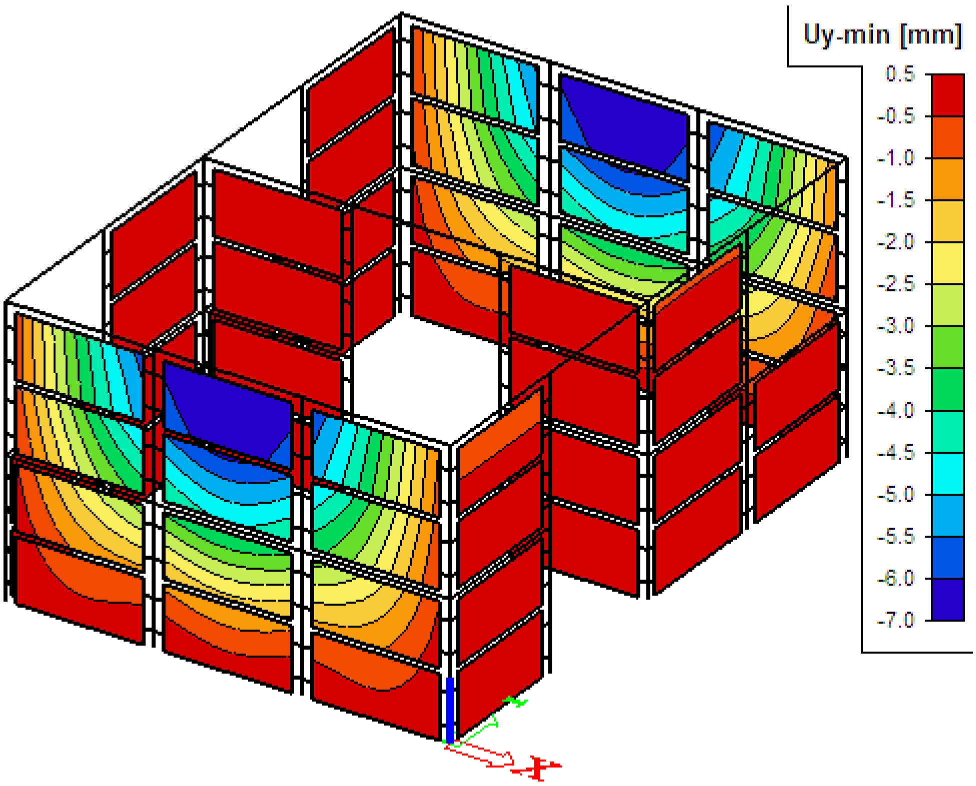


Figure 5.1647: Displacement of elements due to wind loading in the y-direction

The lintel beam can be made out of concrete elements as mentioned earlier or alternatively be stiffened by diagonal bracings to reduce the effective span. The later reduces the effective span of the lintel beam subjected to deflection and as such pushes the punching effect of wind loading concentration towards the middle of the wall which is the ideal scenario expected. The value of maximal displacement also reduces to about 4mm, *figure 5.17*, making the structure safe against wind in the y-direction.

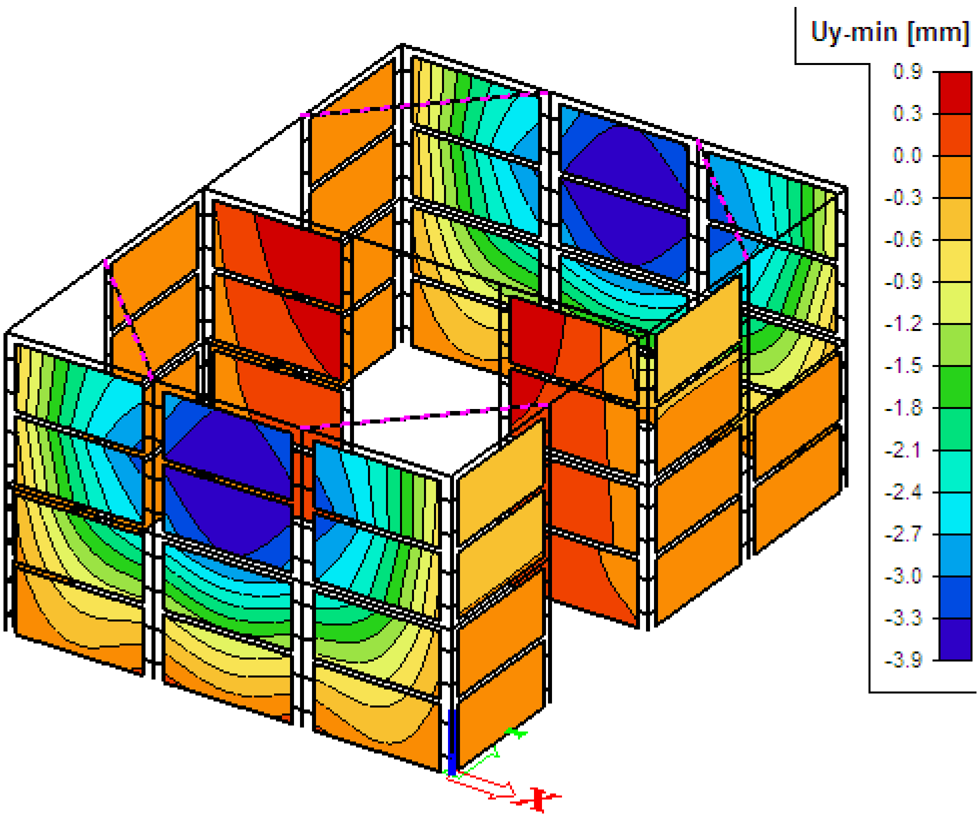


Figure 5.17: Displacement of elements with stiffened lintel beams

b) Secondary variables: Internal Forces

Moments due to wind in x-direction

The maximal moments are exhibited at the mid-line of the slab elements. The values of the maximum moments are highest for the panels above and reduce for lower panels. This is occasioned by the distance from the supports with higher fixity, that is, base of columns and lateral stiffening of partition walls in the respective axis. The results in *figures 5.18* and 5.19 can be attributed to this phenomenon to some extent but not ascertained at this stage of the research.

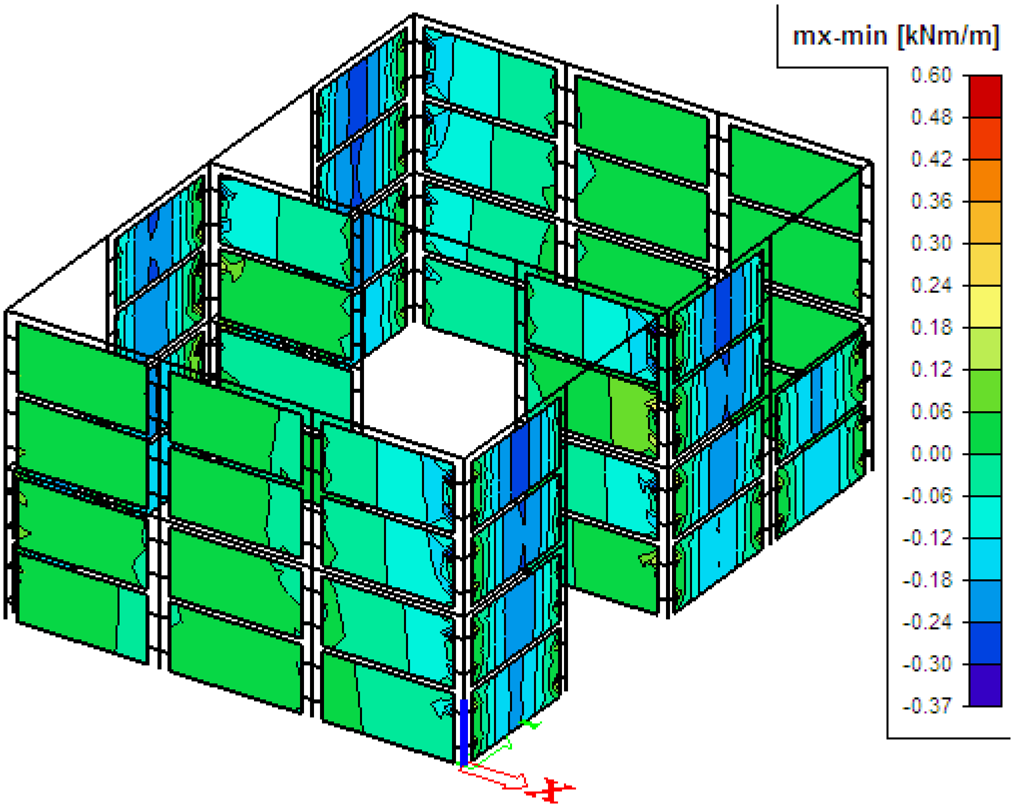


Figure 5.18: Moments developed in elements due to wind load in x-direction

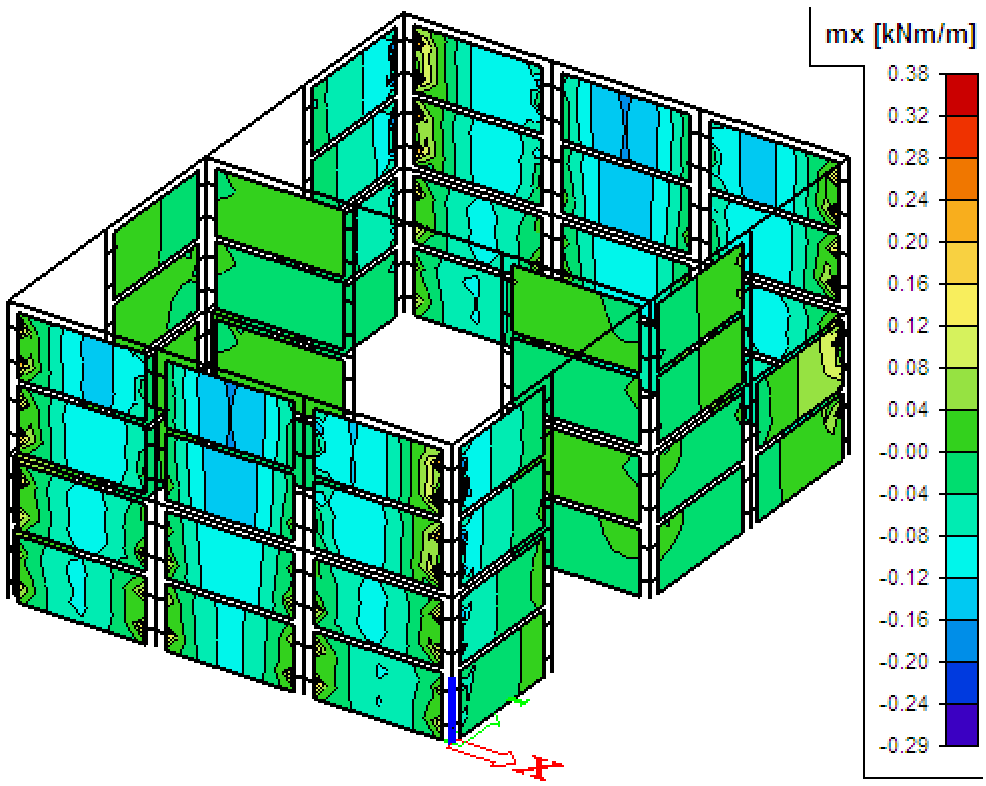


Figure 5.1948: Moments developed in elements due to wind loading in y-direction

5.4.4 Design for reinforcement

a. Beams/columns reinforcement

From the member forces illustrated in *chapter 5.4.3*, the highest loaded element is identified and used as the basis for design. It is also desirable that the beams and columns be made to be interchangeable and as such the column is designed and the result adopted for beams. The column element identified for design is highlighted in *figure 5.20* and has a normal compressive force of -29.27kN. The design of this element gives a reinforcement requirement of area 57mm^2 in yy-axis and also in the xx-axis yielding a total of reinforcement area of 114mm^2 in the column cross-section.

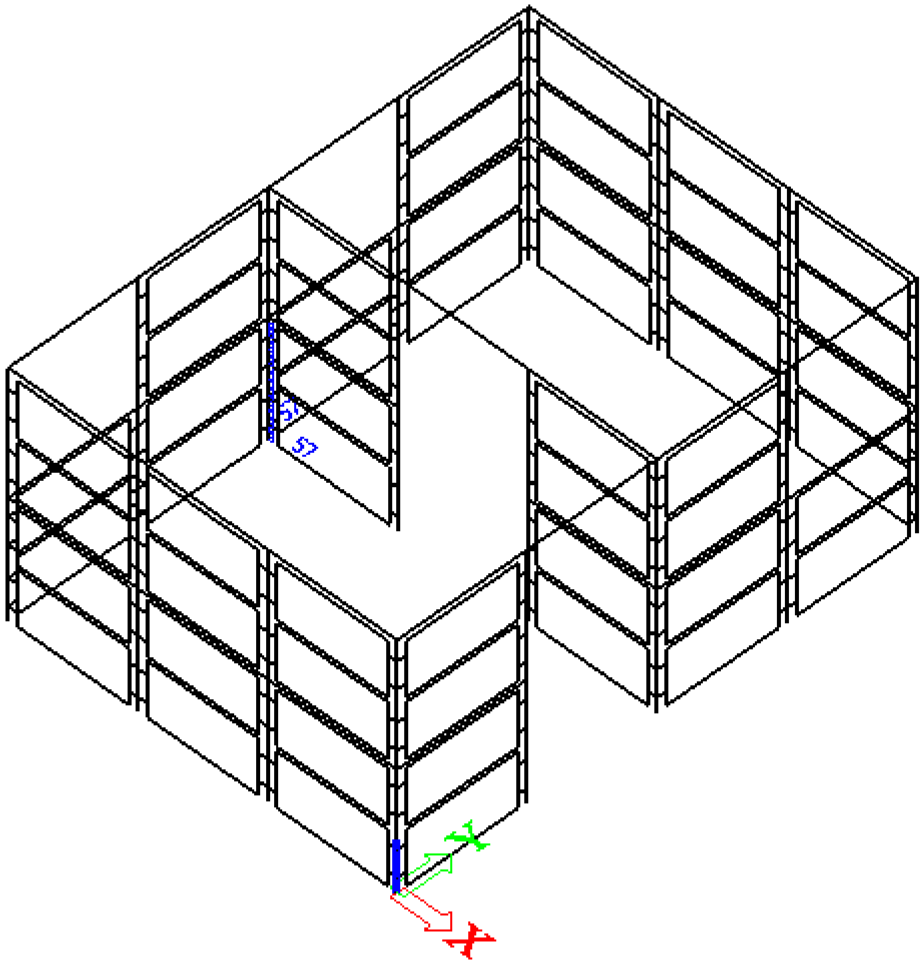


Figure 5.2049: Column identified for design

The sectional view of the column reinforcement shows 4Y6 bars distributed at the four corners leaving 15mm concrete cover as illustrated in *figure 5.21*. However, the reinforcement used in the actual construction includes 4Y8, with a reinforcement area of 201mm^2. This indicates an overdesign and therefore a recommendation is made that 4Y6 should be used.

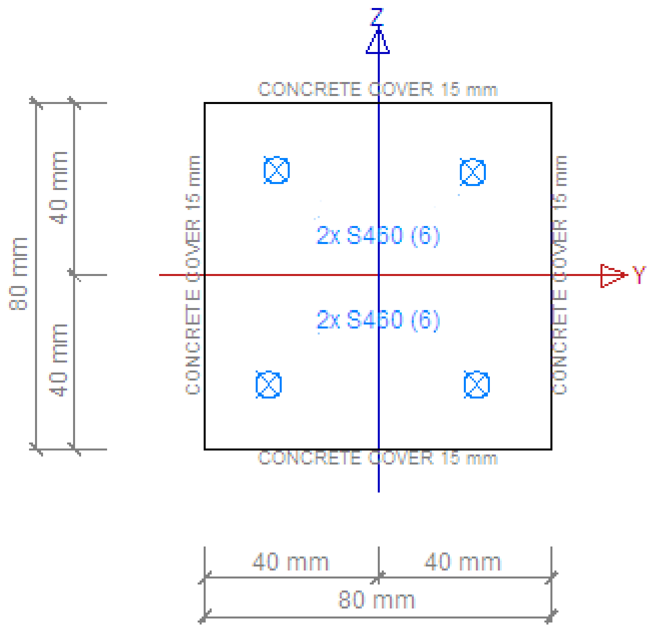


Figure 5.21: Illustration of the column reinforcement

b. Slab panel reinforcement

The slab panel reinforcement constitutes a square mesh of wire diameter 4mm and wire spacing of 70mm. This mesh is placed at the mid-line of the panels as illustrated in *figure 5.22*. In this figure, *d1* = *d2* = 4mm being the wire diameters and *cl* is the concrete cover of about 8mm at either sides.

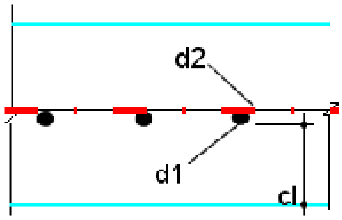


Figure 5.22: Slab panels wire mesh reinforcement

Determination of reinforcement areas

The heaviest loaded panel is identified and taken as the basis of design. For a given direction on the panel, the total amount of reinforcement area is determined and distributed on the cross-section of the panel in the said direction. A_{s1} is the reinforcement area in mm^2/m associated with diameter *d1* and A_{s2} is the reinforcement area associated with *d2* and they span the longer edge and shorter edge orthogonally respectively. From the analysis results *figure 5.23*, the A_{s1} and A_{s2} for the heaviest loaded elements are shown.

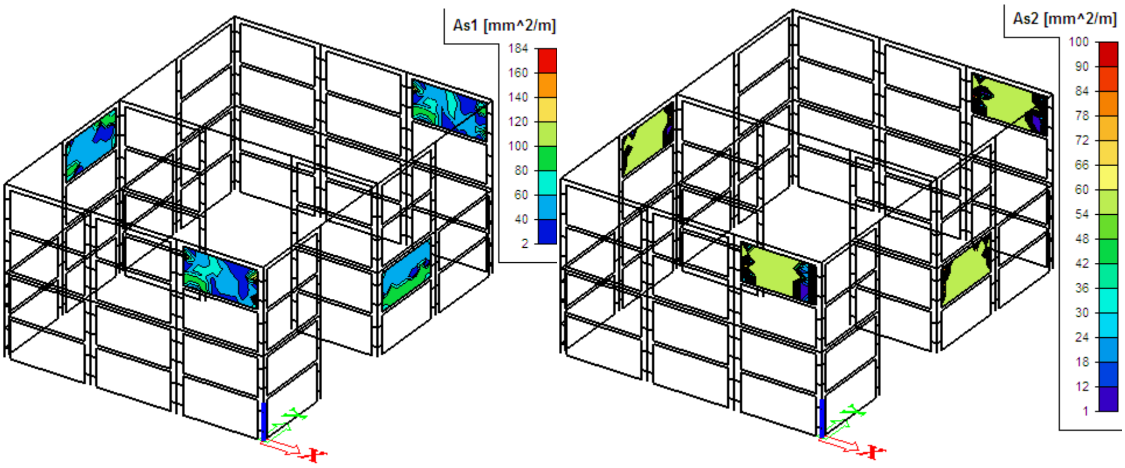


Figure 5.2350: Illustration of reinforcement areas A_{s1} and A_{s2}

Figure 5.24 and 5.25 shows the heaviest loaded panel which require average reinforcement areas of 120mm^2/m and 84mm^2/m for A_{s1} and A_{s2} respectively outside the connection points. These areas are taken to be the basis of design for reinforcement of the slab panels. By observation, concentration of reinforcement requirement is at the connection points and lesser elsewhere.

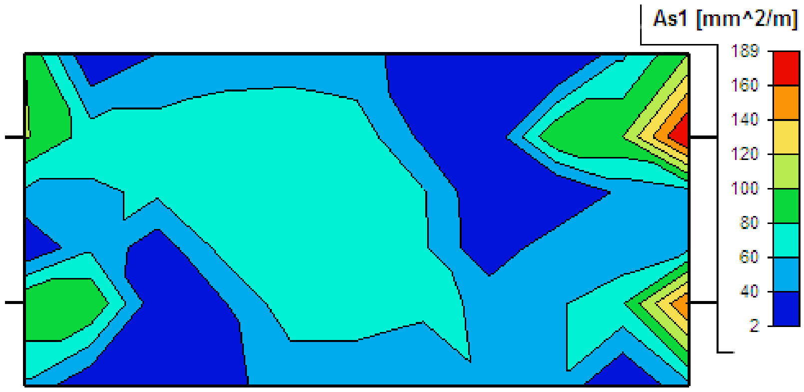


Figure 5.24: Reinforcement requirement A_{s1} distribution for 2D panels

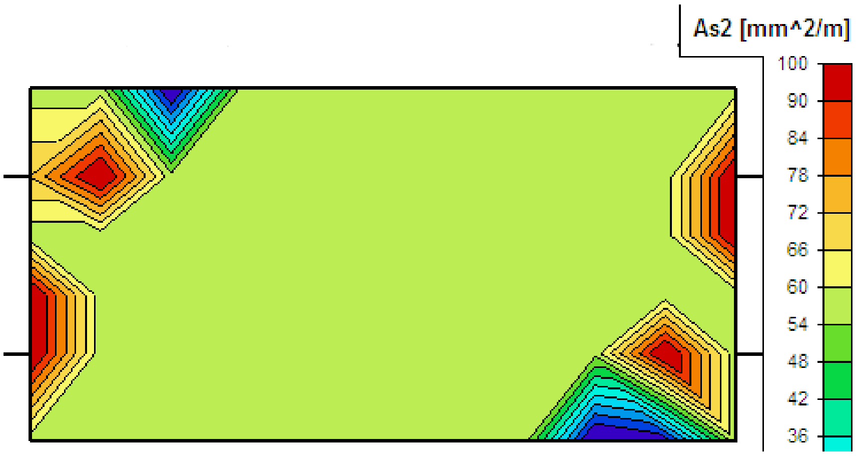


Figure 5.2551: Reinforcement requirement A_{s2} distribution for 2D panels

Considering that A_{s1} spans the longer length of about 1m and A_{s2} spans the shorter length of 0.5m, the total reinforcement areas required is therefore:

$$A_{s1} = 120\text{mm}^2/\text{m}*1\text{m} = 120\text{mm}^2$$

$$A_{s2} = 84\text{mm}^2/\text{m}*0.5\text{m} = 42\text{mm}^2$$

The number of wires in the longer edge is determined by dividing the length by the spacing, that is, 1000mm/70mm = 14 wires. Given that the total area of reinforcement over this length, A_{s1} = 120mm^2, then each wire has an area of 8.57mm^2, which for round wires implies a diameter of at least 3.3mm.

For the shorter ledge of 500mm and a wire spacing of 70mm, the number of wires possible is 7. Given the total reinforcement area of 42mm^2 over this length, one wire has an area of 42mm^2/7 = 6mm^2. This implies a diameter of at least 2.8mm. The square mesh of wire diameter of 4mm and spacing of 70mm therefore suffices in both A_{s1} and A_{s2}.

At connection points of the slab panels, a steel plate of 3mm in thickness and square sides of length 80mm is provided as anchor plates which provide a steel cross-section of 240mm^2. This exceeds the steel required at these connection points which is a maximum of 189mm^2/m as on *figure 5.24*, equivalent to18.9mm^2 assuming an action length of 0.1m at connection points.

5.4.5 Conclusion

From the analysis carried out above, a structure constructed in this manner and having maximum free span of 3m is safe against wind. With partitions added to reduce the free spans severity of the effect of wind loading becomes even lesser. The difference in the reinforcement provisions resulting from hand calculation and that from the Scia software is attributed to the assumption made in the hand calculation that the columns do not carry moments for simplification of analysis while the software has included some bending moment in columns in the axes that have some fixity. The reinforcement determined by Scia software analysis including 4Y6 for both beams and columns and square mesh of wire diameter at least 4mm and wire spacing of at most 70mm should suffice for such a structure and hence recommended.

CHAPTER 6: COST IMPLICATION OF THE TECHNOLOGY

6.1 Introduction

The total cost of the construction is determined by considering and calculating the following cost items:

1. Cost of materials and fabrication of moulds,
2. Cost of each material needed to produce $1m^3$ of concrete mix,
3. Cost of reinforcement, inserts and other attachments,
4. Cost of labour required in producing each element and assembly of construction,
5. Cost of running machines and equipment required during manufacture of elements,
6. Cost of the connection accessories and tools required,
7. Cost of materials and labour for roofing, fittings and services
8. Cost of transport and storage during production and construction

6.2 Cost estimates for various items

6.2.1 Cost of steel moulds: Materials and fabrication

There are three different elements being developed and for each type two moulds are fabricated. The material requirement for development of these moulds and the costs are as in the *table 6.1*.

Table 6.1: Table of breakdown cost of moulds fabrication

Item	Unit size	Unit Cost (KES)	Quantity	Amount (KES)
Slab panels mould				
Steel plate	full size (2500mm*1250mm)	8000	half size (1250*1250)	4000
Angle line bar	full length (6m)	800	full length (6m)	800
Bolts & nuts	pieces	20	10	200
Wooden inserts			lump sum	200
Fabrication labour			lump sum	1000
Beam mould				
Steel plate	full size (2500mm*1250mm)	8000	quarter size (1250mm*625 mm)	2000
Bolts & nuts	pieces	20	20	400
Fabrication labour			lump sum	1000
Styrofoam inserts	Reused waste	no cost		
Connector cube mould				
Steel plate	full size (2500mm*1250mm)	8000	one eighth size (625mm*625m m)	1000
Bolts & nuts	pieces	20	8	160
Fabrication labour			lump sum	500
	Total cost for one mould of each kind			**11,260**
	Cost for two moulds of each kind			**22,520**

6.2.2 Cost of hand tools

Hand tools are required in the processes of mould assembly, disassembly and during actual construction assembly works. The following are the main tools required, *table 6.2*.

Table 6.2: Table of cost of hand tools required

Item	**Quantity**	**Unit Cost**	**Amount**
Hack saw blade	1	300	300
Round & flat files	2	120	240
Grinding disks	2	200	400
		Total cost	**940.00**

6.2.3 Cost of Materials in 1m^3 of SCC

Material cost of producing 1m^3 of SCC is determined by costing each material proportionately with respect to their quantity by weight in the 1m^3 mix. This is accomplished as illustrated in the *table 6.3* yielding a cost intake per material.

Table 6.3: Table of the cost of materials in 1m^3 of SCC mix

Materials	**Volume (m^3)**	**Density (Kg/m^3)**	**Quantity (Kg)**	**Cost (KES) per quantity**	**Total cost (KES)**
Cement	0.1377	3240	446.148	840/50kg	7,495.29
Lime	0.0412	384	15.8208	650/25kg	411.34
Fine Aggregates	0.26	2200	572	1000/1000kg	572.00
Course Aggregates	0.35	2500	875	1100/1000kg	962.50
Super-plasticizer	0.0041	1000	4.1	123,795/210ltrs	2,416.95
Water	0.207	1000	207	200/10,000ltrs	4.14

				Total cost	**11,862.22**
Add 50% and a further 20% of total cost to account for losses and cost variation respectively					**20,165.00**

6.2.4 Equipment requirement

Equipment required include gravity mixer, weighing balance and a compressive strength testing machine. Others include: the slump flow test equipment and test cube moulds, which are omitted in this cost analysis.

Analysis of the hourly cost rating of each item is available in the *appendix 7*, the summary of which is provided in *table 6.4*. The maximum time the equipment can be in operation differs from element to element.

Table 6.4: Table of machine/equipment per hour costs

Equipment	**Cost per hour (KES)**
Gravity tilting mixer	112.44
Weighing balance	12.74
Compression testing machine	15.14

6.2.5 Cost of reinforcement and labour

The items including reinforcement steel and labour are dependent on the particular element being manufactured. For each element therefore, the cost of each of these items is determined with the idea of how many elements are manufactured from every 1m^3 of mix, the cost requirement for the cubic metre mix is determined.

6.2.6 Cost summary of each element

1. **SLAB PANEL**

A summary of the cost of requirements in the manufacture of slab panels is given in *table 6.6.*

Table 6.5: Material costs for the manufacture of slab panels

ITEM			**COST (KES)**
CONCRETE MIX	**Quantity (m^3)**	**Cost per m^3 (KES)**	
Volume of mix for 2 panels	$0.025166m^3$	20,165	507.47
EQUIPMENT	**Quantity (hr)**	**Cost per hour (KES)**	
Gravity tilting mixer running time	1	112.44	112.44
Weighing balance running time	0.5	12.78	6.39
LABOUR			
Mould assembly labour time	2	100	200
Concrete mixing/placing labour time	1	100	100
Demoulding and curing labour time	2	100	200
STEEL COMPONENTS	**Quantity**	**Cost per unit (KES)**	
Weight of steel in 2 panels	5kg	100/kg	500
Steel fabrication labour		lump sum	100
Connection studs & nuts	16	10	160
Cost for two slab panels			**1,886.30**
COST OF ONE SLAB PANEL			**943.15**

2. BEAMS/COLUMNS

A summary of the cost of requirements in the manufacture of beams/columns is given in *table 6.6.*

Table 6.6: Material costs for manufacture of beams/columns

ITEM			**COST (KES)**
CONCRETE MIX	**Quantity (m^3)**	**Cost per m^3(KES)**	
Volume for two beams	0.01310m^3	20,165	264.16
EQUIPMENT	**Quantity (hr)**	**Cost per hour (KES)**	
Gravity tilting mixer running time	1	112.44	112.44
Weighing balance running time	0.5	12.78	6.39
LABOUR			
Mould assembly labour time	2	100	200
Concrete mixing/placing labour time	1	100	100
Demoulding and curing labour time	2	100	200
STEEL COMPONENTS	**Quantity**	**Cost per unit (KES)**	
Weight of steel	3.4kg	100/kg	340
Steel fabrication		lump sum	100
Connection studs & nuts	16 pieces	10	160
Cost of two beams			**1,483.00**
COST OF ONE BEAM			**741.50**

3. CONNECTOR CUBES

A summary of the cost of requirements in the manufacture of connector cube is given in *table 6.6.*

Table 6.7: Cost of manufacture of connector cubes

ITEM			**AMOUNT (KES)**
CONCRETE MIX	**Quantity (m^3)**	**Cost per m^3 (KES)**	
Volume of mix	$0.000512m^3$	20,165	10.30
STEEL COMPONENTS	**Quantity**	**Cost per unit (KES)**	
Weight of steel	0.1kg	100/kg	10.00
Fabrication labour		lump sum	100
Connection studs & nuts	12 pieces	10	120
		Total cost	**150.30**

6.2.7 Roofing, fittings and finishes

A summary of the cost of requirements in the construction of the roof, fabrication of the windows and doors and finishing the structure is given in *table 6.8.*

Table 6.8: Summary of materials for roofing & fittings

ITEM	Quantity	Unit Cost (KES)	Amount (KES)
ROOFING MATERIALS			
Rectangular hollow sections 25mm*50mm*6000mm	4	1,200	4,800
Square hollow sections 40mm*40mm*6000mm	5	900	4,500
Angle line bars 40mm*40mm*6000mm	5	1,100	5,500
Galvanized iron sheets 2000mm	14	900	12,600
J bolts, kgs	2	250	500
Bolts and nuts 50mm, pieces	30	20	600
FITTINGS			
Windows	3	3,500	10,500
Metal door	1	10,000	10,000
Foundation Bolts	40	100	4000
LABOUR			
Roofing work, lump sum			3000
Finish and painting			4000
		Total Cost	**60,000.00**

6.2.8 Total cost of pilot house

The cost of the pilot structure from the above details with unit costs rounded up to whole figures is summarised in the *table 6.9*.

Table 6.9: Table of the summary of cost of model construction

COMPONENT	**QUANTITY**	**UNIT COST**	**AMOUNT**
MOULDS & TOOLS			23,460
CONNECTOR CUBES	14	150	2,100
BEAMS	12	800	9,600
COLUMNS	28	800	22,400
SLAB PANELS	84	1,000	84,000
FLOOR, ROOFING, FITTINGS & FINISHES	8	7,500	60,000
HOUSE ASSEMBLY COST	20	500	10,000
Add 10% Contingencies			21,156
	TOTAL COST		**232,716.00**

THE COST OF CONSTRUCTION IS APPROXIMATED TO BE KES 235,000.00

6.2.9 Cost of typical brick and mortar house

Cost of a similar construction made of masonry bricks and cement mortar is summarised in *table 6.10*. Alternatively, a conventional per square rate can be used which gives a range of KES 20,000 to KES 25,000 per square meter of floor plan. Using the minimum value of KES 20,000 for the $8m^2$ floor plan, the total construction cost can be estimated at KES 160,000.00.

Table 6.10: Summary of cost of a typical masonry structure

COMPONENT	**QUANTITY**	**UNIT COST**	**AMOUNT**
TOOLS & SCAFOLDING			10,000
BRICKS	1500 PCS	10	15,000
CEMENT	35 BAGS	900/BAG	31,500
SAND	5 TON	1,000	5,000
ROOFING, FITTINGS & FINISHES			57,000
TRANSPORT			10,000
LABOUR: 30% OF THE COST OF MATERIALS			31,200
Add 10% Contingencies			15,520
	TOTAL COST		**170,720.00**

6.2.10 Cost of a typical dwelling unit

A typical dwelling unit of about $48m^2$ constitutes 1 bedroom, 1 living room, 1 kitchen and 1 washroom as sketched in the plan in *figure 6.1*.

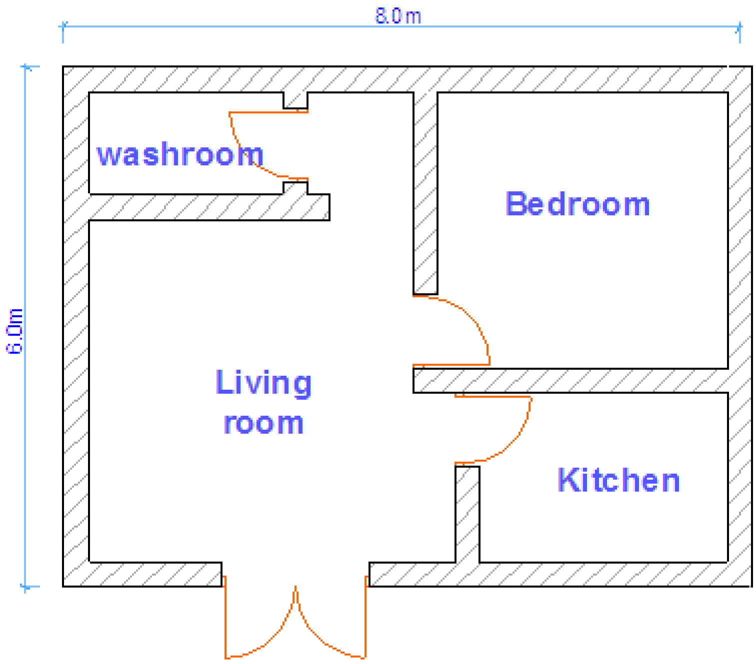


Figure 6.152: Plan of a typical dwelling unit

The cost of the construction at market rates of between KES 20,000 to KES 25,000 per square metre of the plan area gives a minimum cost of the construction being KES $20{,}000/m^2 * 48m^2$ = KES 960,000.00

Using the 4D technology, the units required are determined by actual count of elements and by adopting the unit rates determined earlier, a summary of the cost of the construction whose plan is given on *figure 6.1* is in the *table 6.11*.

Table 6.11: Summary of cost estimate for a typical dwelling unit

COMPONENT	QUANTITY	UNIT COST	AMOUNT
MOULDS & TOOLS			23,460
CONNECTOR CUBES	36	150	5,400
BEAMS	36	800	28,800
COLUMNS	72	800	57,600
SLAB PANELS	232	1,000	232,000
FLOOR, ROOFING, FITTINGS & FINISHES	48	7,500	360,000
HOUSE ASSEMBLY COST	120	500	60,000
Add 10% Contingencies			76,726
	TOTAL COST		**843,986.00**

6.2.11 Comment

The cost of the pilot house construction, *table 6.9* is observed to be about 30% higher than the cost of a typical construction of the same size made using masonry brick and cement mortar, *table 6.10*. This cost difference is associated with the fact that there is some built in flexibility in the design and construction using precast elements which permits future alteration of design and reuse of components in new configurations and also the reduction of wastes associated with the alteration of design or demolition of constructions. The cost of these considerations are not quantified at this stage and for the case of mass production or repetitive precasting, the initial cost of moulds can be neglected to bring down the cost of the technology.

Further comparison of the cost of construction of a 48m^2 dwelling unit under conventional brick and mortar technology and the 4D precast concrete technology shows that the 4D technology becomes cheaper with mass production. In fact it is about 10% cheaper for the 48m^2 dwelling unit as shown in *chapter 6.2.10.*

CHAPTER 7: CONCLUSIONS AND RECOMMENDATIONS

7.1 Conclusions

With the design, construction and analysis of the 4D precast concrete pilot house, the following can be deduced from the concepts, the development of elements, the assembly of model house and the outcomes of the structural analysis:

1. Precast concrete can be used to introduce the 4D design principles of construction on to concrete works where basic elements are developed and assembled allowing for adaptability of constructions to changing user needs and reconfiguration or redesign of the construction with no or minimal wastes.
2. The precast concrete elements can be manufactured and used as components of a whole structure or for partitioning of building spaces and in non-storey constructions.
3. The possible addition and removal of elements in an assembled precast concrete structure allows adoption of stage construction techniques hence as funds become available a structure can be extended or redesigned. This is the typical way of construction among the low and medium income groups.
4. The aspect of reuse of elements and reconfiguration to suit new demands and the fact that disassembly can be done without wastes ensures environmental sustainability, cost efficiency and conservation of raw materials that can go into development of new components.
5. Quality control and standardization of the precast process is possible with this kind of factory production of elements and this also minimizes site works hence

improves on site tidiness, construction time saving and builds sustainability into the concrete industry.

7.2 Recommendations

1. Monitoring of the pilot structure over time should be done to observe the impact of the environment and the repetitive loading due to wind and rain and other natural causes on the overall structure, the connections and serviceability of the construction.
2. The connections provided by the connector cube between the beams and columns and that of the slab panels to the columns should be locally analysed in more detail and supported by simulation and experiments to establish the optimal length of anchors/hooks and the ultimate withdrawal capacity of the attachments. Alternative connection methods such as wedged connections should be investigated.
3. The structural analysis has been done for only a limited loading scenario, further study can be done on loadings such as vibratory, impact and adverse horizontal loads on the construction.
4. This research adopted double panels system that has a gap of about 30mm between panels. This gap is envisaged to be useful for placing thermal and acoustic insulation materials and for passing services such as electrical and other pipe works. Further, research on use of single panels with light-weight core, and an investigation of the aesthetic aspects and their possible structural implication may also be of interest.
5. Strength development of precast concrete elements should be investigated experimentally by applying the weighted maturity method discussed in brief in *chapter 2* so as to develop the time, temperature and strength relationships to

determine critical values of time lapse and strengths to be attained before demoulding, lifting, transportation and use of the concrete elements.

6. The cost of the super-plasticizer used in this research is observed to be high; it is therefore recommended that an alternative plasticizer that is cheaper and/or locally available be considered to further lower the cost of SCC and its products.

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APPENDICES

Appendix 1: Determination of weights of elements

1. **WEIGHT OF SLAB PANELS**

Concrete volume: length*width*thickness

$1.024\ m * 0.512\ m * 0.024\ m = 1.2583*10^{-2}\ m^3$

Concrete density $2400 kg/m^3$ (average value from cube test)

Weight of concrete = volume* density *gravity constant

Weight of concrete = $1.2583*10^{-2}\ m^3 * 2400 kg/m^3 * 9.81 N/kg$

Weight of concrete in panels is 296.25N

Steel wire mesh: diameter of wire = 0.003m and total length is 10.24m

Steel volume = pie*radius*radius*length

$= 3.142*0.0015m*0.0015m*10.24m = 7.23823*10^{-5}\ m^3$

Anchor Plate: plate thickness is 3mm, width is 0.08m and total length is 0.96m

Steel volume = $0.003m*0.080m*0.96m = 2.304*10^{-4}\ m^3$

Plate hooks: rod diameter is 0.004m and total length about 1.44m

Steel volume = pie*radius*radius*length

$= 3.142*0.002m*0.002m*1.44m = 1.81*10^{-5}\ m^3$

Total steel volume for wire mesh, plate and hooks is: $3.21323*10^{-4}\ m^3$

Given steel density is $7700kg/m^3$ (typical value adopted), the weight of steel is given by:

Weight of steel = steel volume* steel density *gravity constant

$= 3.21323*10^{-4}\ m^3 * 7700kg/m^3 * 9.81N/kg$

Weight of steel in panels 24.27N

Total weight of panels

Weight of concrete + weight of steel = 296.25N + 24.27N = **320.52N**

2. WEIGHT OF BEAMS/COLUMNS

Concrete volume: length*width*thickness

$= 1.024m*0.08m*0.08m = 6.5536*10^{-3}\ m^3$

Concrete density $2400kg/m^3$

Weight of concrete = volume* density *gravity constant

$= 6.5536*10^{-3}\ m^3 * 2400kg/m^3 * 9.81N/Kg$

Weight of concrete 154.30N

Steel reinforcement: steel volume = 4*pie*radius*radius*length

$= 4 * 3.142 * 0.003m * 0.003m * 0.95m = 1.0744 * 10^{-4}\ m^3$

Endplates: steel volume = thickness*width*length

$= 0.003 * 0.080 * 0.24 = 5.76 * 10^{-5}\ m^3$

Total steel volume $1.650 * 10^{-4}\ m^3$

Density of steel $7700 kg/m^3$

Weight of steel = steel volume* steel density *gravity constant

Weight of steel = $1.650 * 10^{-4}\ m^3 * 7700 kg/m^3 * 9.81 N/kg$

= 12.46N

Total weight of beams/columns = Weight of concrete + Weight of steel

= 154.30N + 12.46N = **166.76N**

3. **WEIGHT OF CONNECTOR CUBES**

Connector cubes: cubic of sides 0.08m

Concrete volume: $0.08m * 0.08m * 0.08m = 5.12 * 10^{-4}\ m^3$ (less volume of voids due to nuts inserted $2.18 * 10^{-5} m^3$) = $4.902 * 10^{-4} m^3$

Density of concrete $2400 kg/m^3$

Weight of concrete = volume* density *gravity constant

Weight of concrete = $4.902 * 10^{-4}\ m^3 * 2400 kg/m^3 * 9.81 N/Kg$

Weight of concrete 11.54N

Approximation for weight of steel: one nut weighs is 0.017Kg, (or 0.017Kg*9.81N/Kg) = 0.16677N; therefore for the 12 nuts in the connector cube approximate to 12*0.16677N = 2N

Total weight of connector cube: 11.54N + 2N = **13.54N**

Appendix 2: Design for roof loads

Whilst the house model constructed is 2288mm by 4496mm a shown in *chapter 3, figure 3.20*, the design and calculations checks are done for a 3392mm by 4496mm model as shown in the floor plan in *figure A1*so as to check the critical free span of 3392mm as recommended in span/stage construction checks in *chapter 5.3*. The roof area acted upon by the imposed load is 2.248m*0.918m hence an imposed point load of 1.55kN on an interior node and an additional 0.86kN as dead load.

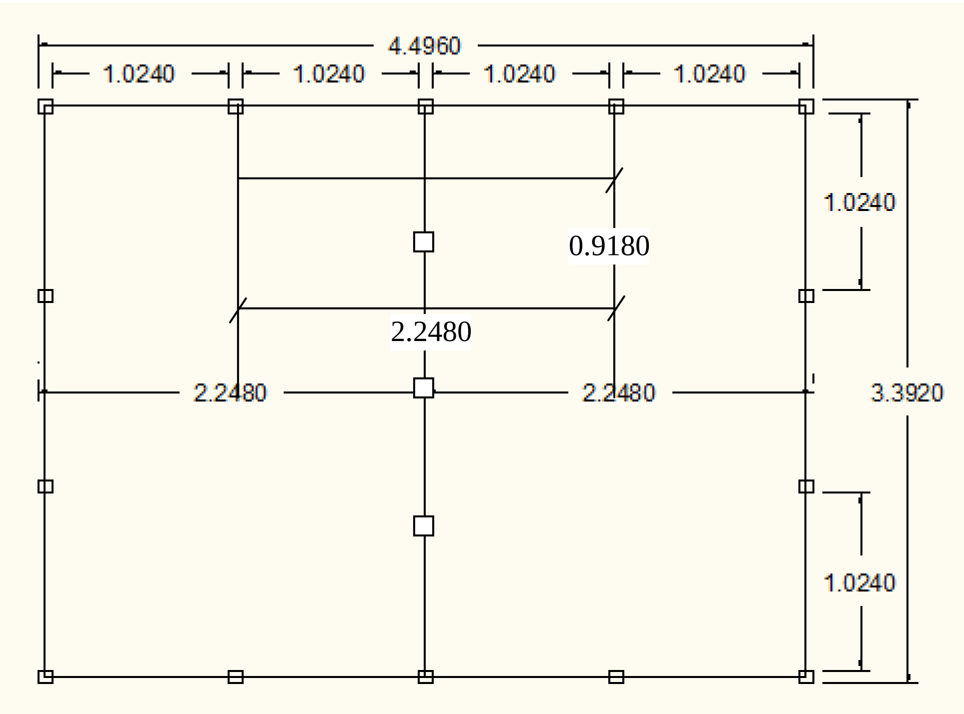


Figure A1: Plan representation of the structure

The three trusses span the shorter edge, 3.392m at spacing of 2.248m

Truss considered for preliminary design is the middle truss since it carries higher loads and hence the column to carry higher loads is by inspection either of the columns supporting this truss and is thus the column to be designed.

Design load on an interior node of the truss is determined, exterior nodes carry half as much. With this, the support loads of the truss is determined and used for further design of the column elements.

Load combination 1:

Design load = 1.4*Dead load + 1.6*Live loads

Dead load kN = $(0.2kN/m^2*2.248m*0.918m) + (0.2kN/m*2.248m)$

= 0.86kN

Live loads kN = $(0.75kN/m^2*2.248m*0.918m)$

= 1.55kN

Design load kN = 1.4*0.86kN + 1.6*1.55kN

= 3.7kN

Interior nodes of the trusses therefore carry 3.7kN and the exterior nodes carry 1.85kN as in the illustration *figure A2*. The support loads are equal to 7.4kN with a factor of safety of 1.5 yields a design load of about 11.1kN and are transferred to the columns as vertical loads.

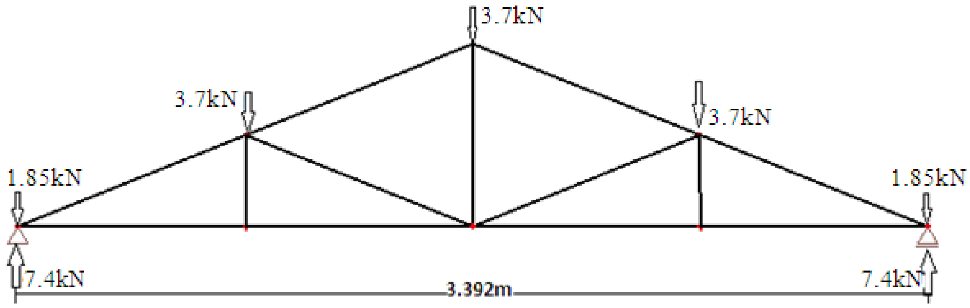


Figure A2: Sketch of the truss and loads for design

Appendix 3: Design for reinforcement in beams/columns

Design of the beams

The design is according to *BS 8110, part 1; section 3.4*

Length 1000mm, cross-section 80 mm*80 mm

Load on the beam constitute the self-weight only thus factored load is1.4*Dead load

Self-weight: 0.17kN/m; load factor: 1.4,

Design load, w = 0.24kN/m

Design moment at mid span of a simply supported beam, $M_u = wl^2/8$

M_u = 0.030kNm and factored design moment is $M_n = M_u/0.9$ = 0.0333kNm

Beam moment diagram and the beam shear force diagram are illustrated in *figure A3*.

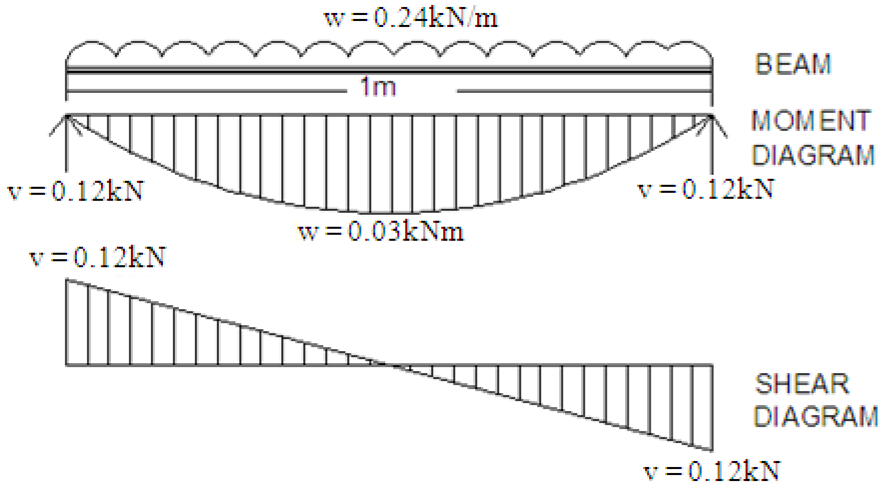


Figure A3: Beam moment and shear force diagram

The effective depth *d,* is the beam depth *h*, less the concrete cover of 15mm and less half the reinforcement 4mm (assume Y8 reinforcement in initial design):

d = 80mm – 15mm – 4mm = 61mm

Section 3.4.4.4: Reinforcement requirements

Moment redistribution factor, $K = M_n/bd^2 f_{cu}$

K = 33330/ (80*61^2*20) = 0.00598

Given K' =0.156, factor for up to 10% redistribution, *K<K' hence compression reinforcement is not required.*

$z = d$ {0.5+√ (0.25-K/0.9)}

= 61*{0.5+√ (0.25-0.00598/0.9)}

= 60.6mm

Therefore z = the lesser of {z = 60.6mm} and {z=0.95d (= 57.95mm)}; thus z = 57.95mm

Reinforcement area A_s, required is therefore:

$A_s = M_n/0.95 f_y z$

= 33330/0.95*460*57.95

= 1.316 mm^2

The reinforcement provided of 4Y8, A_s = 201.0 mm^2 exceeds the $A_{s\,req}$ for the section. Check for maximum reinforcement in cross-section: (201 mm^2/ 6400 mm^2)*100 = 3.14% which is less than maximum allowable 4% hence A_s provided is permissible. It is however recommended that the higher values from the design of columns be adopted since beams and columns are made to be interchangeable.

Shear stress in the beam elements

Section 3.4.5.2, BS 8110

Shear stress is given by: $v = V/bd$ where $v = wl/2$ for simply beam under distributed load w.

Shear force; V = 0.24kN * 1/2 = 0.12kN

Shear stress; v = 0.12kN/(80mm*61mm) = 0.0246N/mm^2

Requirement: v should not exceed $0.8\sqrt{f_{cu}}$ (= 3.58 N/mm^2) therefore OK

Shear reinforcement

Design shear in concrete, v_c,

$v_c = 0.79\{ (100A_s)/bd\}^{1/3} (400/d)^{1/4}\}/1.25$

given: $100\ A_s/bd = 7850/(80*61) = 1.6$

$v_c = 0.79\{ (1.6\}^{1/3} (400/61)^{1/4}\}/1.25 = 1.183$ N/mm^2 and $0.5\ v_c = 0.59$N/mm^2

Since v (=0.0246N/mm^2) < 0.5 v_c (=0.59N/mm^2) the shear reinforcement is not required as in *table A1*.

Table A1: Table of Shear reinforcement for beams: Table 3.7 of BS 8110

Value of v N/mm^2	Form of shear reinforcement to be provided	Area of shear reinforcement to be provided
Less than $0.5v_c$ throughout the beam	See NOTE 1	—
$0.5v_c < v < (v_c + 0.4)$	Minimum links for whole length of beam	$A_{sv} \geq 0.4b_v s_v/0.95f_{yv}$ (see NOTE 2)
$(v_c + 0.4) < v < 0.8\sqrt{f_{cu}}$ or 5 N/mm^2	Links or links combined with bent-up bars. Not more than 50 % of the shear resistance provided by the steel may be in the form of bent-up bars (see NOTE 3)	Where links only provided: $A_{sv} \geq b_v s_v(v - v_c)/0.95f_{yv}$ Where links and bent-up bars provided: see 3.4.5.6
NOTE 1 While minimum links should be provided in all beams of structural importance, it will be satisfactory to omit them in members of minor structural importance such as lintels or where the maximum design shear stress is less than half v_c.		
NOTE 2 Minimum links provide a design shear resistance of 0.4 N/mm^2.		
NOTE 3 See 3.4.5.5 for guidance on spacing of links and bent-up bars.		

Deflection Check

Deflection in the beams is limited to a *span/effective depth* ratio of 20 for simply supported beams as in the *table A2* for rectangular beams. For this case, span depth ratio is 1000/61 = 16.39 < 20, therefore OK

Table A21: Table of Basic span/effective depth ratios: Table 3.9 of BS 8110

Support conditions	**Rectangular section**	**Flanged beams with b_w/b <= 0.3**
Cantilever	7	5.6
Simply supported	20	16.0
Continuous	26	20.8

Design of the columns

The column is designed according to *section 3.8.2.3* of the *BS 8110* the axial force in a column is calculated on the assumption that all beams and panels transmit force in a simply supported manner and bending moment due to horizontal load excluded in the hand calculation. The ultimate design axial load is considered only together with a design moment representing nominal allowance for eccentricity.

The axial load for a maximally loaded column includes:

32 slab panel load: 32*320N = 10240N (= 10.24kN)

8 beams load: 8*167N = 1336N (= 1.336kN)

2 columns load; 2*167N = 334N (= 0.334kN)

Total dead load is thus: 11.91kN; factoring by 1.4; design dead load is thus: 16.67kN

Addition of roof load of 11.1kN gives a total design load of 27.77kN.

Eccentricity: Minimum eccentricity, *e-min* is equal to 0.05b or 0.05d and not greater than 20mm according to *clause 3.8.2.4*, and the resulting moment is therefore:

Ne-min = 27.77kN*0.05*80/1000 = 0.11kNm

Moment induced by deflection of the column is also taken into account by a factor $\alpha u = \beta aKh$; where $\beta a = (l_e/b)^2/2000$ and h = b

K is a reduction factor that corrects the deflection to allow for effect of axial load, it is conservatively taken as 1

$\beta a = (1000/80)^2 /2000 = 0.078$

$\alpha u = 0.078*1*80 = 6.25\text{mm}$

The additional moment due to deflection is thus $M_{add} = N\,\alpha u$: N is the applied axial load approximated as 27.77kN

$$M_{add} = N\,\alpha u = 27.77\text{kN}*(6.25/1000)\ \text{m} = 0.1735\text{kNm}$$

From *figure A4*, the initial moments are zero; the design moment is the greater of *Ne-min* and the M_{add}, and as such the M_{add} = 0.1735kNm is considered for design.

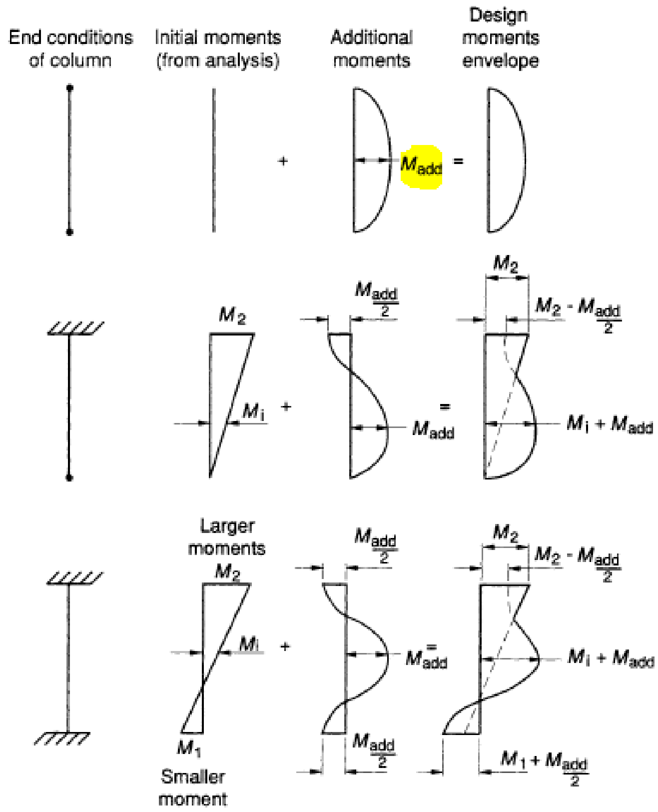


Figure 3.20 — Braced slender columns

Figure A4: Moment considerations in column design

The required reinforcement is derived from the *design charts in part 3 of the BS 8110*; *figure A5*, which gives reinforcement areas in relation to the properties given. These include f_{cu} = 20N/mm^2, f_y = 460N/mm^2, d/h = 0.95 and relates N/bh to M/bd^2.

Where; N/bh = 27770N/ (80mm*80mm) = 4.339N/mm^2 and,

M/bd^2 = 173500Nmm/ (80mm*80mm*80mm) = 0.3389N/mm^2

With the above values and *figure A5*, the minimum reinforcement ratio $100A_s/bh = 0.4$,

Thus A_s = 0.4*bh/100 = 25.6mm^2

The A_s provided by 4Y8 is 201mm^2 which is greater than A_s required indicating an over design. 4Y6 is recommended which gives A_s of 113mm^2. This yields reinforcement per cross-section of 113mm^2/6400mm^2 = 1.76%. The maximum reinforcement allowance of 4% of cross-sectional area is therefore preserved.

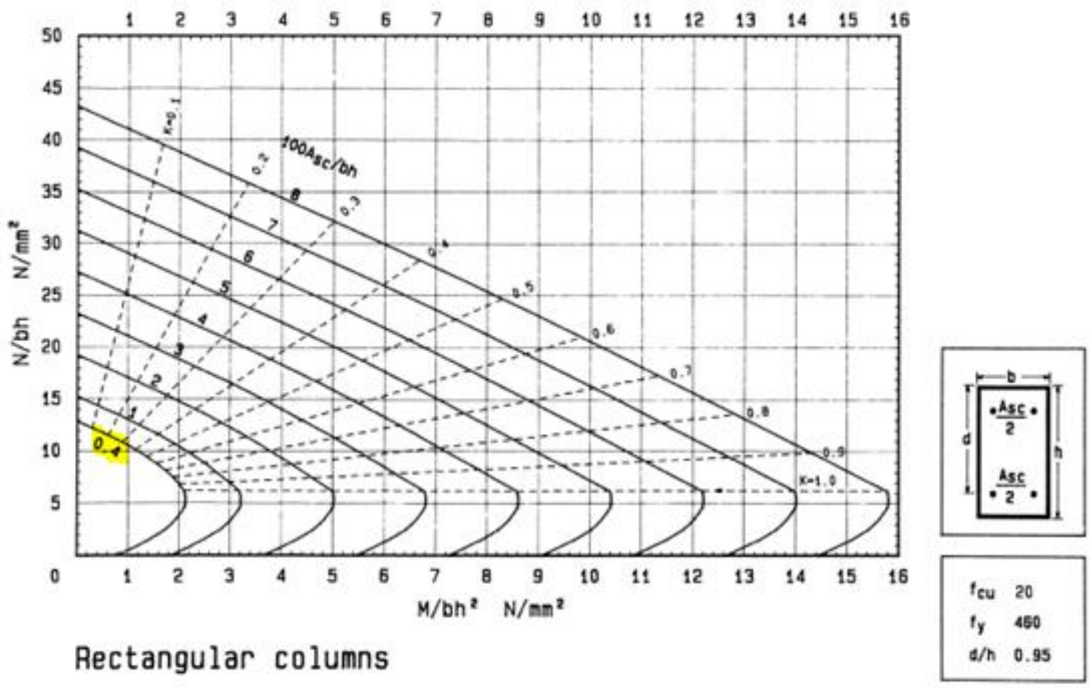


Figure A5: Chart for Reinforcement areas for columns

Shear check in the column

Clause 3.4.5.12, shear and axial load in columns

The design shear stress v_c' for a section subject to both axial and shear forces without shear reinforcement is calculate by the formula;

$v_c' = v_c + 0.6\ NVh/A_cM$

Where: $v_c = 0.79\{(100A_s)/bd\}^{1/3}\ (400/d)^{1/4}\}/1.25$

$100\ A_s/bd = 15700\text{mm}^2/(80\text{mm}*80\text{mm}) = 2.453$

$v_c = 0.79\{\ (2.453\}^{1/3}\ (400/80)^{1/4}\}/1.25 = 1.275\ \text{N/mm}^2$

V is the shear, horizontal load applied, taken to be 8kN, the resultant of wind loads on panels

Moment on the column, M = 0.1735kNm

A_c is the gross area of concrete

With Vh/M conservatively taken as 1, as in *note 2* of the clause and N/A_c = 4.339 N/mm^2

$v_c' = 1.275 + 0.6 * 4.339 = 3.8784\text{N/mm}^2$

$v = V/bd = 8000/\ (80*80) = 1.25\text{N/mm}^2$

Clause 3.8.4.6: Shear in compression elements;

Since; v (= 1.25N/mm^2) is less than v_c' (= 3.8784N/mm^2) and also less than 0.8 $\sqrt{f_{cu}}$ (= 3.58 N/mm^2) and M/N (= 0.1735kNm/27.77kN = 0.00625m) < 0.6h {(= 0.6*80mm*0.001m/mm) = 0.048m} therefore only minimum shear reinforcement is required and is provided.

Appendix 4: Determination of wind load

i. Determination of the external and internal pressure coefficients

The external pressure coefficients (*Cpe*) are determined as in *section 2.5.2.4* for which factors are given in *table A3* that relates to illustration in *figure A6* and the geometrics of the structure. It also relates to the two directions of wind and various roof pitches. The roof pitch considered in this case is 22.5^{o}.

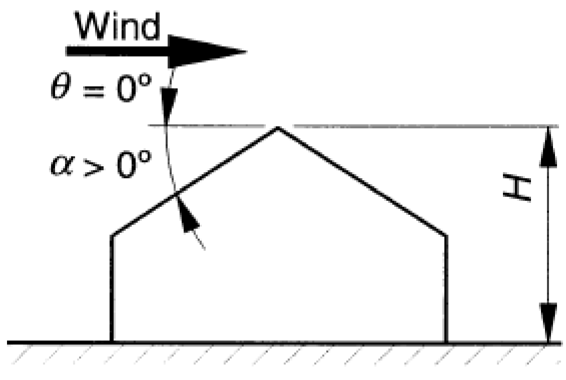


Figure A6: Wind action on pitched roofs

Figure A6 shows roof pitch $\alpha = 22.5^{o}$, and *figures A7*and *A8* show the different zoning of the roof for the respective values of pressure coefficients, *table A3.*

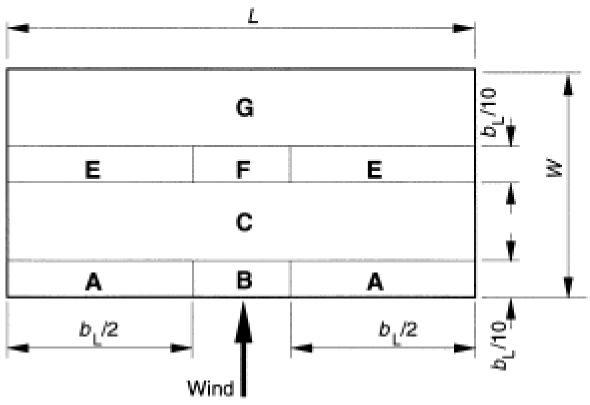


Figure A7: Roof zoning in wind design

Section 2.5.2.2 zone dimensioning; L = 4.496m, H=3.3m, bl=L since L< 2H hence;

bl/10 = 0.45m bl/2 = 2.248m

Table A3: Wind pressure coefficients: Table 10 of BS6399

Pitch angle α	Zone for $\theta = 0°$						Zone for $\theta = 90°$			
	A	B	C	E	F	G	A	B	C	D
–45°	–0.9	–0.8	–0.9	–1.1	–0.7	–0.7	–1.5	–1.3	–1.0	–0.9
–30°	–1.7	–1.0	–0.9	–0.8	–0.7	–0.7	–1.7	–1.3	–1.0	–0.8
–15°	–2.6	–1.0	–0.9	–0.7	–0.5	–0.5	–2.6	–1.4	–0.8	–0.8
–5°	–2.4	–1.2	–0.8	–0.5	–0.3	–0.5	–2.2	–1.5	–0.7	–0.7
+5°	–1.8	–1.2	–0.6	–0.9	–0.3	–0.4	–2.0	–1.1	–0.6	–0.5
	+0.0	+0.0	+0.0	–0.9	–0.3	–0.4				
+15°	–1.1	–0.8	–0.4	–1.3	–0.9	–0.5	–1.6	–1.5	–0.6	–0.4
	+0.2	+0.2	+0.2	–1.3	–0.9	–0.5				
+30°	–0.5	–0.5	–0.2	–0.9	–0.5	–0.5	–1.2	–1.1	–0.6	–0.5
	+0.8	+0.5	+0.4	–0.9	–0.5	–0.5				
+45°	–0.0	–0.0	–0.0	–0.4	–0.3	–0.3	–1.2	–1.2	–0.6	–0.4
	+0.8	+0.6	+0.7	–0.4	–0.3	–0.3				
+60°	+0.8	+0.8	+0.8	–0.8	–0.7	–0.6	–1.2	–1.2	–0.7	–0.6
+75°	+0.8	+0.8	+0.8	–0.9	–0.6	–0.8	–1.2	–1.2	–1.15	–0.6

NOTE 1 At $\theta = 0°$ the pressure changes rapidly between positive and negative values in the range of pitch angles $+5° < \alpha < +45°$. Two sets of values are given at these pitch angles and they should be treated as separate load cases.

NOTE 2 Interpolation for intermediate pitch angles may be used between values with the same sign. Between pitch angles +5° and –5° interpolation is not permitted and the data for flat roofs in 2.5.1 should be used instead.

Interpolation is done for values of pitch angle 22.5^{o} and gives the following:

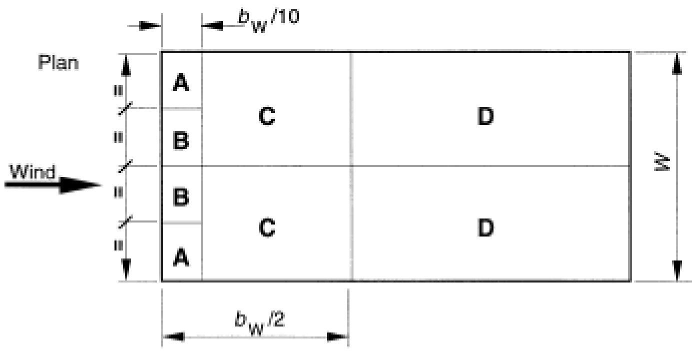


Figure A8: Wind zoning in wind design

W= 3.392m. $b_w = W$ since $W<2H$= 6.6m hence $b_w/10$ = 0.3392m

Internal pressure coefficient (*Cpi*) is taken as the most onerous of -0.3 and +0.2. A summary of the determination of net pressures are shown in *table A4*.

Table A4: Summary of determination of zonal net wind pressures

		15^o	**22.5^o**	**30^o**	***Cpe - Cpi***	***Cpe – Cpi***	**Net pressure (KN/m^2)**	**Net pressure (KN/m^2)**
0^o					*Cp*i = +0.2	*Cp*i= -0.3	*Cp*i=+0.2	*Cp*i= -0.3
	A	-1.1	-0.8	-0.5	-1	-0.5	-1.38	-0.69
		0.2	0.5	0.8	0.3	0.8	0.414	1.104
	B	-0.8	-0.65	-0.5	-0.85	-0.35	-1.173	-0.483
		0.2	0.35	0.5	0.15	0.65	0.207	0.897
	C	-0.4	-0.3	-0.2	-0.5	0	-0.69	0
		0.2	0.3	0.4	0.1	0.6	0.138	0.828
	D	-	-	-				
	E	-1.3	-1.1	-0.9	-1.3	-0.8	-1.794	-1.104
		-1.3	-1.1	-0.9	-1.3	-0.8	-1.794	-1.104
	F	-0.9	-0.7	-0.5	-0.9	-0.4	-1.242	-0.552
		-0.9	-0.7	-0.5	-0.9	-0.4	-1.242	-0.552
	G	-0.5	-0.5	-0.5	-0.7	-0.2	-0.966	-0.276
		-0.5	-0.5	-0.5	-0.7	-0.2	-0.966	-0.276
90^o								
	A	-1.6	-1.4	-1.2	-1.6	-1.1	-2.208	-1.518
	B	-1.5	-1.3	-1.1	-1.5	-1	-2.07	-1.38
	C	-0.6	-0.6	-0.6	-0.8	-0.3	-1.104	-0.414
	D	-0.4	-0.45	-0.5	-0.65	-0.15	-0.897	-0.207

The net pressure – the last two columns of the *table A4* – is given by: *NP = (Cpe – Cpi)*dynamic pressure**C_a, C_a is the factor for the non-simultaneous action of the gust wind across the external surface of the structure, most severe case is when $C_a = 1$

The wind pressures derived above are acting perpendicular to the roof and are so applied on the analysis of the construction introducing both effects of vertical and horizontal effects.

ii. Wind loading on the panels/walls

Wind loads on the panel constitute the horizontal force that the structure has to withstand via the lateral stiffening of the panels and the connections. The panels are the main elements that take up the lateral loads.

Wind on the wall is designed according to the *BS 6399 Part 2, Code of Practice for wind loads*. The wind is regarded to be acting at 0^o when acting toward the crosswind breadth of 4.496m and at 90^o when acting towards the shorter length of 3.392m.

External pressure coefficient *(Cpe)* for the wall is described in *section 2.4*, for rectangular plan buildings. This is dependent on the ratio of dimensions described in *figure A1*. The ratio *D/H* for case of wind along the short face is equal to *W/H* = 3.392/3.5 approximating to 1 and along the longer face of *D*= 4.496m, the ratio is 1.3, for the case considered as isolated, *table A5* gives values for windward side as +0.85 and -0.5 for leeward side.

Table A5: Table for the Cpe values for vertical walls: Table 5 of the BS6399

Vertical wall face	Span ratio of building		Vertical wall face		Exposure case	
	$D/H \leq 1$	$D/H \geq 4$			Isolated	Funnelling
Windward (front)	+0.85	+0.6	Side	Zone A	–1.3	–1.6
Leeward (rear)	–0.5	–0.5		Zone B	–0.8	–0.9
				Zone C	–0.5	–0.9
NOTE Interpolation may be used in the range $1 < D/H < 4$. See 2.4.1.4 for interpolation between isolated and funnelling.						

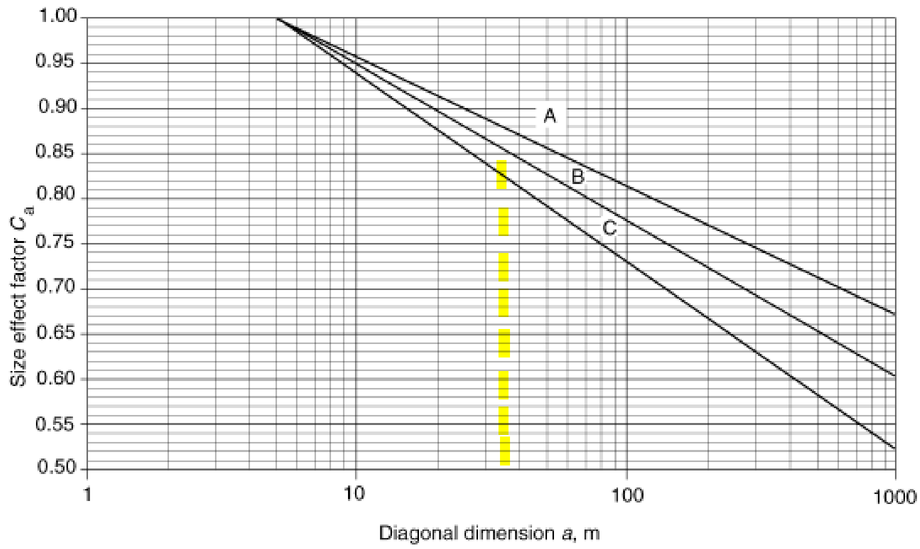
Internal pressure coefficient *(Cpi)* according to *table A6*, the values are +0.2 for wind normal to permeable face and –0.3 for wind normal to impermeable face for cases where two opposite walls are equally permeable while other faces are impermeable.

Table A6: Table for the Cpi values for enclose buildings: Table 16 of the BS 6399

Type of walls	C_{pi}
Two opposite walls equally permeable; other faces impermeable	
- Wind normal to permeable face	+0.2
- Wind normal to impermeable face	-0.3
Four walls equally permeable ; roof impermeable	-0.3

The diagonal dimension, α is a factor of the internal volume given as 10*cubic root of Internal Volume. This give a value of $\alpha = 10 * \sqrt[3]{(Internal\, Volume)} = 10 * \sqrt[3]{45.75} = 35$

Dynamic pressure is given by $0.613V_e^2$ and the V_e is the effective wind speed. The Net pressure on the surfaces is a factor of the internal and the external pressures, and is given as *P = (Pe –Pi)*, where *Pe* and *Pi* are given by multiplying the dynamic pressure by the respective pressure coefficients and the factor for size effect C_a. the C_a from *figure A9* and the diagonal dimension determined above is 0.83, a more critical case is considered as C_a =1.



Key to lines on Figure 4							
Effective height H_e	Site in country: closest distance to sea (km)				Site in town: closest distance to sea (km)		
m	0 to < 2	2 to < 10	10 to < 100	≥ 100	2 to < 10	10 to < 100	≥ 100
≤ 2	A	B	B	B	C	C	C
> 2 to 5	A	B	B	B	C	C	C
> 5 to 10	A	A	B	B	A	C	C
> 10 to 15	A	A	B	B	A	B	B
> 15 to 20	A	A	B	B	A	B	B
> 20 to 30	A	A	A	B	A	A	B
> 30 to 50	A	A	A	B	A	A	B
> 50	A	A	A	B	A	A	B

Figure 4 — Size effect factor C_a of standard method

Figure A9: Wind C_a factor using standard method: Figure 4 of the BS6399

Appendix 5: Expanded model

Model structure expanded to 4m by 4m in dimensions is shown in *figure A10*. The structure has a partition a shown and is analysed under the same conditions as the pilot house to demonstrate the effect of increasing the unsupported free span.

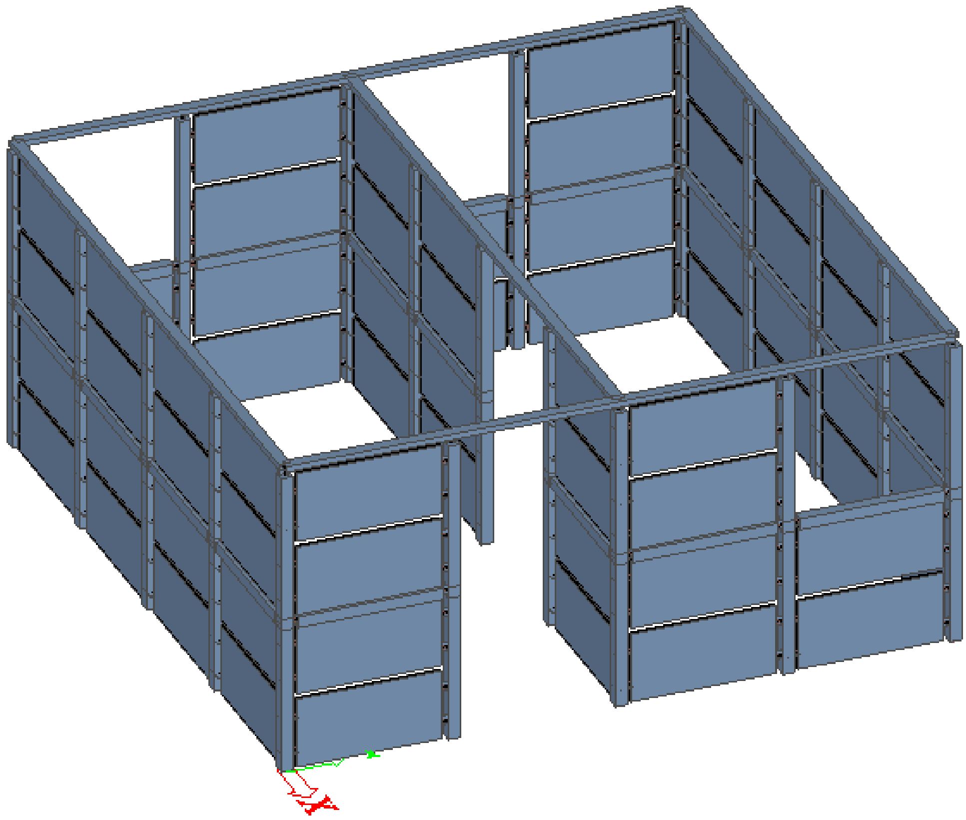


Figure A10: Illustration of an expanded model

Analysis results

The structure has a more pronounced deflection in the y-direction due to wind loading from this direction as shown in *figure A11.* A deflection value of 8.8mm is exhibited at the top of the mid-span. For the span of 4000mm, the deflection limit is (4000mm/500 =) 8mm which is exceeded by the value exhibited by the structure due to the long unsupported span.

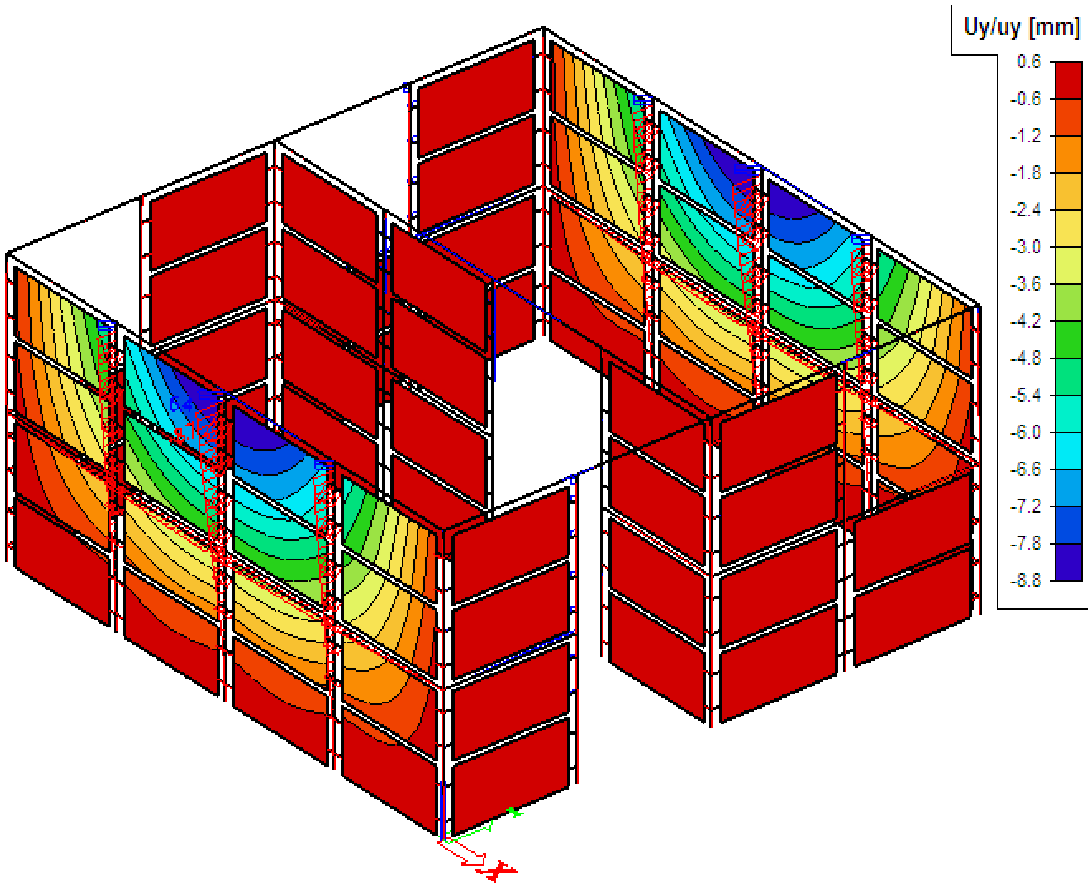


Figure A11: Deflection of model in figure A1 in the y-direction

Appendix 6: Partitions introduced in the model in appendix 5

Model structure shows a wall free span reduced to 2m due to introduction of partition walls, *figure A12*. This model is analysed as above.

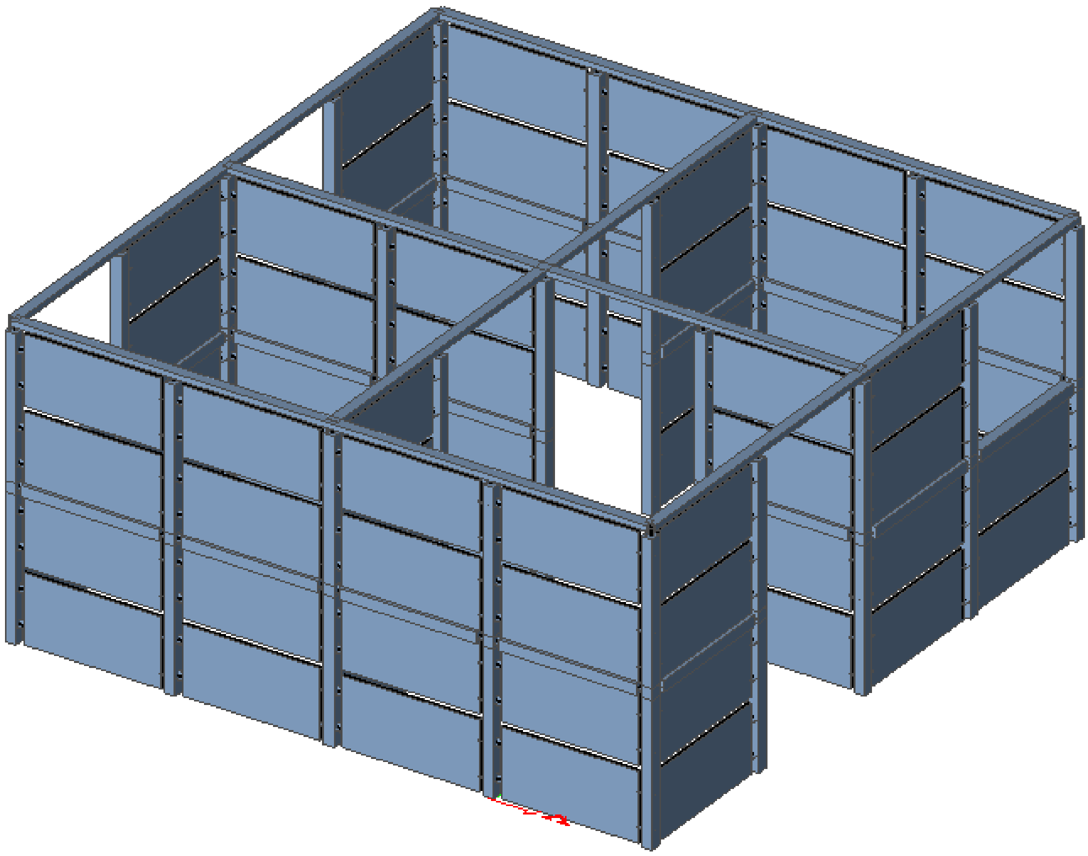


Figure A12: Partitions introduced in model appendix 5

Analysis results

The deflection in the y-direction reduces to a value of 1.5mm, *figure A13*. Compared to the limit, (2000mm/500 =) 4mm, the structure is safe against wind loading from all directions. This demonstrates that the partitions (or short spans) act favourably to reduce susceptibility of the structure to horizontal loads.

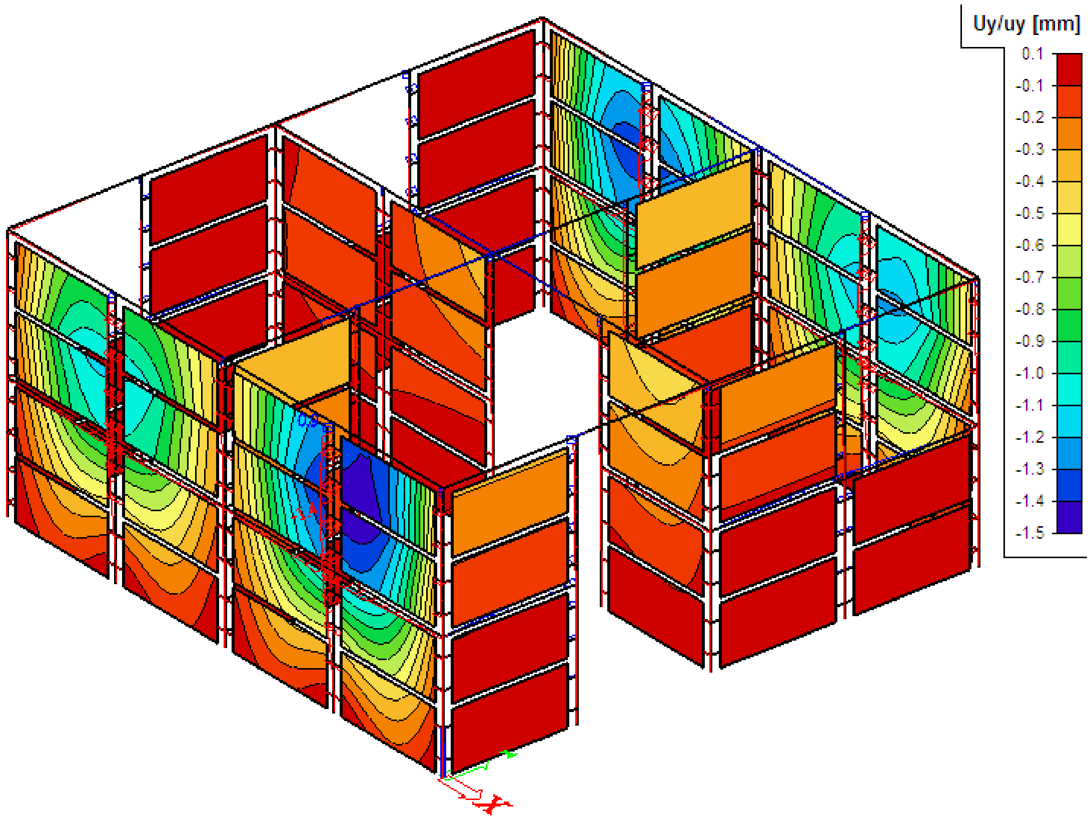


Figure A13: Deflection of structure in figure 110 due to wind loading in y-direction

Appendix 7: Derivation of machines hourly cost estimates

Derivations of machine hourly costs are given in *tables A7*, *A8* and *A9* for respective machines.

a. Gravity tilting mixer

Table A7: Derivation of hourly cost of gravity tilting mixer

Capital cost of equipment: KES. 193,560.00			
Expected useful life: 10 years			
Assume yearly working at 75%			
Yearly working hours	50weeks * 40hours * 75%	1,500 hours per year	
Assuming a resale value after 5 years of KES 50000			
Annual depreciation = (cost – resale)/useful life		KES	14,356
Hourly depreciation = Annual depreciation/1500 hours		KES	9.57
Net hourly cost = Hourly depreciation		KES	9.57
Add 10% of net hourly cost for insurance		KES	0.957
Add 20% of net hourly cost for maintenance		KES	1.914
Fuel cost per hour		KES	100
Total cost per hour		**KES**	**112.44**

b. Weighing machine

Table A8: Derivation of hourly cost of weighing machine

Capital cost of plant KES 207,480.00
Expected useful life 10 years

Assume yearly working at 75%			
Yearly working hours	50weeks*40hours*75%	1,500 hours per year	
Assume a resale value after 5 years		KES	60,000
Annual depreciation = (cost – resale)/useful life		KES	14,748
Hourly depreciation = Annual depreciation/1500 hours		KES	9.8
Net hourly cost = hourly depreciation		KES	9.8
Add 10% of net hourly cost for insurance		KES	0.98
Add 20% of net hourly cost for maintenance		KES	1.96
Total cost per hour		**KES**	**12.74**

c. **Compression testing machine**

Table A9: Derivation of hourly cost of compression testing machine

Capital cost of plant KES 549,285.00			
Expected useful life 20 years			
Assume yearly working at 75%			
Yearly working hours	50weeks*40hours*75%	1,500 hours per year	
Assume a resale value after 5 years		KES	200,000
Annual depreciation = (cost – resale)/useful life		KES	17,464.25
Hourly depreciation = Annual Depreciation/1500 hours		KES	11.64
Net hourly cost = hourly depreciation		KES	11.64
Add 10% of net hourly cost for insurance		KES	1.164
Add 20% of net hourly cost for maintenance		KES	2.33
Total cost per hour		**KES**	**15.13**