FEASIBILITY STUDY ON POLYURATHENE SANDWICH PANEL FOR DOMESTIC CONSTRUCTION

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Degree of Master of Engineering in Structural Engineering Designs

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Sri Lanka

March 2016

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Thesis submitted in partial fulfilment of the requirements for the degree of Master of Engineering in Structural Engineering Designs

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DECLARATION

"I declare that this is my own work and this thesis does not incorporate without acknowledgement any material previously submitted for a Degree or Diploma in any other University or Institute of higher learning and to the best of my knowledge and belief it does not contain any material previously published or written by other person except where the acknowledgement is made in the text.

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ACKNOWLEDGEMENT

I would like to express my deepest appreciation to all those who provided me the possibility to complete this thesis. A special gratitude with respect I give to my supervisor Dr. K.Baskaran, Department of Civil Engineering, University of Moratuwa, whose contribution in stimulating suggestions, encouragement, their gratitude and consent on supervision as well as for providing necessary information regarding the project.

I am also grateful to the course coordinator Dr.C.S.Lewangamage, teachers and the head of department of Civil Engineering at University of Moratuwa for all kind of support and help.

I would also gratitude to the staff of Structural laboratory Mrs Cooray, Mr. Nalinda, Mr. Leenus and Mr. Piyal for their needful help on material testing in September 2012.

Also I sincerely thank the Library staff of Sri Lanka Standard Institution for providing me reading material on American Standard Institution publications.

Finally my sincere gratitude goes to my family for their understanding and supports extended to me in completing this project successfully. I am in the opinion that the completion of this report would have been a nightmare without any of the support mentioned above.

Abstract

The place to live is the third need of mankind. Everybody try to build up a suitable mean to meet their own requirements. The Sri Lankan need always changes drastically after two decades from initial construction as the social and economical changes in the society. The use of none renewable material for short period may degrade the scarce resources. And also generate ample amount of green house gasses, which lead the global warming. Therefore in time to come, we have to switch to renewable material or reusing material. There are some materials, those are produced from garbage. This creates regenerative products on earth resource extraction circle. The polyurethane sandwich panel is a reusing material which is produced from garbage. This thesis is on feasibility study on polyurethane sandwich panel for domestic constructions. The product establishment is a derivation as a regenerative product to meet the human need of this scenario.

Additionally there is a shortage of skilled labour in the country. And the cost of labour for domestic construction is a considerable portion. The time consumed for domestic construction is more than months. Therefore by introduction of polyurethane sandwich panels for domestic constructions may resolve the major problems in the domestic construction field in the country.

The aim of this thesis is to introduce an engineered solution from polyurethane sandwich panel to aforesaid problems. The only drawback is the less fire rating. But currently produced materials meet the legislative and regulatory stipulations. The science and technology on this field is to be improved in time to come.

The sandwich panels are having very high stiffness compared to weight and a cost effective product. Polyurethane sandwich panel material may last more than two decades without much maintenance.

The polyurethane sandwich panels are used for the construction of walls and ceiling on the cold room constructions as a good thermal barrier. This thesis is to see the validity on cold room construction material for the domestic constructions.

The material properties changes from supplier to supplier. Therefore it is very difficult to adopt the standard practice in design. Even though “European Recommendations for Sandwich Panels Part 1; Design”;[14] has released on year 2000. The publication has been criticized by various researches such as Narayan Pokharel and Mahen Mahendran on their publication to “Thin Walled Structures” [13]. In addition the both published documents’ equation ranges on “European Recommendations for Sandwich Panels Part 1; Design” [14] and “Thin Walled Structures” [13] do not comply with the encountered polyurethane insulative sandwich panel. Therefore the serviceability limit published by “European Recommendations for Sandwich Panels Part 1; Design” [14] has been incorporated for design serviceability limit checking.

This thesis is on feasibility study of sandwich material for house constructions by means of walls, slabs and roofs. The typical two-story house and the two story cluster houses are modelled to see the engineering viability under standard loadings. The outcome revealed that the construction up to two stories is safe. Therefore further studies in this stream shall be followed in future. As per the project outcome on the clause 6.6; it reveals that the domestic constructions up to two stories may be possible under some form of local capacity enhancement methods adapted to high stresses applied locations.

The economical analysis is also made in Chapter five. Accordingly the cost on individual houses and cluster houses do not change and it revealed that there is more than 41% saving compared to the conventional constructions.

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NOTATIONS

Greek Symbols

α	-	Honeycomb cell size
δ	-	Length of contact between the central roller and the top skin or Length of top distributed-load
υ_{cxz}	-	Out of plane honeycomb Poisson’s ratio
υ_{fxy}	-	Poisson’s ratio of the skin material
ρ_c	-	Honeycomb core density
ρ_s	-	Honeycomb constituent material density
σ	-	Out of plane stress
σ_{cc}	-	Out of plane compressive strength of the honeycomb core
σ_{fi}	-	In plane compressive stress for intracellular buckling of the skin
σ_{fw}	-	In plane wrinkling strength of the skin
σ_{fy}	-	In plane yield strength of the skin
σ_{txx}, $\sigma_{txx\ i}$	-	In plane normal stress in the skin
σ_{txx}	-	Out of plane normal stresses in the core
τ_{31}, τ_{32},	-	Out of plane shear strength of the honeycomb for transverse and longitudinal directions.

Latin Symbols

A_t, A_b	-	In plane rigidity of top and bottom skin
b	-	Beam width
c	-	Core thickness
D	-	Flexural stiffness of the beam
D_t,D_b	-	Flexural rigidity of top and bottom skin
d	-	Distance between the mid planes of top and bottom skin
E_c or E_3	-	Out of plane Young's modulus of the honeycomb core
E_s	-	In plane Young's modulus of the honeycomb core in the x direction.
G_{31},G_{32}	-	Out of plane shear module of the honeycomb for transverse and Longitudinal ribbon direction respectively
G_s	-	Shear modulus of the honeycombs solid material
I	-	Second moment of area of the sandwich beam
I_f	-	Second moment of area of the skins with respect to their own Centroidal
L	-	Span of the sandwich beam
M	-	Maximum beam bending moment
t,(t_t ,t_b)	-	Skin thickness (top/ bottom)
u_t ,u_b	-	In plane centroidal displacements of top or bottom skin
v	-	Base width of triangular pressure elements
w	-	Load per unit width
y	-	Leaver arm to the point considered

Note - Other symbols not mentioned are defined in the text or figure whenever they appear.

CHAPTER 1

POLYURETHANE SANDWICH PANEL FOR DOMESTIC CONSTRUCTION

1.1. Introduction

The thermaly efficient house is a dream. But there is a possibility to come closer to the ideal situation. Recently new construction form was revealed as a material for the cold room construction. The way that it erected is much simple. The study is going based on the possibility of application of thermal insulative material as new mean of construction of domestic unit to meet Sri Lankan requirements. If this concept is adapted, future dwellers can construct their own house by themselves within a week or two. This is a burning problem for most of the Sri Lankans. The study is going to cover up the economical viability of the new concept for a two story house and two story cluster house model.

Thermal comfort of a house is an important technological concern because of their great utility, widespread use, energy consumption, and contribution to environmental degradation. The energy required to maintain a thermally conditioned interior volume within a compartment depends on how it is used, the thermal resistance of its shell, and the efficiency of its mechanical systems.

1.2. Problems

The place to live is the third basic need of mankind. According to Sri Lankan culture, every one dreams to have their own house. The government is also helping people to construct their own house.

Most of the houses constructed in Sri Lanka would have a useful life span of 25 years. Within the period the architecture and the need of the dwellers may change drastically and would emerge a new house.

The shortage of construction material is becoming a critical concern now. Therefore leading to a shortage and price hike. Since no sustainable alternative was introduced in the past, whole community still hangs on traditional construction materials.

The contribution to the global warming is not simple. If an average house may have the floor area of 1800 sqft – 2000 sqft, the cement requirement is nearly 1,200 bags. The Carbon dioxide emission due to cement itself is nearly 16 tons. Due to the brick burning process, the carbon dioxide emission for the aforesaid volume house is also nearly 25 tons. The carbon dioxide emissions from minor facts were not considered.

The history reveals that Sri Lanka has more than 6,000 years of colonies and civilizations. If there were constructions based on none sustainable materials, there would be no any materials even to see. Therefore, it is required to replace the construction materials or at least a portion of use.

The traditional construction material may have the densities nearly 1800kg/m^3-2400kg/m^3. The proposed construction material density is 430kg/m^3. Therefore it is easily possible to construct houses on marshy lands without much improvement.

The proposed material is from polyurethane. These materials are normally extracted from the recycled materials. The direct natural resource utilization is very minor.

The shortage of labour in construction industry is a major problem. Therefore it is required to deliver less labour utilized techniques for the domestic construction.

Since this material is polyurethane it is vulnerable to fire. The extended protection is providing to improve. But the inbuilt system delivers adequate retention as

(a) To extinguish the fire, to control the fire, or to provide exposure protection for structures on site, by a sprinkler system, or a foam system,

(b) To introduce additives into resin formulations, by incorporating halogens into resin formulations fluoride, chlorine, bromine and iodine family of chemicals or combining synergists in the resin (e.g. Het acid resin), adding epoxy-layered silicate nano-composites at the time of formulating the resin. The process is complicated and at present, it is expensive for the civil engineering industry,

(c) To apply a passive fire protection system to treat the surface of the manufactured composite by using in tumescent coating technology. These coatings incorporate an organic material which will char and evolve gases at a designed temperature so as to foam the developing char.

1.3. Objective & Boundaries

The introduction of new construction material is the main goal. Therefore the possibility of construction of load bearing wall is incorporated for the houses up to two stories in this thesis.

The economical viability is another concern. Therefore the economical viability has been proven in Chapter five.

User friendly construction technique is another basis. Therefore the owner can understand and might be able to construct his own needs.

The technical implications are minimum for houses up to two stories which have the room spans up to 3.5 metres. Therefore, detailed engineered analysis may not be required. A common guideline can generally be expressed up to two story houses.

It is to design two house models according to the standard loadings. The engineered validity of material in the form of shear and bending capacity will be proved.

The main drawback in the polyurethane is the high flammability. A proper analysis shall be delivered for the hot working areas. The available retarding system to be further strengthened in time to come with the development of science and technology.

1.4. Scopes

The possibility of utilization of sandwich panel for domestic construction is the main scope. There are several deliverables based on the scope. The introduction of a new construction material to meet Sri Lankan need is a socio economic solution.

1.5. Methodologies

It is required to analyse the samples to scrutinize the shear and bending properties of the polyurethane sandwich panels. Accordingly the two house models were planed and analysed. The model buildings were applied with the required standard loading parameters typical to Sri Lanka. The available thickness was used to model the house.

Based on the total cost of the ownership from new construction technique and the traditional technique shall be elaborated to see the economical viability.

1.6. Outline of the thesis

This thesis consists of Six Chapters. The initial Chapter is for the introduction. In this Chapter the need for introduction of the new construction material has been discussed. Its validity in the form of environmental concern, natural resources scarcities, the human need changes, the human problems in the form of housing were discussed in this Chapter.

The Chapter two is on Literature survey. The sandwich concept comes from the Mother Nature. The history of the sandwich constructions, its modern evolutions, scientific derivations are included in this Chapter. The properties of insulated sandwich panels and various failure criteria identified are discussed. How the structures be modelled with the openings and the sandwich material service compliances are also discussed.

The Chapter three is to derivate the material properties for the structural analysis. Since sandwich panel has three elements mainly face material, core material and rear face material. It is a complex work in structural modelling and analysis. Therefore sandwich material is modelled as a hypothetical isotropic material having the same physical properties throughout the section. It requires mainly three properties for structural analysis. First one is the density. It can be easily found from shipping documents. The second property needed is the Young's modulus. This was found from experimentally. The three samples flexural behaviour was studied to derivate the common Young's modulus. The Poisson's ratio was literally found. The material limits such as the yield point stresses and the ultimate stress properties were also experimentally derived.

The Chapter four is on the structural modelling and analysis. Two house models comprised of a common individual two stories house unit and a typical cluster house having eight units in two stories were considered. The selected cluster house plan is a typical plan used for 'Urban regeneration project' implementing at city of Colombo by Urban development authority of Sri Lanka. The common free span is limited to 3.5m for all models. The structural stiffeners were introduced to the models to comply with the published guidelines by the Society of Structural Engineers Sri Lanka; “Guidelines for Building at Risk from Natural Disasters [17]. The finite element was sized to 0.5m x 0.5m for the modelling works. The walls, slabs and roofs also modelled from the same material. The standard design loads were considered for the applications. The serviceability limit state checks were done to comply the “European Recommendation for Sandwich Panel; part1, Design [14]. The house constructions from polyurethane sandwich panel were established as possible. These models can further be fine tuned for the material optimisations.

The Chapter five is on economical analysis. Eventhough the material safety factor is more than two, the economical evaluation revealed that the saving is more than 41% from the traditional constructions. On material optimisations more saving can be derived. This revealed that the domestic constructions from polyurethane sandwich panel are not dreams.

The Chapter six is on outline the project scope. This chapter validate the project in terms of engineering, economics, quality control and time saving. Apart from that this project has the validity over natural resource saving, low emission of green house gases and re-usage of garbage.

CHAPTER 2.0

LITERATURE REVIEW

2.1. History on Sandwich structures

The Mother Nature derived the system of sandwich with the beginning of time and creatures development from the ancient world day towards. The Efficient use of materials and energy leading to minimal weight is a basic principle in nature and the concept of sandwich.

Human skull is an example of a foam core sandwich structure. Many other examples can be found such as the skeleton of organisms, in leafs of plants as well as in the wings of birds. The animal Skeleton keeps its strength while having filled the core by light weight material such as flesh.

2.1.1. Sandwich material structures

Komarika- Aloevera leaf, displaying in the below Figure 1 is a good example in local environment. It has strong skin and a core filled with some form of gel. The combination form a sandwich structure and it can bear the heavy leaf weights comparative to individual element of core and skin.

Figure 1-Alovera plant

Skeleton on tree leaf

The tree leaf is an example for sandwich. The microscopic view of the tree leaf is in Figure 2 as follows. The tree leaf also has the hard shell element and soft core elements. This phenomenon is same as sandwich concept. The shell elements itself cannot sustain with out the core as the shell elements are weak. The synergy of hybrid concept deliver the high value added phenomenon to sandwich.

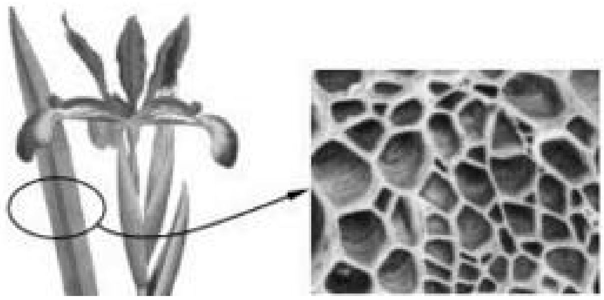


Figure 2-The microscopic view of the tree leaf

Skeletal structures

The skeletal structure of birds' wing is elaborated in Figure 3. The skeletal structural elements also cannot sustain itself as those elements are slender and weak in compression. The filed core is from flesh and it gives pressure force on either side of weak skeletal element. This system delivers very strong product.

Figure 3- The enlarged view of a section of a bird wing

The Human Skull

The human skull a part of the skeletal structure of human is also another example from Mother Nature. The load bearing capacity of the human skull is much higher than the skull itself as from inert brain cells make an internal pressure on the skull synergies more strength. The following Figure 4 shows a sectional view of human skull.

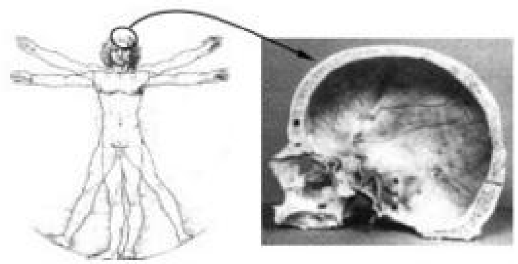


Figure 4-Section of a human skull

2.2. Scientist on modern evolutions

The science and technology derived from Mother Nature after having done the studies in the scientific behaviour of natural systems. Few of the scientists derived the sandwich concepts from the ancient era.

Archimedes – 230 BC

Archimedes is a scientist in 230 BC. He laid the foundation of engineering and discovered besides density with the law of lever, the first element in understanding the moment of inertia of sandwich constructions.

Roman bridges over the river Rhine and Danube were constructed in year 55 BC. This was a form of lattice sandwich structure. The Figure 5 shows a mural showing the Roman bridge over the river Rhine and Danube.

Figure 5-The Roman bridges over the river Rhine and Danube

This is a remarkable example of the practical capabilities, experience and understanding of the Romans. The 1000 m long bridge over the river Rhine and Danube build by Apollodorus under the Emperor Trojan.

Marcus Vitruvius Pollio - 25 BC

The book on architecture and technology by Vitruvius documents efficient use of materials in architecture by the Romans. Vitruvius (90-25 BC) reports in "De architectura libri decem" about the discoveries of Archimedes and describes Roman truss roof structures in detail. The Figure 6 expresses the Palladio drawing in the 1556 edition of Vitruvius book.

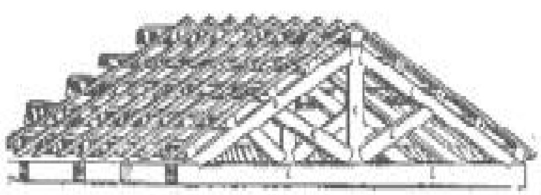


Figure 6-Palladio's drawing in the 1556 edition of Vitruvius book

Galileo Galilee - 1638

Galilee works on bending problems and describes the efficiency of tubes versus solid rods. He had said that "I want to add the theory of resistance of hollow solids. Art and nature even more, makes use of these in thousands of operations in which robustness is increased without adding weight, as is seen in the bones of birds and in many stalks that are light and very resistant to bending and breaking." The strength addition without addition of weight was discussed. Later this phenomenon was developed.

Alplionse Duleau – 1820

The first deflection calculation was done by Duleau in 1820. He was the first person who first used the relation on second moment of area. He expressed the relationship for hollow materials as $I = b\ (h^3\text{-}h_1^3) / 12$. The conceptual expression is figure out in Figure 7 as follows.

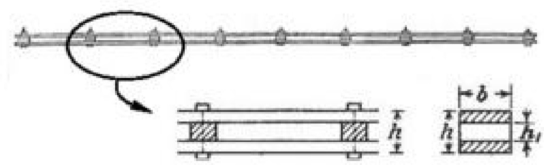


Figure 7-. First use of the relationship for the hollow sections, $I = b\ (h^3\text{-}h_1^3) / 12$

Robert Stephenson – 1830

Robert Stephenson introduced the first sandwich beam in transportation. An Ash wooden beam plated with wrought iron. This reduced the weight of the locomotives.

The term sandwich was first used for this three layer structural applications on beam construction of the locomotive frames of Stephenson. Robert Stephenson constructed railway bridges also by use of the sandwich concept.

Octave Chanute – 1894

Octave Chanute, a railway bridge engineer, invented sandwich biplane aircraft construction with wooden struts and diagonal wires. Chanute offered his findings to successful flying machine. The Figure 8 expresses octave flying machine.

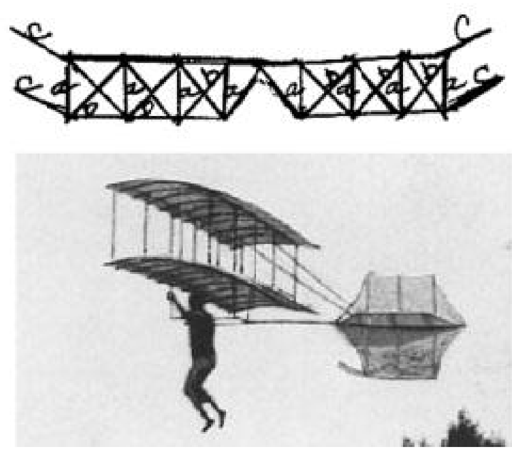


Figure 8- Octave Chanute’s flying machine

Hugo Junkers - 1915

Hugo Junkers patented for the first honeycomb cores for aircrafts. He reasoned that a metal sheet can also be loaded in compression, if it is supported at very small intervals. The Figure 9 expresses an enlarged sandwich element.

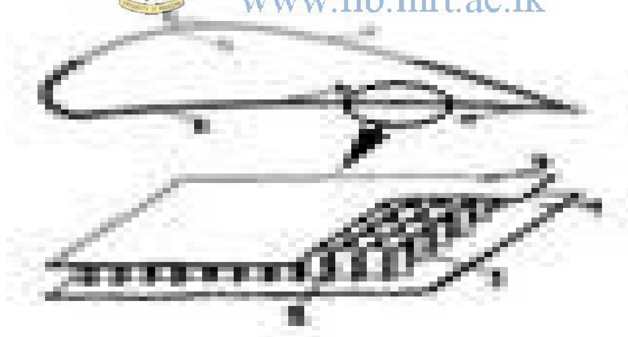


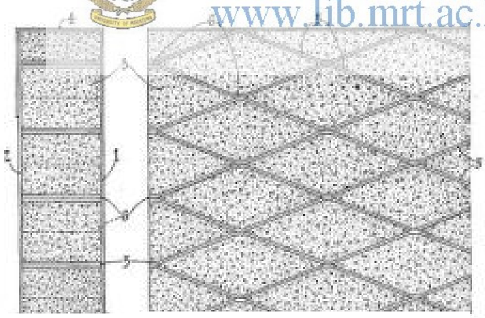
Figure 9-.Junkers honeycomb structure

The Junkers has expressed that “may be produced by arranging side by side series of square or rectangular cells, triangular or hexagonal hollow bodies.” However, the problem of bonding a continuous skin to cellular cores led Junkers to open corrugated structure, which could be riveted or welded together. The Junkers invented passenger plane becomes the prototype for the modern civil aircraft. The Figure 10 shows the Junkers passenger plane F13.

Figure 10-Junkers passenger plane F13.

George Thomson - 1931

He first expanded the paper honeycombs in structural application. He invented the application of expanded paper honeycombs as a core material for lightweight plasterboard panels. The Figure 11 expresses the expanded paper honeycombs in plaster boards.



Section Sectional elevation

Figure 11-The expanded paper honeycombs in plaster boards

Edward Budd – 1934

He introduced the welded steel honeycomb sandwich panel from corrugated metal sheets. He was the first all-steel car manufacturer and in the 1920´s his company became leading in automotive steel stamping. He designed and built in 1934 the record breaking Zephyr train. The Figure 12 expresses the Edward Budd’s invention.

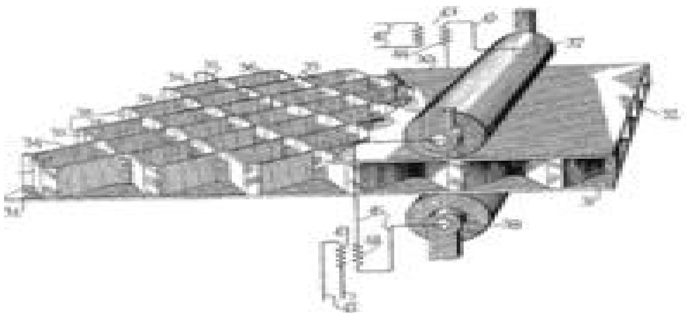


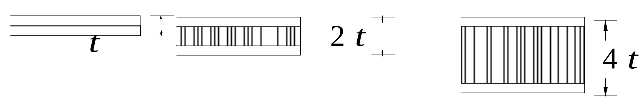
Figure 12-Steel Sandwich core panel production

2.3 Modern Evolution

The Forest Products Laboratory in Madison, Wisconsin, USA, introduced the idea of Structural Insulated Panels (SIPs) in 1935. The Laboratory's prototype panels consisted of framing members, plywood and hardboard sheathing, and insulation. These initial panels were used to build test homes that were disassembled and tested after thirty years to reveal that the panels retained their initial strength values. Frank Lloyd Wright used a form of structurally insulated panels in the Union homes built in the 1930's and 1940's. In 1952, Alden B. Dow created the first foam core SIPs which were being mass-produced by the 1960's.

The main advantage of structurally insulated panel is the very high bending stiffness gain with comparative to the weight. The following table illustrates the bending stiffness gain with weight. The following Table 1 was extracted from [Achilles Petras and M.P.F.Sutcliffe, 1] and [product cater-log of Haxel composite, 16] express the structural efficiency of sandwich panel with core thickness.

Table 1-Structural efficiency of sandwich panel with core thickness [1]



Relative Bending Stiffness	1	7.0	37
Relative Bending Strength	1	3.5	9.2
Relative Weight	1	1.03	1.06

2.3.1 Structurally Insulated Panel and its properties

Structurally insulated panels are composed of an insulated foam core between two rigid board sheathing materials. The foam core is generally one of the following:

Expanded polystyrene (EPS),

Extruded polystyrene (XPS), or

Polyurethane foam (PUR), which can be with EPS and XPS foam

The assembly is pressure laminated together with PUR, the liquid foam is injected and cured under high pressure. The most common sheathing boards are Oriented strand boards (OSB). Other sheathing materials include: sheet metal, plywood, fibre-cement siding, magnesium-oxide board, fibreglass, gypsum sheathing, and composite structural siding panels. Each sheathing material and foam type has its benefits and drawbacks. The type of SIPs selected depends upon the building type and site conditions. The following tables outline the benefits and drawbacks of the most common sheathing and foam types.

Table 2- Sheathing types benefits and drawbacks [12]

Sheathing Type	Benefits	Drawbacks
Oriented Strand Board (OSB)	• Load bearing; • readily available; • tested; • large panel size up to 8' x 24'	• Subject to mould and a reduction in structural capacity if exposed to moisture; • not fire resistant; must be treated for termites; • Difficult substrate for most common joint tapes
Sheet Metal	• Easy to mould; • Can be load-bearing; • Very light; • Unlimited lengths when made from coil stock	• Must be galvanized or stainless steel; • Not load bearing
Plywood	• Lateral strength	• Availability; • price; • limited panel size; • subject to mould and reduced structural capacity if exposed to moisture for a prolonged period of time; • Not fire resistant; • Must be treated for termites
Fibre Cement Siding	• Resistant to mould, termites, and fire	• Availability; weight; testing; limited panel size
Magnesium Board	• Resistant to mould, termites, and fire	• Availability; testing; limited panel size
Fibreglass Mat Gypsum Sheathing	• Resistant to termites and fire	• Not structural; limited panel size

Table 2- Sheathing types benefits and drawbacks [12] continue…..

Sheathing Type	Benefits	Drawbacks
Composite Structural Siding Panels	• Resistant to mould and termites; • pre-primed materials available	• Not fire resistant

The form core types benefits and draw backs are tabulated as below

Table 3-Form core types benefits and drawbacks [12]

Foam Core	Benefits	Drawbacks
Expanded Polystyrene (EPS)	• Least expensive; • Thickness options are only limited by the foam manufacturer; • Availability; • Fastest to modify in field; most benign blowing agent	• Produced with HBCD*
Extruded Polystyrene (XPS)	• Strength; • Water resistant	• Availability; • Produced with HBCD*
Polyurethane Foam (PUR)	• Highest R-value/metre; strength; • Water resistant	• Most expensive; • Harder to modify thickness limitations; • Creep; • Availability; • Produced with chlorinated phosphate flame retardants**

*HBCD: hexa-bromocyclododecane - a brominated fire retardant classified by the European Union (REACH program) as persistent, bio accumulative, and toxic (PBT).

**Not as hazardous as most brominates flame retardants, but health and environmental concerns still exist.

The following table illustrates the Foam Technical Data

Table 4-Foam technical data [12]

Foam Type*	**EPS Foam**	**XPS Foam**	**PUR Foam**
Density in Panel (kg/m^3)	15.1	25.1	38-42
Compressive Strength @ 10% deformation (psi)	10	20	35
R-value/in @ 75° F	3.6	5.0	6.54
Common Fire Retardant	HBCD	HBCD	TCPP
Common Fire Rating Class	1	1	1
Common Blowing Agent	Pentane	HFC-134a	HFC-245fa

Most SIP manufacturers use a 0.95 minimum density

2.3.1.1. Thermal Performance

The quality of a building's envelope is measured by its ability to prevent infiltration of outside air. The Table 5 on R value is for elaborations as follows.

Table 5-Typical SIP with whole wall R-values [12]

Thickness	EPS	XPS	PUR
112mm	13.1	17.7	22.7
160mm	19.9	27.2	35.1
210mm	26.0	35.5	46.0
260mm	32.9	45.0	NA
310mm	39.8	54.6	NA

2.3.1.2. Fire Safety

The PUR SIP provides 1 hour fire protection. But as per the domestic regulations of Sri Lanka, United Kingdom and other countries 30 minutes fire protection is adequate. The science and the technology is under the developments as by doping some wastes to core material to increase the fire retarding time further.

2.4. Structural Design and Construction

SIPs behave similarly to a wide flange steel column in that the foam core acts as the web and the sheathing responds as the flanges. Under axial loads, the sheathing responds similarly to a slender column, and the foam core acts as continuous bracing preventing the panels from buckling. Just as wide flange sections increase in strength with increased depth, thicker cores result in stronger panels in compression and bending.

SIPs are designed to resist not only axial loads, but also shear loads and out of plane flexural loads. The panels' ability to resist bi-axial bending and lateral shear allow them to be used as roofs and floors. SIPs are acceptable to use as shear walls in all seismic design categories.

To date, the tallest structure constructed exclusively of SIPs is four stories. Taller structures are possible; however, design limitations are due to the fact that SIPs are bearing walls and therefore open spaces at lower floors are more difficult to achieve. Often large SIP structures rely on a secondary framing system of steel or timber to satisfy requirements for unobstructed spaces. Unique screw connections are available to attach SIPs to wood, light gauge steel, and structural steel up to 6 mm thick.

It is imperative for foundations for Structurally Insulated panels to be level. There is little tolerance for differential settlement. If there is substructure shift, it will compromise the sealant of the panels' joints which may cause moisture infiltration. Allowable deflection tolerances set by the manufacturer of the panels and sealants should be consulted when designing the foundation. Minor imperfections may be accommodated with careful, skilled installation.

Two of the most widely used panel joint connections are the surface spine and the block spine. The surface spine joint connection consists of strips of plywood inserted in slots in the foam just inside each skin of the SIP. The block spine is a thin and narrow SIP assembly that is inserted into recesses in the foam along the panel edges. The surface spine connection and the block spine connection result in a continuous foam core across the panels.

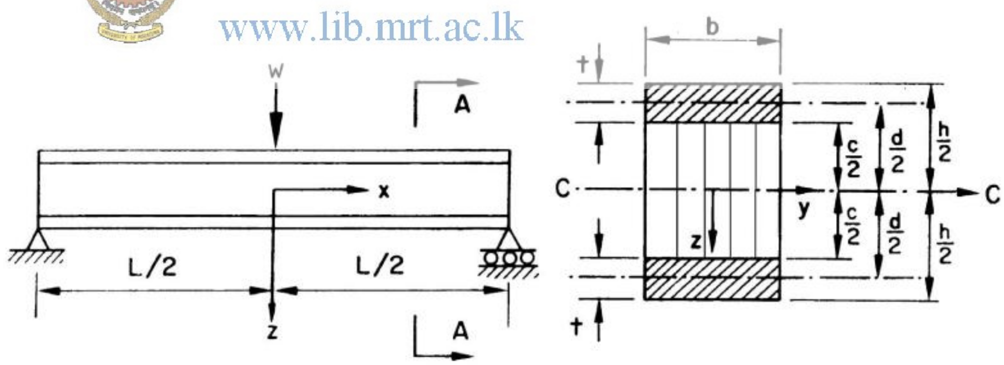
Openings can occur anywhere within the panel, including at the edges and corners. However, panel openings can be reinforced at headers so that additional structure is not required.

Typical wall panel thickness are 100 mm and 160mm. Curved panels are also possible although not common and it is often more practical to use stud framing for non-orthogonal geometries[1].

Roof panels are typically 275mm and 310mm thick. Roof panel thickness depends upon the required R-value and span. EPS, XPS and PUR panels can be made up to 310mm thick. End wall panels for various roof profiles can be achieved with SIP[1].

2.5. Beam theory on sandwich panels

This section outlines the elastic analysis of sandwich beams in three point bending. This will be used to evaluate the stresses in the core or skin and hence the failure loads due to the various mechanisms. Consider a simply supported sandwich beam of Span "L", width "b" and loaded in three points bending with a central load "W" per unit width as illustrated in following Figure 13. The skins each have thickness "t" and are separated by a thick layer of honeycomb core of thickness "c".



a Simply supported beam b Cross section on A-A

Figure 13- Simply supported testing arrangement on three points loading

Assumed that the skins remain firmly bonded to the core that the beam bends in a cylindrical manner with no curvature in the "YZ" plane and those cross sections remain plane and perpendicular to the longitudinal axis of the beam the flexural rigidity "D" of the sandwich beam is then given by,

Equation 1

$D = E_{fx}bt^3/6 + E_{fx}btd^2/2 + E_{cx}bc^3/12$

where "d" is the distance between the mid planes of the upper and bottom skins "E_{fx}" and "E_{cx}" are the in plane Young's module of the skin and core respectively for loading in the "X" direction along the axis of the beam Subscripts 'f' and 'c' denote the face material and the honeycomb core respectively. The subscript's' is used in later expressions for the solid material from which the honeycomb is made.

The three terms on the right hand side correspond to bending of the skins about their centroidal axes, bending of the skins about the centroid of the whole beam and bending of the core respectively.

This equation can be simplified by assuming that bending of the skins about the centroid of the beam is the dominant term. The contributions of the first and third terms amount to less than 1% of this when

Equation 2

$d / t > 5.77$ and $(E_{fx}/E_{cx})\ (t/c)\ (d/c)^2 > 16.7$ respectively

Therefore

Equation 3

$D \quad = \quad E_{fx}\ btd^2/2 \quad = \quad E_{fx}I$

Where "I" is the second moment of area of the cross section of the sandwich beam with three points bending the maximum bending moment "M" is at the mid span and the corresponding maximum stress.

Equation 4

$$\sigma_{fx} = \frac{ME_{fx}}{D}\frac{d}{2} = \frac{WL}{4dt}$$

However, the above theoretical model neglects the effect of shear defection in the core which becomes significant for low density cores.

Therefore as per the suggestion of Allen [2] for the maximum axial stresses in the faces

Equation 5

$$\sigma_{fx} = \frac{WbL}{4}\left(\frac{c+2t}{2I} + \frac{WL}{4}\frac{t}{2I_f}\frac{I}{\theta}\right)$$

Where

Equation 6

$$\theta = \frac{L}{c}\left[\frac{G_{cxz}}{2E_{fx}}\frac{c}{t}\left(1+\frac{3d^2}{t^2}\right)\right]^{1/2}, \quad I = \frac{bt^3}{6} + \frac{btd^2}{2}, \quad I_f = \frac{bt^3}{6}$$

"G_{cxz}" is the out of plan shear modulus of core, "I" is the second moment of area of sandwich with respect to its neutral axis and "I_f" is the second moment of the area of the face material with respect to their own neutral axis. The equation 6, "θ" highly depends on the relative sandwich with respect to its neutral axis and "I_f" is the second moment of the area of the face material with respect to their own neutral axis. The equation 6, "θ" highly depends on the relative stiffness of the skin and the core. Finally Equation 5 gives.

Equation 7

$$W = 4\sigma_{fx}\xi\frac{t}{L}$$

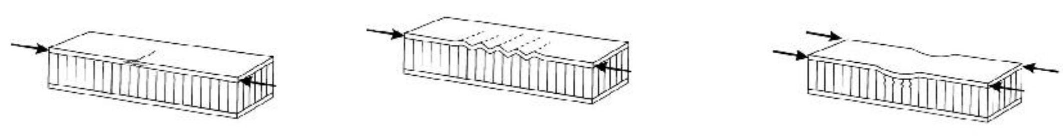
Equation 8

$$\text{Where } \xi = \theta\,\frac{\frac{t^5}{9}+\frac{t^3d^2}{3}}{ht^3(\theta-1)/3+t^4/3+t^2d^2}$$

As the span length “L” or the core shear stiffness “G_{cxz}” approach infinity Equation 7 tends to a simple beam model of Equation 4.

2.6. Skin failure

Previous section gives an expression for the maximum stress σ_{fx} in the skins. This can be used to predict beam failure due to the skin failure modes of face yielding intra cell dimpling or face wrinkling as illustrated in Figure 14 as follows.



(a) Face yielding (b) Intra_cell dimpling (c) Face wrinkling

Figure 14-Skin failure modes in the SIP

2.6.1. Face Yielding

Failure occurs in the top skin due to face yielding when the axial stress in either of the skins reaches Equation 5 the in plane strength "σ_{fy}" of the face material for loading along the beam axis.

Equation 9

$$\sigma_{fx} = \sigma_{fy}$$

It is assumed that the skin behaves in a brittle manner with a symmetrical beam. The stress is the same in tension and compression faces. The face materials in the compressive face are generally the critical one.

2.6.2. Intra cell Dimpling

A sandwich with a honeycomb core may fail by buckling of the face where it is unsupported by the walls of the honeycomb core. This concept is illustrated in Figure 14 (b). Simple elastic plate buckling theory can be used to derive an expression for the in plane stress on intra cell buckling occurs as,

Equation 10

$$\sigma_{fi} = \frac{2E_{fx}}{1-\gamma_{fxy}^2}\left(\frac{2t}{\alpha}\right)^2$$

Where α is the cell size (i.e. the diameter of the inscribed circle) of the honeycomb and E_{fx} and γ_{fxy} are the elastic modulus and Poisson's ratio for the skin for loading in the axial direction. A similar expression verified experimentally by Kuenzi [3] has been given by Norris[4] in Equations 9 and Equation 10. Those equations can be used to derive the value of cell size above which, there is transition from face yielding to intra cell bulking as,

Equation 11

$$\alpha = 2t\sqrt{\frac{2}{1-\gamma_{fxy}^{2}}\frac{E_{fx}}{S_{fy}}}$$

2.6.3. Face Wrinkling

Face wrinkling is a buckling mode of the skin with a wavelength greater than the cell width of the honeycombs. This concept is illustrated in Figure 14 (c). Buckling may occur either in towards the core or outwards depending on the stiffness of the core in compression and the adhesive strength in practice with 3 point bending inward wrinkling of the top skin occurs in the vicinity of the central load. By modelling the skin as a plate on an elastic foundation Allen [2] gives the critical compressive stress σ_{fw} that wrinkle the top skin,

Equation 12

$$\sigma_{fw} = \frac{3}{(12(3-\gamma_{cxz})^{2}(1+\gamma_{cxz})^{2})^{-1/3}} E_{fx}^{1/3} E_{3}^{2/3}$$

Where, γ_{cxz} is the out of plane Poisson's ratio and E_3 is the out of plane Young's modulus of the honeycomb core.

2.7. Core failure

Honeycomb sandwich structures loaded in bending can fail due to core failure. The prominent failure modes are shear failure or indentation by local crushing in the vicinity of the loads as illustrated in Figure 15.

a) Core Shear b) Local Indentation

Figure 15- The failure modes in the core

2.7.1. Core shear

Assuming simple beam behaviour the shear stress varies through the face and core in a parabolic way under three point bending. If the faces are much stiffer and thinner than the core the shear stress can be taken as linear through the face and constant in the core. Neglecting the contribution from the skins, the mean shear stress in the core is given by,

Equation 13

$$T_{cxz} = W/2d$$

Assuming brittle behaviour, and isotropic properties of sandwich panels, the failure occurs when the applied shear stress equals the shear capacity of the honeycomb,

Equation 14

$$T_{cxz} = T_{cx}$$

2.7.2. Local indentation

Failure of sandwich panels in three points bending can occur at the load point due to local indentation. Failure is due to core crushing under the indenter. The bending stiffness of the skin and the core stiffness determine the degree to which the load is spread out at the point of application. It is important here to mention the main failure characteristic by which indentation differs from skin wrinkling. In indentation the top skin deflects after failure with a wavelength of the same scale as the indenter top skin contact length, whereas in skin wrinkling the defection of the top skin after failure exhibits wavelengths that are larger than the contact length between the indenter and the top skin.

Indentation failure has not been adequately modelled for honeycomb sandwich panels. To include this important failure mechanism a simple empirical approach was used in handbook [3] on sandwich panel construction.

The length of contact between the central roller and the top skin is δ, the load W is transferred uniformly to the core over this contact length, therefore the out of plane compressive stress σ_{zz} in the core is given by,

Equation 15

$$\sigma_{zz} = W/\delta$$

Failure may occur when the compressive stress equal the out of plane stress capacity of the honeycomb,

Equation 16

$$\sigma_{zz} = \sigma_{cc}$$

Since this is not a significant failure mode, the further study on this mode is not made.

2.8 Honeycomb mechanics

To evaluate the failure criteria in above chapter, the stiffness and the strength properties of the honeycomb core are required. In this section, the results of reference [4, 5] are used to express the properties of the honeycomb as a function of properties of the solid material on which the honeycomb is made and the skin is made.

The honeycomb's Poisson's ratios γ_{cxz} required for the failure analysis is γ_{13} or γ_{23} for in-plane Poisson's strains due to out of plane loading in the 3 direction.

The first approximation is the in plane strains at the core material taken as the strain of the core. That is the core behaves as an isotropic material.

Therefore $\gamma_{13} = \gamma_{23} = \gamma_s$

The Young's modulus of the honeycomb in the out of plane third direction is given by the rule of the mixtures expression as,

Equation 17

$$E_3/E_s = \sigma_c/\sigma_s$$

In honeycomb, failure under out of plane compressive stresses occurs due to fracture of the cell walls or due to elastic or plastic buckling of the cell walls. Honeycomb failure is due to a crushing mechanism initiated by elastic buckling and developing as a plastic buckling process. The relevant collapse strength σ_{cc} simply estimated using the rule of mixtures expression.

$\sigma_{cc}/\sigma_{sc} = \rho_c/\rho_s$ where σ_{sc} is the compressive strength of the solid of which the core is made. Wierzbicki [6] gives an alternative expression for the failure stress based on plastic collapse model.

Equation 18

$$\sigma_{cc}/\sigma_{sc} \quad = \quad 3.25(\rho_c/\rho_s)^{5/3}$$

Zhang and Ashby [5] show that the out of plane shear strength and stiffness of honeycombs are independent of height and cell size. Honeycomb cores exhibit slight an-isotropy in their out of plane shear strength and stiffness due to the set of doubled walls. By using simple mechanics models and considering the double wall effect approximate expressions for the shear strengths are derived as,

Equation 19

$$\tau_{31}/E_s \quad = \quad 2.1(\rho_c/\rho_s)^3$$

Shear module,

Equation 20

$$G_{31}/G_s \quad = \quad 0.487\,(\rho_c/\rho_s)$$

The cores shear modulus G_{cxz} calculations in equation 6 and the results in Equation 20 can be used if the core is an isotropic material.

2.9 Experimental evaluation of Honeycomb Mechanics and failure mode map

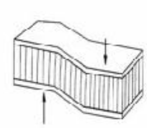
The theoretical correlations detailed in section 2.5, 2.6, 2.7 are compared with the experimental data references [5]. It is observed that there was a deviation from theoretical derivations and experimental derivations. In addition from manufacturer to manufacturer there are deviations reflecting wide manufacturing tolerances.

The out of plane compressive properties were made by testing honeycomb sandwich under restriction of cell wall slipping between the specimen and the rig plates.

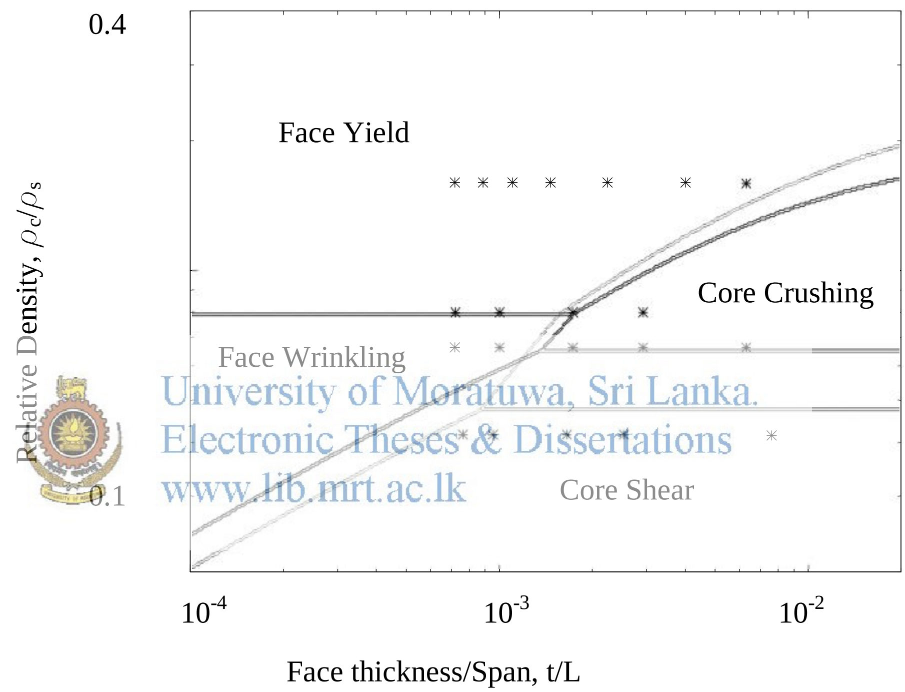
The experimental data of Zhang [5] showed $\rho_c/\rho_s > 0.1$ is the criteria for none-de-bonding skin from core.

The failure criteria developed by Triantafillou and Gibson [7] has been summarized as below in Table 6..

Table 6 - Summary of Failure criteria

Top skin yield		$W_o = 4\sigma_{fy}\frac{t}{L}\xi$
Intra-cell Buckling		$W_o = \frac{8}{1-\nu_f^2}(t/a)^2 E_f \frac{t}{L}\xi$
Face Wrinkling		$W_o = 4B_1 E_c^{1/3} E_s^{2/3}\left(\frac{t}{L}\right)(\rho_c/\rho_s)^{2/3}\xi$
Core Shear		$W_o = 2AE_s d(\rho_c/\rho_s)^3$
Indentation		$W_o = 3.25\sigma_{sc}(\sigma_c/\sigma_s)^{5/3}\xi$

Zhang [5] has noticed indentation is not a critical criterion to be considered for the general constructions. Main parameters such as face yielding, core crushing, face wrinkling and core shear has been graphed out by Zhang [5] as elaborated in Graph 1 for two products. Dark line is for Polyurethane sandwich panel with skin thickness of 0.65 mm from G550 steel. The pale line is for Polyurethane sandwich panel with skin thickness of 0.5 mm from G250 steel. Since the steel we used for this thesis lay in between the above two, it behaves in between two graphs.



Graph 1-Sandwich material failure criteria

The stars referred to some of the tests performed and their relevantness was confirmed by Zhang [5] experimentally.

2.10. The design standards

It is difficult to find the precise and approved design standard for the PUR SIP. But there is a publication done named "European Recommendations for Sandwich Panel Part 1: Design" by the Joint Committee "European Convention for Constructional Steelwork" and "International Council for Research and Innovation in Building and Construction" [14]. But their publications are criticized as invalid and shall be further developed by researches as per Narayan Pokharel and Mahen Mahendran on their publications on "Thin walled Structures" [13]. The reason is the material quality and the properties are highly depending on the supplier and product specifications. In addition the material samples encountered for this thesis testing is found to be out of the range of both above publications.

The serviceability limits published by "European Recommendations for Sandwich Panel Part 1: Design" [14] has expressed as follows.

For roof panels and ceilings,

The deflection caused by the short-term loads should not exceed;

the value = span / 200.

Correspondingly, the long-term deflection of roof panels and ceilings including the effects of creep should not exceed;

the value = span / 100.

For wall panels,

The deflection should not exceed *the value = span / 100.*

In special cases there may be other considerations in the design of sandwich panels, which necessitate more stringent deflection limits.

2.11. Local buckling of the PUR SIP

The experiments done by Pokharel and Mahendran [13] shown that the form thickness has negligible effect on the buckling strength. In addition following are the conclusions with compared to the PUR-SIP analysed with skin material of having the steel of G550 and G250.

The results form FEA and the experiments agreed reasonably well for both G550 and G250 steel plates. The mean value of the ratio of FEA and experimental buckling and ultimate stresses were found to be 1.00 and 0.94, respectively for G550 steel plated SIP material and 1.05 and 0.93 respectively for G250 steel plated SIP. The corresponding coefficients of variations were between 0.06 - 0.11 for G550 steel plated SIP and between 0.08 - 0.12 for G250 steel plated SIP.

In practice, since all panels are interconnected with tongue and grove method, the local buckling effect does not arise as their local buckling capacities exceed the elastic compressive stress. Therefore this phenomenon check is not critical for the SIP interconnected structures.

2.12. Material Properties

The material properties for the SIP are as follows

Young's Modulus

E_s = 205 GPa.

E_c = 3.77 MPa. [9],[13]

Shear Modulus

G_s = 76 GPa. [1]

G_c = 2.9 to 1.76 MPa. [1],[9] and [13]

Poisson's ratio

ν_s = 0.3

ν_c = 0.05 – 0.08 [1], [14]

Density

ρ_s = 7850 kg/m^3.

ρ_c = 380 kg/m^3. [9]

ρ_{sip} = 420 kg/m^3.

Thickness

t = 0.50 mm. (for steel only)

d = 121 mm.

Shear strength

σ_c = 0.12 MPa. [9]

Compressive strength

σ_c = 0.19 MPa. [9]

Tensile Strength

σ_c = 0.09 MPa. [9]

2.13. Sandwich panels with openings

The structural sandwich panels used for walls need to have construction openings for doors windows and HVAC (Heat, Ventilation, Air conditioning) ducts, pipes and other penetrations. The opening may vary with shape and size. Any opening in a structural sandwich panel represent weakening by decreasing the effective area of faces and core that reduce bending, shear and tensional rigidity and resistance of panel cladding.

Existing European standard EN 14509 does not provide design and testing procedures for sandwich panel with openings. The technical solution for the openings with sandwich panels to use a "Replacement" in the form of additional support in the structure. Such reinforcement elements have to replace the load carrying capacity of the removed portion. This is in the form of self supporting system and sandwich panels with the openings are firmly hold and transferred the applied loads to the remaining panel sections. These reinforcement systems provide the stiffness against bending and shear than the original panel section. This is a form of corrugated profile with adequate thickness.

But, there is a common practice to use the structural sandwich panels with openings and cut-outs without any additional support. These design solutions are based on the design solutions derived from the SIP manufacturers.

As per the studies done by Metod Cuk, Silvo Stih and Boris Jerman [8] the analysis results of typical examples found to comply with the European standard EN 14509 and does not need additional supports for small openings. But for the horizontal cuts for the openings with more than two panel width need to transfer shear loads by additional support on the opening. The shear load enhancement on adjacent panels due to this load transfer is not noticeable and does not need any further load transfer.

2.14. Material compliances for Service Life

The service life expectancy of components that are mated with the SIP assembly should match the service life expectancy of the SIP wall itself. Components include durable flashing materials, structural components in the SIP panel, sealants, foam, tape, gaskets, fasteners, etc.

2.15. Young's modulus and deflection

It is required to derivate the Young's modulus of polyurethane sandwich panels. The reliable form is the derivation the relationship with deflection and Young's modulus on three points loading.

Deflection caused by two scenarios

- Deflection due to bending
- Deflection due to shear

The individual element contribution varies with load arrangement. The Anonymous method of flexure test as per ASTM 15.03 (C393-62) [23] describes the three point loading system as below Figure 16.

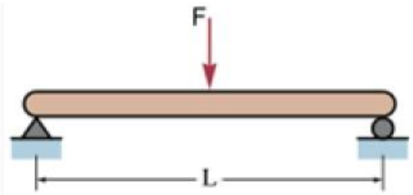


Figure 16- Simply supported centre point loading arrangement

The elastic flexural deflection at the midpoint of a beam, loaded at its centre, supported by two simple supports is given by [24]

Equation 21

$\delta_m \quad = \quad FL^3/48EI$

Where

F = Force acting on the centre of the beam

L = Length of the beam between the supports

E = Modulus of elasticity

I = Area moment of inertia of cross section

The deflection caused due to shear effect also given by [24]

Equation 22

$\delta_s \quad = \quad 19.2\, M_{max}/AE$

Where

M_{max} = Maximum flexural moment

A = Area of section

Therefore the total deflection can be derived as

Equation 23

$$\Delta_{total} = \delta_m + \delta_s$$

Equation 24

$$\Delta_{total} = FL3/48EI + 19.2\ FL/4AE$$

Equation 25

$$\Delta_{total} = FL3/48EI + 4.8FL/AE$$

Since other parameters are known Young's modulus can be calculated.

This expression is used to evaluate the Young's modulus in Chapter 3.

2.16. Sri Lankan guideline on housing construction.

Society of structural engineers, Sri Lanka; published a book on "Housing construction guide lines" [17]. It has requested additional robustness on the perimeter walls, roof bands, wall ties etc.

2.16.1. Foundation.

2.16.1.1. Essential requirements.

- Buildings can have shallow foundations on stiff sandy soil. When there is risk of scouring due to flooding, a minimum foundation depth of around 1.0 m below natural ground level should be provided in the Coastal Zone. In other regions, it can be around 0.6 m to 0.75 m.
- When a building is constructed on stilts, these stilts should be properly braced in both the principal directions. This will provide stability to the complete building under lateral loads.
- Knee braces are preferred to full diagonal bracing so as not to obstruct the passage of floating debris during a tidal surge or tsunami. The wall foundation should have a width of two and half times the thickness of wall. The minimum thickness shall be not less than 0.6 m.

- Footings should be constructed in stone or solid cement blocks, and not in brickwork.
- The plinth height should be not less than 0.45 m above natural ground level and as per topography requirement, for buildings at risk from flooding, the columns should be founded on pad footings.

2.16.1.2. Desirable Features.

- The individual reinforced concrete column footings should be connected by means of reinforced concrete beams at plinth level. These beams will intersect at right angles and thus create an integral housing unit.
- The plinth beam should be at one level throughout and be connected continuously.

- Continuous reinforced concrete footings are considered as the most effective, not only for earthquake resistance, but also to avoid differential settlements under normal vertical loads.

2.16.2. Walls

2.16.2.1. Desirable Features

- All external walls or wall panels must be designed to resist the out of plane lateral pressure adequately. The walls should be sufficiently buttressed by transverse walls.

- A small building enclosure with properly interconnected walls is ideal. Buildings having long walls should be avoided.

- It is necessary to reinforce walls by means of at least one horizontal reinforced concrete band or beam.

- The thickness of the external walls should ideally be not less than 200 mm; other walls can be 100 mm thick. If external walls are 100 mm thick, they must be of solid block work or brickwork.

- Since tensile and shear strengths are important for lateral resistance of masonry walls, use of mud or very lean mortars should be avoided.

- A mortar mix leaner than 1:6 cement: sand should not be used.

- To get the full strength of masonry, the usual bonds specified for masonry should be followed so that the vertical joints are broken properly from course to course.

- Concrete columns founded on pad footings must be provided at least at the four corners of the building. These columns should be connected by a continuous roof or lintel beam / band

- The wall height should be not greater than around 3 m.

2.16.2.2. Aditional Desirable features.

- In addition to the roof beam and corner columns, a continuous plinth beam and lintel band can be considered on external and even internal walls. This will make the building act as an integral unit under lateral forces.

- The reinforcing ties can be introduced at wall intersections and joists of openings. These should be surrounded by concrete of size 100 mm x 100 mm. The anchorage should extend from the plinth beam to the roof beam. All bars should have an "L" bend length of 300 mm. This is mainly for earthquake resistance.

- Although plastering is best avoided to save material, it may be useful to improve the strength and weather resistance of external walls, especially if they are made of only 100 mm thick masonry.

2.16.3. Roof.

2.16.3.1. Essential requirements.

- Light weight (G.I. or Asbestos sheet) low-pitched roofs should be strongly held down to joists, with fastenings not exceeding 1.5 m spacing in both directions. That is along and across slope.

- Similarly, joists should be tied to rafters and the rafters to the wall plate.

- The wall plate should be held down by 10 mm threaded bars cast into the roof beams at around 1.5 m centers'; the bars should have an L-bend of around 100 mm.

- If the threaded bars are to be anchored in vertical ties, the straight anchorage length should be around 250 mm.

- Pitched roofs needed with slopes in the range of 22º to 30º. This will reduce suction on roofs and facilitate quick drainage of rainwater.

2.16.3.2. Desirable features.

- The above essential requirements are mainly for buildings in cyclone prone areas. However, they are desirable features for buildings in other areas also.

- Gabled walls and simple double pitch roofs should be avoided except for very small structures. It is better to have hipped roofs with wall plates on all four sides.

- If the roof is from concrete the minimum thickness of 100 mm and minimum grade of 20 should be used. A gentle slope of 1:100 shall be provided for the flat roof will enable quick drainage of rainwater.

- Vertical reinforcement bars from the columns should be tied and anchored in the roof beams. That will be monolithic with the slab.

- If cantilevers cannot be avoided, they should be well anchored to protect them from earthquake damage.

2.17 Suggested actions

Accordingly for SIP house models, the robustness is to improve. Therefore equal angle of 100 mm x100mm x10mm is introduced in lintel level and along the outer perimeter. The typical foundation was also detailed to comply the guidelines in chapter 5 for evaluations.

The theoretical core-relationships found from literature survey are given in this Chapter 2. To derive the SIP engineering properties such as the Young's modulus, elastic strength of SIP in tension, compression and shear, the experiments were done by the author and the details are given in the Chapter 3.

CHAPTER 3

EXPERIMENTAL EVALUATION TOWARDS FINITE ELEMENT MODELLING

3.1 Introduction

The finite element analysis of individual items in structurally insulated sandwich panel is a complex work. Sandwich panel does comprise of three individual elements such as front face material, core material and back face material. Based on the application, the producers generally produce the different front face and back face material. But the material found for this research use is having the same properties as front face material and back face material. On this research, the simple hypothetical element is considered for finite element modelling works. It needs mainly two properties for finite element analysis. One is Young's modulus and the other is Poisson's ratio. On this study the Poisson's ratio is literally found as 0.05 [1]. This chapter is to derive the Young's modulus of sandwich panels.

3.2. The elastic behaviour of sandwich panels

3.2.1 Stress and strain

The stress and strain may be described in the case of a material when under tension or compression. If a material of cross-sectional area A_0 is pulled by a force F at each end, the bar stretches from its original length L_0 to a new length Ln. (Simultaneously the cross section decreases.) The stress is the quotient of the tensile force divided by the cross-sectional area, or F/A_0. The strain or relative deformation is the change in length, $Ln - L_0$, divided by the original length, or $(Ln - L_0)/L_0$. This expression is illustrated in the below Figure 'Explanatory figure on Elastic behaviour'. Thus Young's modulus may be expressed mathematically as, Young’s modulus = stress/strain = $(FL_0)/A_0\,(Ln - L_0)$.

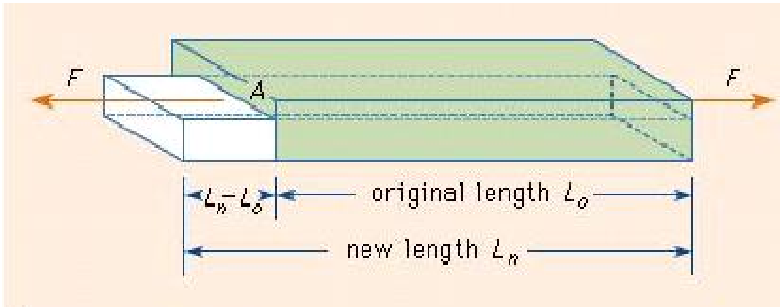


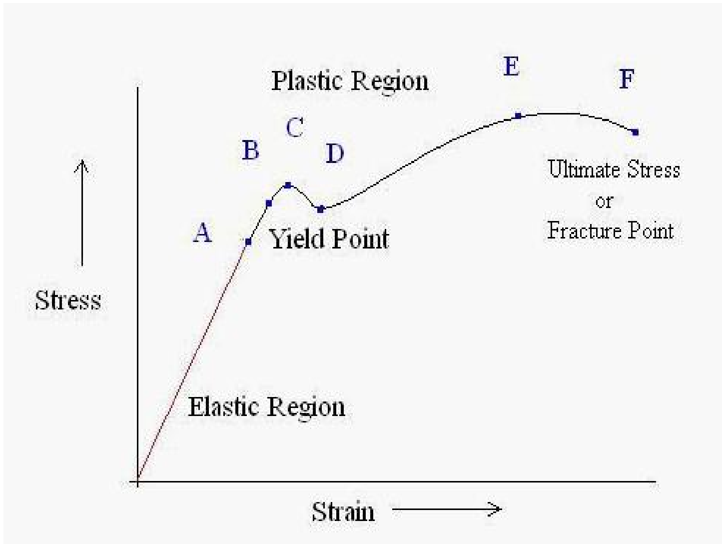
Figure 17-Explanatory figure on Elastic behaviour.

Young’s modulus is valid only in the range in which the stress is proportional to the strain, and the material returns to its original dimensions when the external force is removed. This type of material is described as a linear elastic material. As stresses increase, Young’s modulus may no longer remain constant but decrease, or the material may either flow, undergoing permanent deformation, or finally break. The Young’s invention is not valid for the plastic behaviour region.

Let the change in length be $\Delta L = Ln - L_0$

If the instantaneous minima cross-sectional area can be measured during a test along with F and L and if the constant-volume deformation assumption is, valid while plastic deformation is occurring, then a true stress-strain diagram can be constructed.

The stress strain relationship is described in the following graph 2 as below.. There are few notations in the graph indicating valuable points.



Graph 2-Typical Stress versus Strain graph.

Point A: At origin, there is no initial stress or strain in the test piece. Up to point A Hooke's Law is obeyed according to which stress is directly proportional to strain. That is why the point A is also known as proportional limit. This straight-line region is known as elastic region and the material can regain its original shape after removal of load.

Point B: The portion of the curve between AB is not a straight line and strain increases faster than stress at all points on the curve beyond point A. Point B is the point after which any continuous stress results in permanent, or inelastic deformation. Thus, point B is known as the elastic limit or yield point.

Point C & D: Beyond the point B, the material goes to the plastic stage until the point C is reached. At this point, the cross- sectional area of the material starts decreasing and the stress decreases to point D. At point D the work piece changes its length with a little or without any increase in stress up to point E.

Point E: Point E indicates the location of the value of the ultimate stress. The portion DE is called the yielding of the material at constant stress. From point E onwards, the strength of the material increases and requires more stress for deformation, until point F is reached.

Point F: A material is considered to have completely failed once it reaches the ultimate stress. The point of fracture, or the actual tearing of the material, does not occur until point F. The point F is also called ultimate point or fracture point.

3.2.2. Hooke's law

Hooke's law can be derived from this formula, which describes the stiffness of an isotopic material under pure tension load of F,

$F = (EA_0/L_0)\Delta L$ let $k = EA_0/L_0$ and $x = \Delta L$

$F = kx$

3.2.3. Poisson's ratio

An isotropic material under tension increases its length and decreases in cross section. When an isotropic material under tension is elongated, its width is slightly diminished. This lateral shrinkage constitutes a transverse strain that is equal to the change in the width divided by the original width. The ratio of the transverse strain to the longitudinal strain is called Poisson's ratio. By considering the literature (2.10) the Poisson's ratio of steel as 0.3, and for the sandwich, it is 0.05[1]. Therefore a reasonable assumption is made for Poisson's ratio of polyurethane sandwich panel as 0.05 considering the worst scenario.

3.2.4. Thermal expansion

Thermal expansion is the tendency of matter to change in volume in response to a change in temperature.

Materials which contract with increasing temperature are unusual; this effect is limited in size, and only occurs within limited temperature ranges.

The typical values of polyurethane form is $70x10^{-6}$ m/mk, for steel is $13x10^{-6}$ m/mk, for sandwich panels the thermal expansion coefficient is $65x10^{-6}$ m/mk [10].

3.2.5. Shear modulus or modulus of rigidity

Generally denoted by "G" is defined as the ratio of shear stress to the shear strain,

$$G \stackrel{\text{def}}{=} \frac{\tau_{xy}}{\gamma_{xy}} = \frac{F/A}{\Delta x/l} = \frac{Fl}{A\Delta x}$$

Where

$\tau_{xy} = F/A$ = shear stress;

F is the force acts.

A is the area on which the force act.

In engineering, $\gamma_{xy} = \Delta x/l = \tan\theta$ = shear strain. Elsewhere, $\gamma_{xy} = \theta$,

Δx is the transverse displacement.

l is the initial length.

3.2.6. Relation among elastic constants

For homogeneous isotropic materials simple relations exist between elastic constants (Young's modulus "E", shear modulus "G", bulk modulus "K", and Poisson's ratio "v") that allow calculating them all as long as two are known as below;

$$E = 2G(1+\nu) = 3K(1-2\nu).$$

3.3. Laboratory Test performances

The laboratory tests were performed to find the shear strength, Young's modulus, compressive strength and tensile strength of polyurethane sandwich panels. Eleven specimens were tested. The specimen number one, two and three to perform the shear test from Universal testing machine. The specimen no four, five and six to perform the deflection tests to derive the Young's modulus. The specimen no seven, eight and nine to perform the compressive test to determine the compressive strength. The specimen number ten and eleven on SIP skin samples. Those are to do the tensile testing from Hounsfield Tensometer. Except sample number ten and eleven, all tests were performed from Universal testing machine.

3.3.1. Shear strength on polyurethane sandwich panel

Three different samples were tested namely Specimen number 1, Specimen number 2 and Specimen number 3. Since each specimen width is different, all testing results were plotted in a graph of shear stress versus deflection.

3.3.1.1. Testing of Specimen no 1

The Specimen no 1 had the length of 1,250mm, width of 317.5mm and height of 121.5mm. It was loaded as simply supported from each end with two rollers of 32 mm diameter. The specimen centrally loaded from a 32mm diameter rod. The support roller span was 1,150 mm. The results were observed until the ultimate behaviour. The behaviour was noted based on the visual inspections. The results are tabulated as in following table 7.

Figure 18-Test set up for Specimen no 1.

Table 7-Test results of Specimen no 1.

Force units	Calibrated reading (N)	Shear Force (N)	Shear Stress (N/mm^2)	Central deflection (0.1mm)	Central deflection (mm)
0	0	0	0	0	0
5	277	138.5	0.00359	8	0.8
10	555	277.5	0.007194	15	1.5
15	833	416.5	0.010797	20	2
20	1111	555.5	0.0144	30	3
25	1388	694	0.01799	35	3.5
30	1666	833	0.021594	40	4
35	2000	1000	0.025923	50	5
40	2222	1111	0.0288	55	5.5
45	2555	1277.5	0.033116	65	6.5
50	2971	1485.5	0.038508	70	7
55	3201	1600.5	0.041489	80	8
60	3489	1744.5	0.045222	90	9
65	3777	1888.5	0.048955	100	10
70	4064	2032	0.052675	110	11
75	4352	2176	0.056408	150	15
78	4525	2262.5	0.05865	170	17

3.3.1.2. Testing of Specimen no 2

The Specimen no 2 had the length of 1,250mm, width of 298 mm and height of 121.5mm. It was loaded as simply supported from each end with two rollers of 32 mm diameter. The specimen was centrally loaded by a channel section of 98mm x 50mm x 8mm. The support roller span was 1,150 mm. The results were observed until the ultimate behaviour. The behaviour was noted based on the visual inspections. The results are tabulated in the following Table 8.

Figure 19-Test set up for Specimen no 2.

Table 8- Test results of Specimen no 2.

Force units	Calibrated reading (N)	Shear Force (N)	Shear Stress (N/mm^2)	Central deflection (0.1mm)	Central deflection (mm)
0	0	0	0	0	0
5	277	138.5	0.003825	7	0.7
10	555	277.5	0.007664	15	1.5
15	833	416.5	0.011503	20	2
20	1111	555.5	0.015342	25	2.5
25	1388	694	0.019168	30	3
30	1666	833	0.023007	40	4
35	2000	1000	0.027619	45	4.5
40	2222	1111	0.030685	55	5.5
45	2555	1277.5	0.035283	60	6
50	2971	1485.5	0.041028	70	7
55	3201	1600.5	0.044204	75	7.5
60	3489	1744.5	0.048181	85	8.5
65	3777	1888.5	0.052158	95	9.5
70	4064	2032	0.056122	100	10
75	4352	2176	0.060099	110	11
80	4640	2320	0.064076	120	12
85	4927	2463.5	0.068039	135	13.5
90	5215	2607.5	0.072016	160	16

3.3.1.3. Testing of Specimen no 3

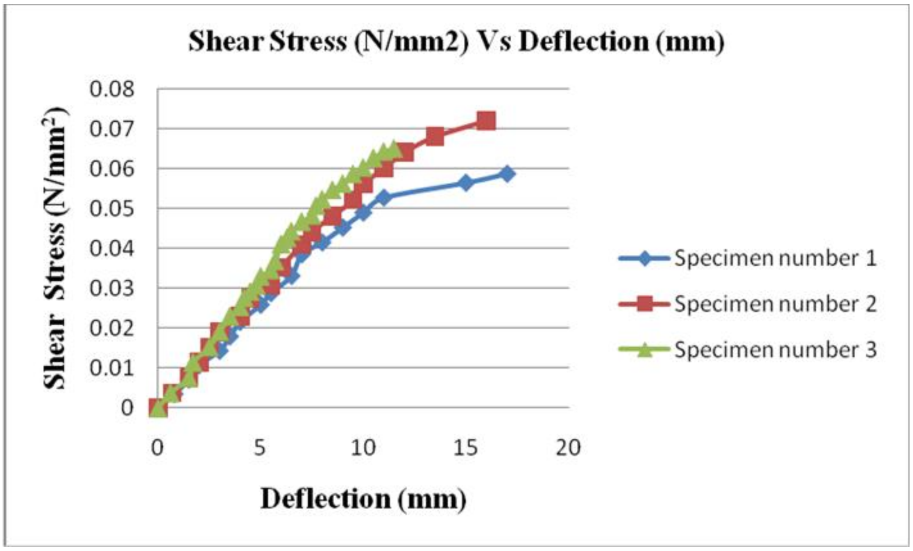
The Specimen no 3 had the length of 1,250mm, width of 297.5 mm and height of 121.5mm. It was loaded as simply supported from each end with two rollers of 32 mm diameter. The specimen was centrally loaded by a channel section of 98mm x 50mm x 8mm. The support roller span was 1,150 mm. The results were observed until the ultimate behaviour. The behaviour was noted based on the visual inspections. The results were tabulated as in the following Table 9.

Figure 20-Shear failure mode on Specimen no 3.

Table 9-Test results of Specimen no 3.

Force unit	Calibrated reading	Shear Force (N)	Shear Stress (N/mm^2)	Central deflection (0.1mm)	Central deflection (mm)
0	0	0	0	0	0
5	277	138.5	0.003832	6	0.6
10	555	277.5	0.007677	15	1.5
15	833	416.5	0.011523	17	1.7
20	1111	555.5	0.015368	25	2.5
25	1388	694	0.0192	30	3
30	1666	833	0.023045	35	3.5
33	1833	916.5	0.025355	40	4
35	2000	1000	0.027665	42	4.2
38	2111	1055.5	0.029201	45	4.5
40	2222	1111	0.030736	48	4.8
43	2388	1194	0.033032	50	5
45	2500	1250	0.034582	55	5.5
48	2666	1333	0.036878	57	5.7
50	2971	1485.5	0.041097	60	6
53	3086	1543	0.042688	63	6.3
55	3201	1600.5	0.044278	65	6.5
58	3374	1687	0.046672	70	7
60	3489	1744.5	0.048262	75	7.5
63	3662	1831	0.050655	77	7.7
65	3777	1888.5	0.052246	80	8
68	3949	1974.5	0.054625	85	8.5
70	4064	2032	0.056216	90	9
73	4237	2118.5	0.058609	95	9.5
75	4352	2176	0.0602	100	10
78	4525	2262.5	0.062593	105	10.5
80	4640	2320	0.064184	110	11
81	4697	2348.5	0.064972	115	11.5

Above Specimen no 1, Specimen no 2 and Specimen no 3 test results have been plotted as follows in Graph 3.



Graph 3-Shear stress versus Central deflection for sandwich panel

Based on Specimen no 1, Specimen no 2 and Specimen no 3 test results; the elastic shear strength limit is noted as 0.05 N/mm^2.

3.3.2. Young's modulus of sandwich panel under flexural behavior

As per the section 2.15, equation 25,

$$\Delta_{total} = FL^3/48EI + 4.8FL/AE$$

Since $I = bd^3/12$

Equation 26

$$\Delta_{total} = (L^3/4bd^3 + 4.8L/bd)\,F/E$$

Force versus deflection graph is plotted, the gradient equals to $(L^3/4bd^3+4.8L/bd)\,/E$. Therefore the Young's modulus can be derived. Since a common behaviour shall be studied, three specimens were tested to find the common flexural behaviour of the sandwich panel. Since three specimens had three different widths, for each specimen a separate graph is needed.

3.3.2.1. Testing of Specimen no 4

The Specimen no 4 had the length of 3,400 mm, width of 270 mm and height of 121.5mm. It was loaded as simply supported from each end with two rollers of 32 mm diameter. The specimen was centrally loaded by a channel section of 98mm x 50mm x 8mm. The support roller span was 3,160 mm. The results were observed until the ultimate behaviour. The behaviour was noted based on the visual inspections. The results are tabulated as in the following table 10.

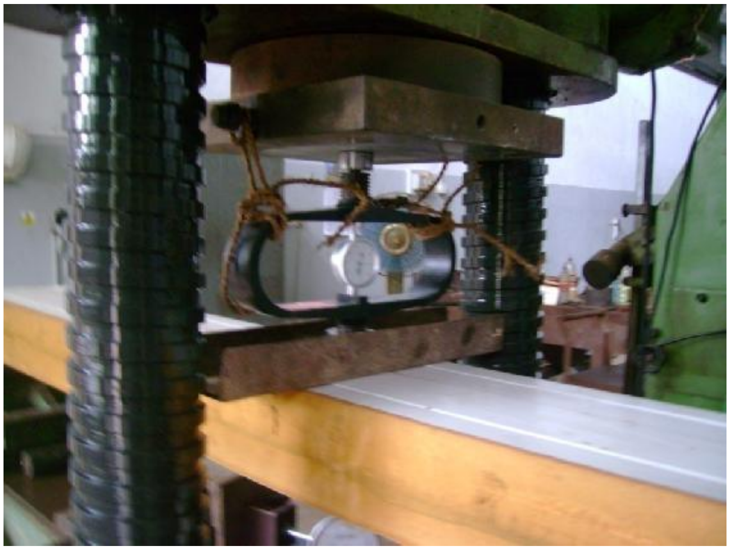
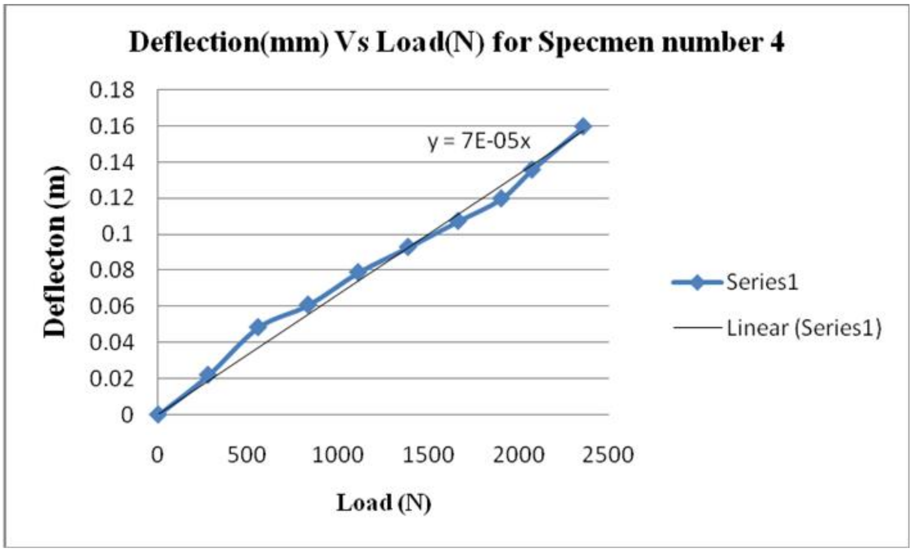


Figure 21-Test set up of Specimen no 4.

Table 10-Test results of Specimen no 4.

force units	Calibrated Force (N)	Central Deformation (0.1mm)	Central Deformation (m)
0	0	0	0
5	277	220	0.022
10	555	485	0.0485
15	833	610	0.061
20	1111	790	0.079
25	1388	930	0.093
30	1666	1075	0.1075
32	1905.86	1200	0.12
35	2076	1360	0.136
40	2361	1600	0.16

The above table test results plotted in the graph as bellow for Specimen no 4.



Graph 4-Delection versus Force graph for Specimen no 4

The linear trend line delivers the gradient as $7x10^{-5}$

From equation 26

$(L^3/4bd^3+4.8L/bd) /E = 7x10^{-5}$

By substituting,

L = 3.16m

b = 0.27 m

d = 0.1215 m

The Young's Modulus derived from Specimen no 4 equals to 0.24 GPa.

3.3.2.2. Testing of Specimen no 5

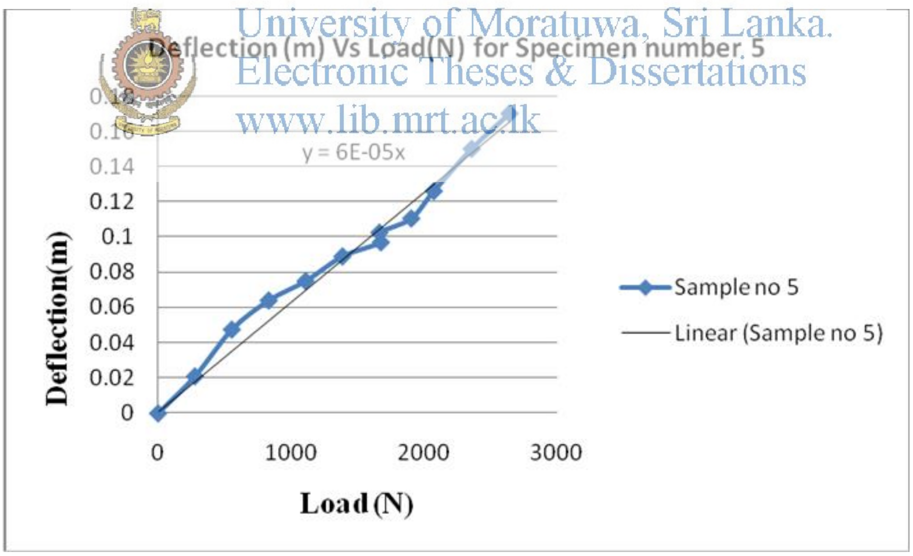
The Specimen no 5 had the length of 3,500 mm, width of 310 mm and height of 121.5mm. It was loaded as simply supported from each end with two rollers of 32 mm diameter. The specimen was centrally loaded by a channel section of 98mm x 50mm x 8mm. The support roller span was 3,160 mm. The results were observed until the ultimate behaviour. The behaviour was noted based on the visual inspections. The results are tabulated as in the following Table 11.

Figure 22-Test set up on Specimen no 5

Table 11-Test results of Specimen no 5.

Force units	Calibrated Force (N)	Central Deformation (0.1mm)	Central Deformation (m)
0	0	0	0
5	277	210	0.021
10	555	475	0.0475
15	833	640	0.064
20	1111	750	0.075
25	1388	890	0.089
28	1677	970	0.097
30	1666	1025	0.1025
32	1905.86	1105	0.1105
35	2076	1260	0.126
40	2361	1500	0.15
45	2646	1700	0.17

The above table test results plotted in the graph as below for Specimen no 5.



Graph 5-Delection versus Force graph for Specimen number 5.

The linear trend line delivers the gradient as $6x10^{-5}$

From Equation 26

$(L^3/4bd^3+4.8L/bd)/E = 6 \times 10^{-5}$

By substituting

L = 3.16m

b = 0.31 m

d = 0.1215 m

The Young's Modulus derived from Specimen no 5 equals to 0.24 GPa.

3.3.2.3. Testing of Specimen no 6.

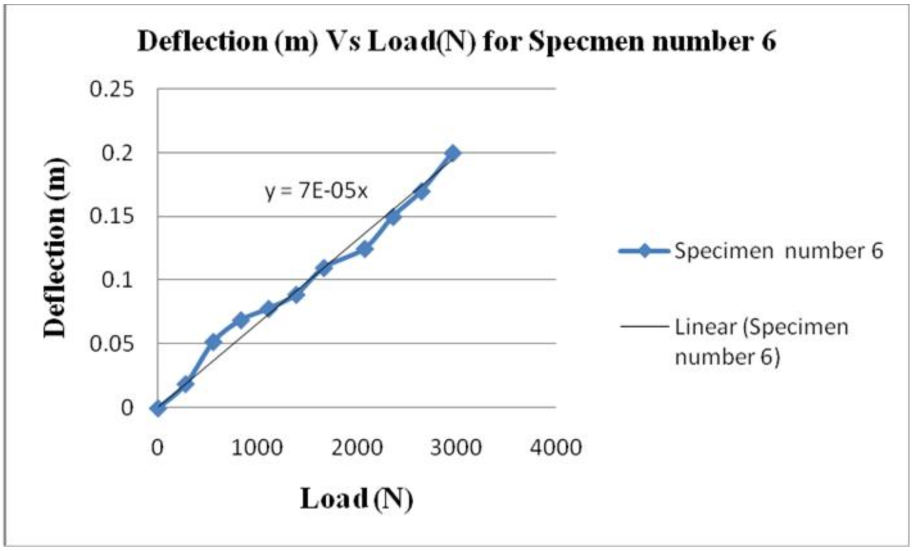
The Specimen no 6 had the length of 3,800 mm, width of 300 mm and height of 121.5mm. It was loaded as simply supported from each end with two rollers of 32 mm diameter. The specimen was centrally loaded by a channel section of 98mm x 50mm x 8mm. The support roller span was 3,160 mm. The results were observed until the ultimate behaviour. The behaviour was noted based on the visual inspections. The results are tabulated as in the following Table 12.

Figure 23- Testing of Specimen no 6.

Table 12- Test results of Specimen no 6.

Force units	Calibrated Force (N)	Central Deformation (0.1mm)	Central Deformation (m)
0	0	0	0
5	277	190	0.019
10	555	520	0.052
15	833	690	0.069
20	1111	780	0.078
25	1388	890	0.089
30	1666	1100	0.11
35	2076	1250	0.125
40	2361	1500	0.15
45	2646	1700	0.17
50	2960	2000	0.2

The above table test results were plotted in the following graph as below for Specimen no 6.



Graph 6-Deflection versus Force graph for Specimen number 6.

The linear trend line delivers the gradient as $7x10^{-5}$

From equation 26

$(L^3/4bd^3+4.8L/bd)\ /E = 7x10^{-5}$

By substituting

L = 3.16m

b = 0.3 m

d = 0.1215 m

The Young's Modulus derived from specimen number 6 equals 0.22 GPa.

By considering the values derived for Young's modulus for Specimen no 4, Specimen no 5 and Specimen no 6, the average value for Young's modulus of sandwich panel (E) = 0.23 GPa.

3.3.3. Elastic compressive strength

Three tests performed to get the common behaviour of the direct compression. Since the sample are man made, the sizes changed from each one. Therefore the compressive stress and the deformation were noticed.

3.3.3.1. Testing of Specimen no 7.

The Specimen no 7 had the length of 198 mm, width of 75 mm and depth of 121.5mm. It was loaded by jaw of loading machine. The results were observed until the ultimate behaviour. The behaviour was noted based on the visual inspections. The results are tabulated as in the following Table 13.

Figure 24-Test set up for Specimen no 7.

Table 13-Test results of Specimen no 7.

Force units	Calibrated reading (N)	Compression Force (N)	Compression stress (N/mm^2)	Shorten deformation (0.01mm)	Shorten deformation (mm)
0	0	0	0	0	0
5	5000	5000	0.5487	50	0.5
10	10000	10000	1.09739	100	1
15	15000	15000	1.64609	185	1.85
18	18000	18000	1.97531	280	2.8
20	20000	20000	2.19479	350	3.5
22	22000	22000	2.41427	420	4.2
23	23000	23000	2.52401	520	5.2

3.3.3.2. Testing of Specimen no 8.

The Specimen no 8 had the length of 205 mm, width of 76 mm and depth of 121.5mm. It was loaded by jaw of loading machine. The results were observed until the ultimate behaviour. The behaviour was noted based on the visual inspections. The results are tabulated as in the following Table 14.

Figure 25- Test set up for Specimen no 8.

Table 14-Test results of Specimen no 8.

Force units	Calibrated reading (N)	Compression Force (N)	Compression Stress (N/mm^2)	Shorten Deformation (0.01mm)	Shorten Deformation (mm)
0	0	0	0	0	0
5	5000	5000	0.54148	47	0.47
10	10000	10000	1.08295	97	0.97
15	15000	15000	1.62443	180	1.8
17	17000	17000	1.84102	250	2.5
20	20000	20000	2.16591	335	3.35
23	23000	23000	2.49079	450	4.5
24	24000	24000	2.59909	580	5.8

3.3.3.3. Testing of Specmen no 9.

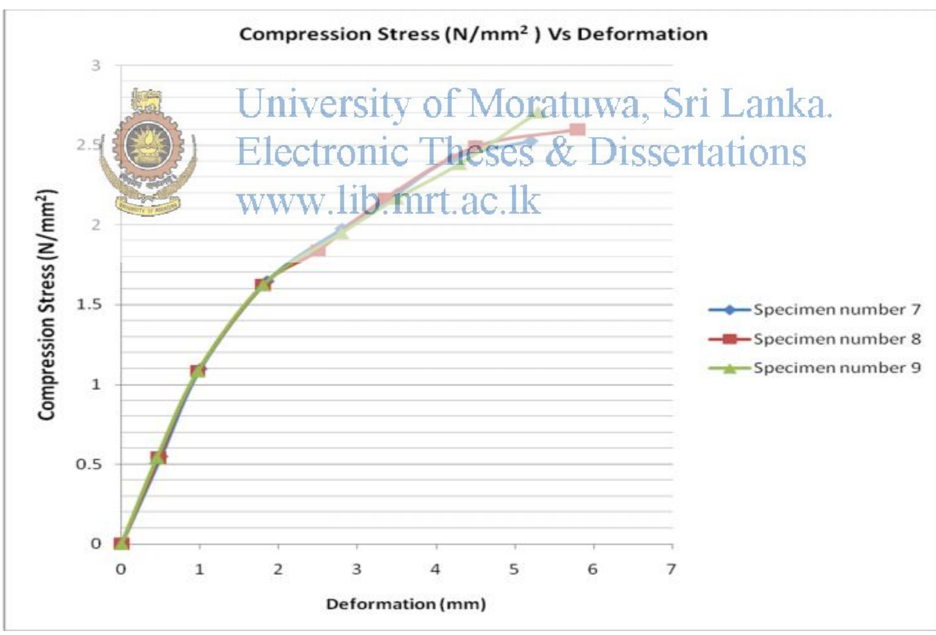
The Specimen no 9 had the length of 200 mm, height of 74 mm and depth of 121.5mm. It was loaded by jaw of loading machine. The results were observed until the ultimate behaviour. The behaviour was noted based on the visual inspections. The results are tabulated as in the following Table 15.

Figure 26-Testing in progress of Specimen no 9.

Table 15-Test results of Specimen no 9.

Force units	Calibrated reading (N)	Compression Force (N)	Compression Stress (N/mm^2)	Shorten Deformation (0.01mm)	Shorten Deformation (mm)
0	0	0	0	0	0
5	5000	5000	0.54148	45	0.45
10	10000	10000	1.08295	97	0.97
15	15000	15000	1.62443	180	1.8
18	18000	18000	1.94932	280	2.8
20	20000	20000	2.16591	350	3.5
22	22000	22000	2.3825	430	4.3
25	25000	25000	2.70739	530	5.3

The compressive stress verses deformation behaviour was plotted in a graph to identify the compressive strength of sandwich panel as follows.



Graph 7-Compressive stress versus Deformation.

Therefore the elastic compressive strength was identified as 1.6 N/mm^2.

3.3.4. Tensile load carrying capacity of sandwich material.

The tensile load caring capacity of sandwich material itself is not practical. Since the tensile capacity of the form is very small, it is considered as zero for the conservative approaches. The face material tensile properties were considered for the material property testing. The Hounsfield Tensometer installed at Department of Civil Engineering at University of Moratuwas was used for face material testing works.

3.3.4.1. Testing of sandwich face material

The two samples were prepared as following Figure 27.

Figure 27-Sandwich panel face material samples before testing

The face material had the thickness of 0.61 mm with paint and Zinc coating. The raw material thickness is 0.5 mm as per the product specifications.

3.3.4.1.1.Testing of Specimen no 10

The specimen was prepared as per the BSI standard test piece, which can be mounted to Hounsfield Tensometer. It had the total length of 63.5mm. The mounting hole had the diameter of 8mm. It was loaded by jaw of Hounsfield Tensometer. The results were observed until the ultimate behaviour. The behaviour was noted based on the visual inspections. The results are tabulated as in the following Table 16.

Figure 28- Test set up for Specimen no 10.

Table 16-Test results of Specimen no 10.

Revolutions (nos)-Displacement	Strain Ax0.085/63.5	Tensile Force (C)	Stress = Force/Area (Cx1000x9.81/0.5x12) (N/mm^2)
0	0	0	0
2	0.0026772	0.03	49.05
4	0.0053543	0.06	98.1
6	0.0080315	0.09	147.15
8	0.0107087	0.13	212.55
10	0.0133858	0.17	277.95
12	0.016063	0.19	310.65
14	0.0187402	0.21	343.35
16	0.0214173	0.23	376.05
18	0.0240945	0.23	376.05
20	0.0267717	0.23	376.05
22	0.0294488	0.23	376.05
24	0.032126	0.23	376.05
26	0.0348031	0.23	376.05
28	0.0374803	0.23	376.05
30	0.0401575	0.23	376.05
32	0.0428346	0.23	376.05
34	0.0455118	0.24	392.4
36	0.048189	0.24	392.4
38	0.0508661	0.24	392.4
40	0.0535433	0.24	392.4
42	0.0562205	0.24	392.4
44	0.0588976	0.24	392.4
46	0.0615748	0.24	392.4
48	0.064252	0.24	392.4
50	0.0669291	0.25	408.75
52	0.0696063	0.25	408.75
54	0.0722835	0.25	408.75
56	0.0749606	0.25	408.75
58	0.0776378	0.25	408.75
60	0.080315	0.25	408.75
62	0.0829921	0.25	408.75
64	0.0856693	0.25	408.75
66	0.0883465	0.25	408.75
68	0.0910236	0.25	408.75

Specimen no 10 results continue…..

Table 16-Specimen no 10 results continue…..			
Revolutions (nos)-Displacement (A)	Strain Ax0.085/63.5	Force (C)	Stress = Force/Area (Cx1000x9.81/0.5x12) (N/mm^2)
70	0.0937008	0.25	408.75
72	0.096378	0.25	408.75
74	0.0990551	0.25	408.75
76	0.1017323	0.25	408.75
78	0.1044094	0.25	408.75
80	0.1070866	0.25	408.75
82	0.1097638	0.25	408.75
84	0.1124409	0.26	425.1
86	0.1151181	0.26	425.1
88	0.1177953	0.26	425.1
90	0.1204724	0.26	425.1
92	0.1231496	0.26	425.1
94	0.1258268	0.26	425.1
96	0.1285039	0.26	425.1
98	0.1311811	0.26	425.1
100	0.1338583	0.26	425.1
102	0.1365354	0.26	425.1
104	0.1392126	0.26	425.1
106	0.1418898	0.26	425.1
111	0.1485827	0.26	425.1
116	0.1552756	0.26	425.1
121	0.1619685	0.24	392.4
126	0.1686614	0.23	376.05
131	0.1753543	0.23	376.05
136	0.1820472	0.25	408.75
141	0.1887402	0.26	425.1
146	0.1954331	0.26	425.1
151	0.202126	0.26	425.1
156	0.2088189	0.26	425.1
161	0.2155118	0.26	425.1
166	0.2222047	0.26	425.1
171	0.2288976	0.26	425.1
181	0.2422835	0.25	408.75
186	0.2489764	0.25	408.75
191	0.2556693	0.23	376.05

3.3.4.1.1.Testing of Specimen no 11

The Specimen no 11 was mounted to Hounsfield Tensometer. It had the total length of 63.5mm. The mounting hole had the diameter of 8mm. It was loaded by jaw of Hounsfield Tensometer. The results were observed until the ultimate behaviour. The behaviour was noted based on the visual inspections. The results are tabulated as in the following Table 17.

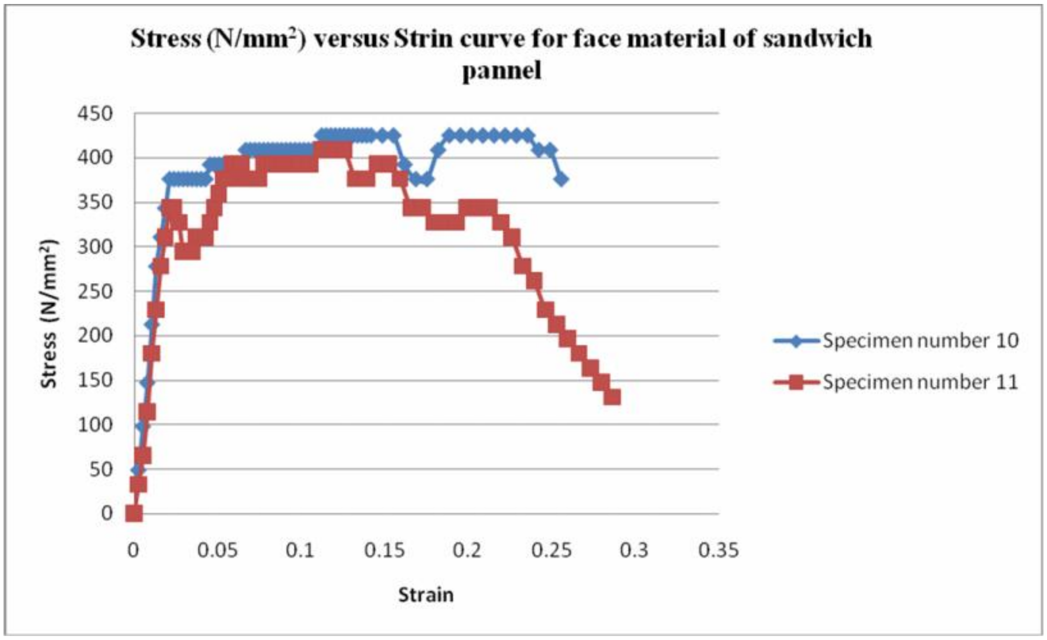
Figure 29-Test set up of Specimen no 11.

Table 17-Test results of Specimen no 11

Revolutions (nos)- Displacement (A)	Strain Ax0.085/63.5	Force (C)	Stress = Force/Area (Cx1000x9.81/0.5x12) (N/mm^2)
0	0	0	0
2	0.002677	0.02	32.7
4	0.005354	0.04	65.4
6	0.008031	0.07	114.45
8	0.010709	0.11	179.85
10	0.013386	0.14	228.9
12	0.016063	0.17	277.95
14	0.01874	0.19	310.65
16	0.021417	0.21	343.35
18	0.024094	0.21	343.35
20	0.026772	0.2	327
22	0.029449	0.18	294.3
24	0.032126	0.18	294.3
26	0.034803	0.18	294.3
28	0.03748	0.19	310.65
30	0.040157	0.19	310.65
32	0.042835	0.19	310.65
34	0.045512	0.2	327
36	0.048189	0.21	343.35
38	0.050866	0.22	359.7
40	0.053543	0.23	376.05
42	0.05622	0.23	376.05
44	0.058898	0.24	392.4
46	0.061575	0.24	392.4
48	0.064252	0.24	392.4
50	0.066929	0.23	376.05
52	0.069606	0.23	376.05
54	0.0722283	0.23	376.05
56	0.074961	0.24	392.4
58	0.077638	0.24	392.4
60	0.080315	0.24	392.4
62	0.082992	0.24	392.4
64	0.085669	0.24	392.4
66	0.088346	0.24	392.4
68	0.091024	0.24	392.4
Specimen number 11 results continue……			

Table 17-Specimen number 11 test results continue……			
Revolutions (nos)-Displacement (A)	Strain Ax0.085/63.5	Force (C)	Stress = Force/Area (Cx1000x9.81/0.5x12) (N/mm^2)
70	0.093701	0.24	392.4
72	0.096378	0.24	392.4
74	0.099055	0.24	392.4
79	0.105748	0.24	392.4
84	0.112441	0.25	408.75
89	0.119134	0.25	408.75
94	0.125827	0.25	408.75
99	0.13252	0.23	376.05
104	0.139213	0.23	376.05
109	0.145906	0.24	392.4
114	0.152598	0.24	392.4
119	0.159291	0.23	376.05
124	0.165984	0.21	343.35
129	0.172677	0.21	343.35
134	0.17937	0.2	327
139	0.186063	0.2	327
144	0.192756	0.2	327
149	0.199449	0.21	343.35
154	0.206142	0.21	343.35
159	0.212835	0.21	343.35
164	0.219528	0.2	327
169	0.22622	0.19	310.65
174	0.232913	0.17	277.95
179	0.239606	0.16	261.6
184	0.246299	0.14	228.9
189	0.252992	0.13	212.55
194	0.259685	0.12	196.2
199	0.266378	0.11	179.85
204	0.273071	0.1	163.5
209	0.279764	0.09	147.15
214	0.286457	0.08	130.8

The test results of Specimen no 10 and Specimen no 11 were plotted in one graph as follows on Graph no 8.



Graph 8-Stress versus Strain graph for sandwich skin material.

Based on above graph the face material has the elastic limit up to 340 N/mm^2. Since the core material tensile capacity is not taken in to account, the face thickness is 0.5mm.

The average tensile strength of sandwich panel = 340 x 0.5 x 2 / 121.5

= 2.79 N/mm^2.

3.4 Results for structural analysis

The experimental evaluations derived the finding of the test results as follows

The elastic shear strength limit was noted as 0.05 N/mm^2.

Young's modulus was noted as 0.23 GPa

The elastic compressive strength was identified as 1.6 N/mm^2.

The elastic tensile strength was identified as 2.79 N/mm^2.

3.5 Validation of derived results

3.5.1. Validation of Young's modulus

3.5.1.1. Instance no 1

Consider Specimen number 3 test results.

As per the Graph 3 and Table 9, when the shear force was 1485 N, the deflection was 6 mm.

From equation 26, the central deflection is given by,

$\Delta_{total} \quad = \quad (L^3/4bd^3+4.8L/bd)\ F/E$

Substitution,

L = 1.15 m

b = 0.2975 m

d = 0.1215m

E = 0.23 GPa

Δ_{total} = 5.5 mm, the result is reasonably matching.

3.5.1.1. Instance no 2

Consider Specimen no 7, From Table 14, consider compression force 10,000 N, the compressive stress was 1.08 N/mm^2

Shortening length was 0.97 mm

Since total length was 205 mm

Young's Modulus = Stress / Strain

= 1.08 N/mm^2/(0.97/205)

= 0.23 GPa

The result is exactly matching the experimental output.

The Young's Modulus values are reasonably matching.

3.6 The technical parameters for structural analysis

The following parameters shall be accompanied for the structural analysis purposes

Young's Modulus 0.23 GPa.

Poisson's ratio 0.05

Coefficient of thermal expansion 6.5x10^{-5} m/mk.

The above parameters and SIP shear strength, compressive strength and tensile strength parameters derived on Chapter 3, were used to carryout the model analysis detailed in Chapter 4.

CHAPTER-4

STRUCTURAL ANALYSIS

4.1. General Introduction

The goal of this research is to find the validity of construction of domestic structures from polyurethane sandwich material for walls, slabs and roofs. The two numbers of models comprising of two story house and two story flat structures will be analysed to see the possibility of structural validity of the material.

The house models are typical to Sri Lanka. By use of these models almost all the typical housing units would be covered. The spans have been limited to 3.5 m typically.

4.2. Structural Modelling

The two types of two story houses were modelled for the structural analysis. One model was a common individual two story house. Its rooms have the wall and slab elements with 3 m to 3.5 m range. But in the living area the double wall height of 6 m were also modelled. The maximum bay size was limited to 6 m by 6 m.

The other two story house model was from a cluster housing scheme. It has four no of houses at ground floor and four no of houses at upper floor. This plan was typical for the "Urban regeneration project" implemented by Urban Development Authority of Sri Lanka. The all walls and slab element have the maximum free spanning of 3.5 m. There is a corridor at the middle of the unit. It has the one way spanning slab of 6 m. On the modelling it was modelled as per the plan.

For the finite element analysis of the models, the finite element was limited to 0.5 m x 0.5 m for both models as it is a reasonable approach. All the elements were modelled as thick shell as the panel flexural capacity is very low. It was also accounted for the calculations. The analysed results deliver more conservative results as the staircases were not considered for the structural analysis. In practice when there is a structural opening, its opening frame is established as the reinforcing form of the weaken structural element. The stiffness generated by the wall opening reinforcing elements were also not modelled and not considered for the structural analysis. Therefore the generated results are more conservative.

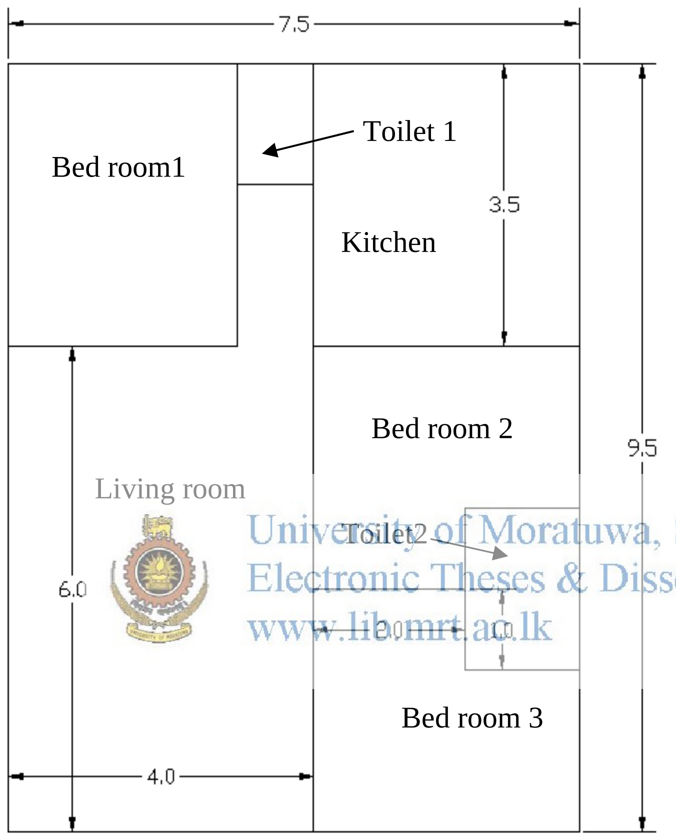
As per the clause 2.16 extraction of "Housing Construction Guidelines" [17] published by the Society of Structural Engineers, Sri Lanka. The additional robustness was requested in perimeter walls and roof level brazing were also requested. Therefore in these two models, 150mm x150mm x10mm steel angle frame was introduced. It moves along the wall top level. Since the corridor model at cluster house is more slender and shown very high deflection, 150mm x 75mm x 10mm channel section was introduced only to the free span area. In addition to comply "Housing Construction Guidelines" [17], the above angle was placed in the lintel level to comply the requirements in both models.

4.2.1. Model of two story housing unit

The two-story house model was selected for the analysis. The floor plans are as below in Figure 30 and Figure 31.

4.2.1.1. Ground floor plan

The ground floor plan of the two-story house unit is shown bellow in Figure 30.

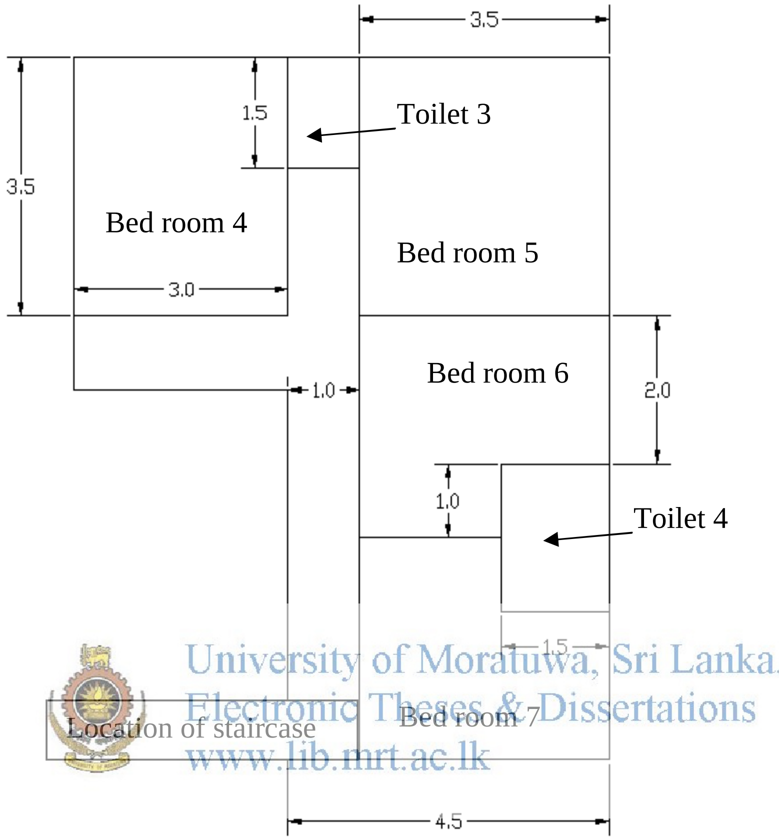


All measurements in metres.

Figure 30- Ground floor plan of two story housing unit.

4.2.1.2. Upper floor plan

The upper floor plan of the house model is as below in Figure 31.



All measurements in metres.

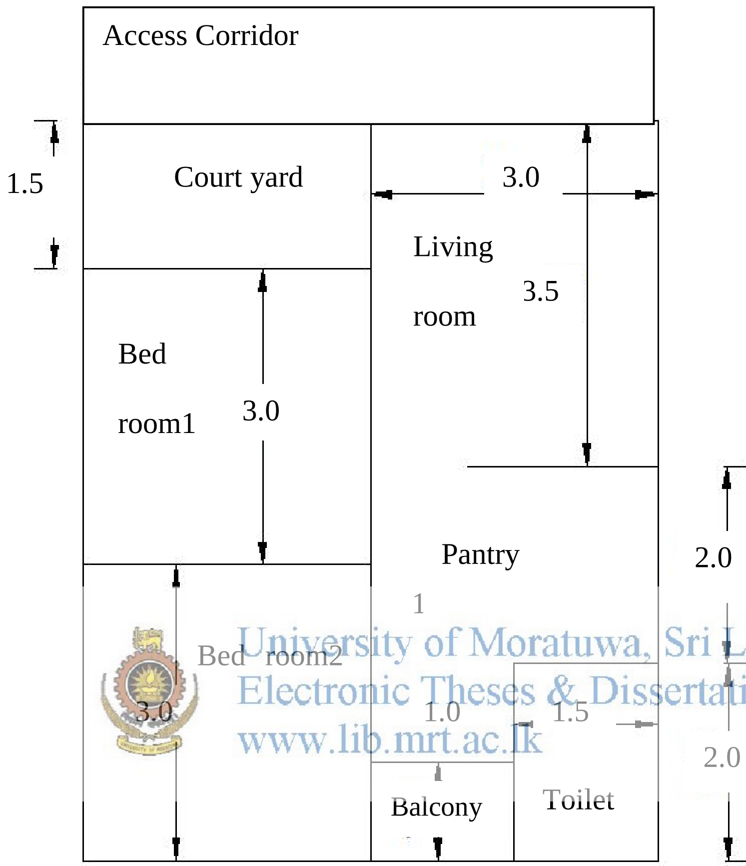
Figure 31-Upper floor plan of two story housing unit.

4.2.2. Model of Two story cluster dwelling unit

The typical cluster house plan was used for this works. It comprised of four numbers of dwelling units for ground floor and four numbers of dwelling units for upper floor. The stair case was not included for analysis as it increases the robustness of the structure and it cannot be made from SIP. It needs to be constructed from steel. Therefore the output of the works is more conservative.

4.2.2.1 Typical plan of dwelling unit

The typical plan of a dwelling unit is shown in Figure 32 as below.

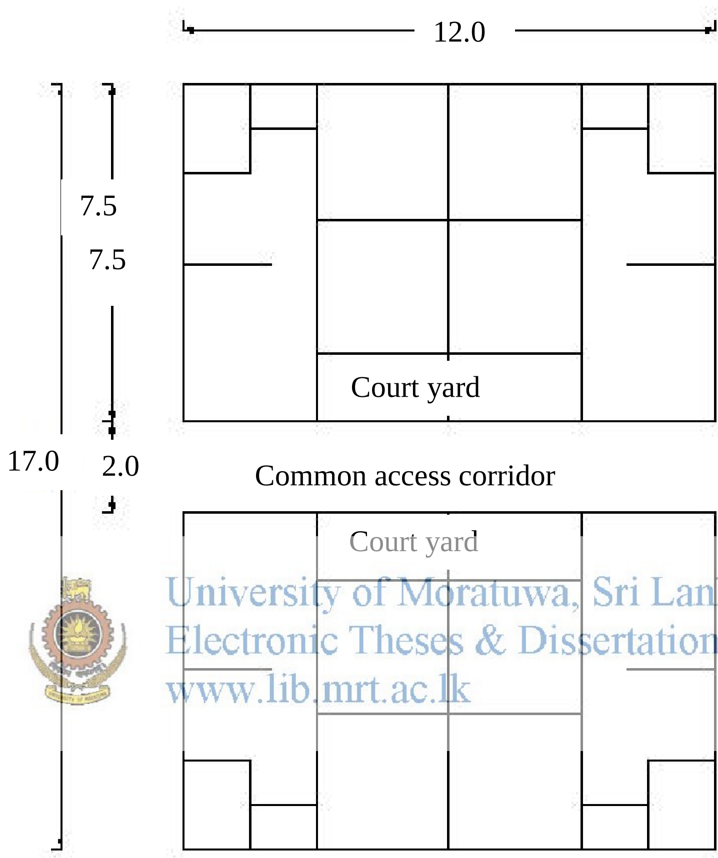


All measurements are in metres.

Figure 32-Typical floor plan of a dwelling unit.

4.2.2.2. The typical floor plan for 4 numbers of cluster dwelling units

The typical floor plan of 4 no of cluster dwelling units is shown in Figure 33 as below.



All measurements in metres

Figure 33-Typical floor plan of 4 nos of cluster dwelling units.

4.3. Design loads

Since the construction material is from a light weight material, the lateral loads caused by the external environment may run a typical role on structural viability. Therefore standard loading was used for the analysis.

The loads for the design works shall be derived from British Standard loading code BS6399 part 1:1996 as follows.

4.3.1. Typical loads

Typical live load 1.5 kN/m^2 for self contained dwelling units

Floor finishing 0.15 kN/ m^2 for light weight material

Roof top live load 0.75 kN/ m^2 for accessible roof

Roof finishing & insulations 0.2 kN/ m^2

Ceiling & Services 0.15 kN/ m^2 for light weight materials

The wind loading was calculated from British standard code of practice for design of building, CP3: Chapter V-2:1972

4.3.2. Wind load calculations

4.3.2.1. Basic wind speed

Assuming that this construction is made on Western province of Sri Lanka,

As per wind zone 3, from table 3.1 basic wind speeds for Sri Lanka 1972,

Assuming that the structure is a normal structure,

Basic wind speed, V = 33.5 m/s.

4.3.2.2. Load calculations

Cl. 4.3(2) CP 3- Chapter V-2:1972

$$V_s = VS_1S_2S_3$$

Cl. 5.4. CP 3- Chapter V-2:1972

Assuming average slope of the ground does not exceed 0.05 within a kilometre radius of the site, the terrain may be taken as level and the topography factor S_1 should be taken as 1.0

Cl. 5.5.2. & Table 3, CP 3- Chapter V-2:1972

Ground roughness factor S_2 = 0.72 assuming country with many wind breakers, small towns, out skirts of large cities.

Cl. 5.6 & Figure 2, CP 3- Chapter V-2:1972

Assuming that the structure does last more than 50 years and the probability of exposure to high wind is 0.63.

Statistical factor S_3 = 1

Dynamic pressure of wind,

Cl. 6. CP 3- Chapter V-2:1972

$q = kV_s^2$

k= 0.613 for SI units

$q=0.613 \times (33.5x1x0.72x1)^2$

$q=0.356\ kN/m^2$.

4.3.2.3. Calculations of pressure for walls and roofs

Table 7, CP 3- Chapter V-2:1972

For two story housing unit,

h= 6 m

w=7.5m

l=9.5m

h/w=0.8

l/w=1.26

Table 7, CP 3- Chapter V-2:1972

Considering all wind angles, C_p= -1.1

Therefore wind loading for surface = 0.356 x 1.1 = 0.3916 kN/m^2.

Let wind force be 0.4 kN/m^2 for all walls.

Calculations of pressure coefficients for pitch roofs.

Roof angle = $\tan^{-1}(1.5/3)$ = 26.5’ for windward direction.

Roof angle = $\tan^{-1}(1.5/3.5)$ = 23.1’ for wind-rear direction.

Table 8, CP 3- Chapter V-2:1972

C_{pe} = 0.8 for worst scenarios of all angle of wind direction.

Therefore wind loading for surface = 0.356 x .8 = 0.284 kN/m^2.

Let wind force be 0.3 kN/m^2 for all roofs.

4.3.2.4. For cluster dwelling flat walls and roof.

h= 6 m.

l=17 m.

W=12 m.

h/w=0. 5

l/w=1.42

Table 7, CP 3- Chapter V-2:1972

Considering all wind angles, C_p = -0.8

Therefore wind loading for surface = 0.356 x 0.8 = 0.28 kN/m^2.

Let wind force be 0.3 kN/m^2 for all walls.

Calculations of pressure coefficients for flats' roofs.

Table 10, CP 3- Chapter V-2:1972

Let l/w be 1.5, b/d be 1.5 and h/b=0.5

C_f = 0.95 for worst scenarios of flat roof independent of wind direction.

Therefore wind loading for surface = 0.356 x .95 = 0.338 kN/m^2.

Let wind force be 0.35 kN/m^2 for flat roof.

4.4. Modelling of two story house

The three dimensional model of two story house is shown in Figure 34 as below.

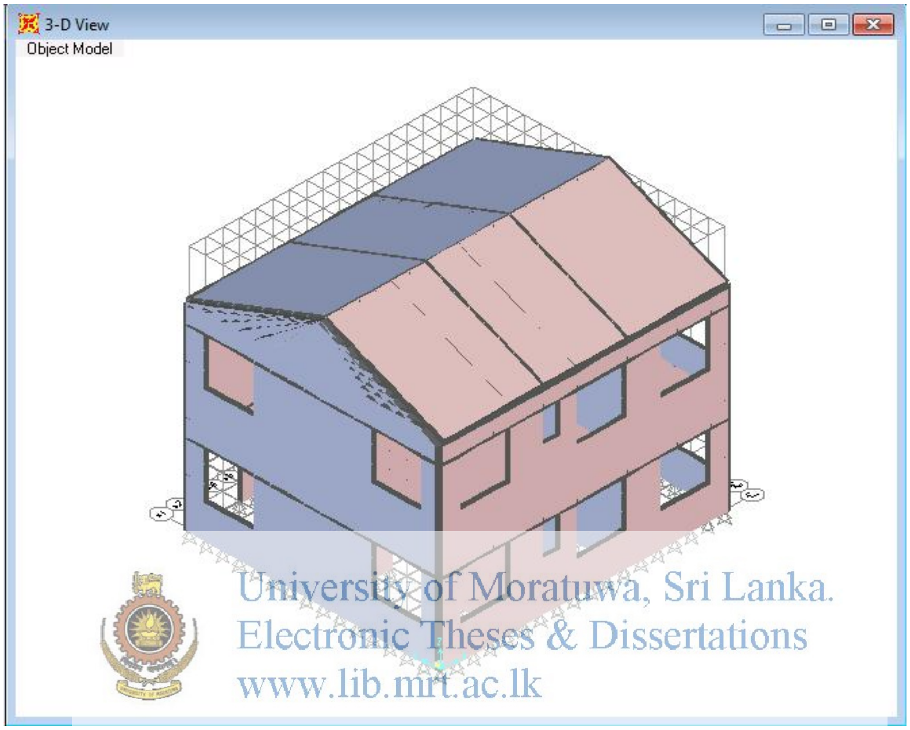


Figure 34-Three dimensional model of two story house.

4.4.1. Load assignments

4.4.1.1. Loading for Roof

The roof elements load assignments were as follows.

Live load-0.75 kN/m^2

Finishes -0.15 kN/m^2

Wind force -0.3kN/m^2

Self load of material-4.3kN/m^3

4.4.1.2. Wind word direction roof load assignment

The Figure 35 shows the wind ward direction roof element load assignment as below.

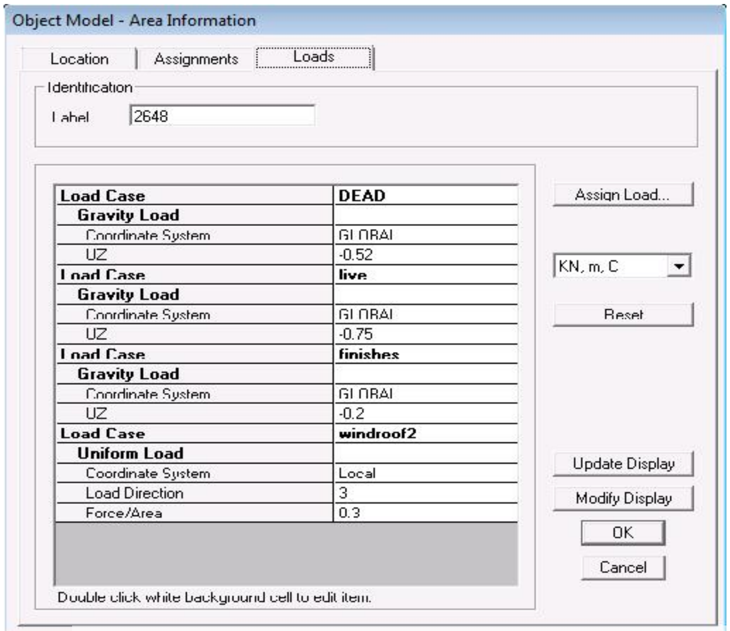


Figure 35-Wind-word direction roof element load assignment

4.4.1.3. Loading for Walls

Wind force -0.4 kN/m^2

Self load of material-4.3kN/m^3

4.4.1.4. Loading for Slab

Live load – 1.5 kN/m^2

Finishes – 0.15 kN/m^2

Self load of material-4.3kN/m^3

4.4.2. Load combinations

4.4.2.1. Combination 1

0.9 dead load+1.4 wind load

4.4.2.2. Load combination 2

1.6 Live load +1.4 dead load+1.4 finishing load+1.4 service load

4.4.2.3. Load Combination 3

1.2 Live load+1.2 dead load+1.2finishing load+1.2 wind load

4.4.3.1. Slab element load assignment

The Figure 36 shows the slab element's load assignment.

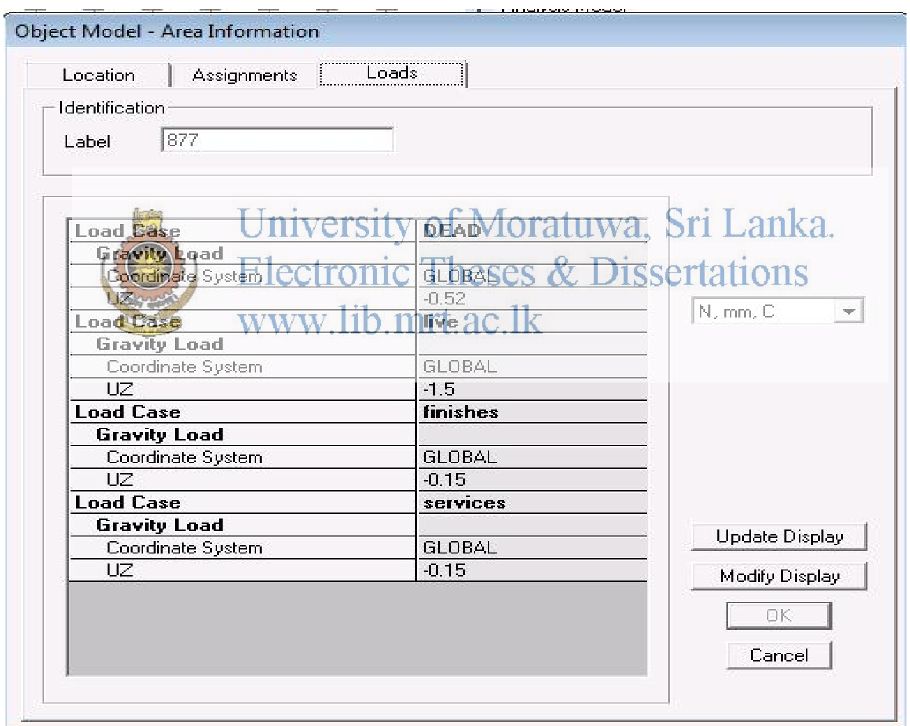


Figure 36-Slab element load assignment

4.5. Verification of SAP 2000 finite element analysis report for two story house

4.5.1. Manual calculation of surface stress

The first floor slab at Bed room number 5 is considered. It has 3.5 m span in both directions. Adjacent two sides are discontinued.

4.5.1.1. Elastic Analysis

Since SAP 2000 is delivering elastic analysis results, to compare the results the manual analysis also shall be from the same concept. Therefore elastic analysis is made from Reinforced concrete designer's hand book;[26].

Unit load

Equation 27

$$w = 1.4 \times \text{Dead loads} + 1.6 \times \text{Live loads}$$

Considered loads

Slab weight	-	0.52 kN/m^2
Live load	-	1.5 kN/m^2
Floor finishes	-	0.15 kN/m^2
Ceiling finishes and Services	-	0.15 kN/m^2

Therefore from equation 27

$$w = 1.4(0.52+0.15+0.15) + 1.6\times1.5$$

$$= 3.548\ kN/m^2.$$

From table 50;[26]

α_{x3} = 0.3

α_{y3} = 0.3

Where

α_{x3}, α_{y3} – Coefficients defined according to support conditions

From clause 14.2.2;[26]

Equation 28

ξ_1 = 1- 0.833k2/(1+k2)

Where

k is the slab span ratio,

k = 1

Therefore

ξ_1 = 0.5835

From clause 14.2.2;[26]

Equation 29

M_{dx} = $\xi_1 \alpha_{x3}(wl_x^2/8)$

Equation 30

M_{dy} = $\xi_1 \alpha_{y3}(wl_y^2/8)$

Therefore

M_{dx} = $0.5835x0.3x3.548x3.5^2/8$

= 0.951 kNm/m.

$M_{dy} = 0.951$ kNm/m.

From elastic bending relationships;[20],

Equation 31

$$M/I = \sigma/y.$$

$$\sigma = My/I.$$

$$y = 0.1215/2$$

$$= 0.06 \text{ m}.$$

$$I = bd^3/12$$

$$= 1x0.1215^3/12$$

$$= 1.5x10^{-4} \text{ m}^4.$$

Therefore from substitutions,

$$\sigma_{sx} = 0.951x10^3x0.06/(1.5x10^{-4}).$$

$$= 380 \text{ kN/m}^2.$$

$$= 0.38 \text{ N/mm}^2.$$

$$\sigma_{sy} = 0.951\ x10^3x0.06/(1.5x10^{-4}).$$

$$= 380 \text{ kN/m}^2.$$

$$= 0.38 \text{ N/mm}^2.$$

4.5.1.2. Collapse mechanism analysis

To benchmark the SAP 2000 analysis results, collapse mechanism also used. To get the collapsed mechanism analysis, BS 8110- Part 1:1977; [25] was used. It was considered that, the slab is restrained slab. As per Clause 3.5.3.4[25]

Equation 32

$$m_{sx} = \beta_{sx} n l_x^2$$

Where

m_{sx} - maximum design ultimate moments either over supports or at mid span on strips of unit width and span lx

β_{sx} - moment coefficient

n - Total design ultimate load per unit area

l_x - length of shorter side

Equation 33

$$m_{sy} = \beta_{sy} n l_x^2$$

Where

m_{sy} - maximum design ultimate moments either over supports or at mid span on strips of unit width and span l_y

β_{sy} - moment coefficient

From table 3.14[25];

Consider positive moment at mid span

β_{sx} = 0.036

β_{sy} = 0.034

Equation 34

n = 1.4 x Dead loads + 1.6 x Live loads

Considered dead loads

Slab weight	-	0.52 kN/m^2
Live load	-	1.5 kN/m^2
Floor finishes	-	0.15 kN/m^2
Ceiling finishes and Services	-	0.15 kN/m^2

Therefore from equation 29

n = 1.4(0.52+0.15+0.15) + 1.6x1.5

= 3.548 kN/m^2

Therefore from equation 27

m_{sx} = 0.036 x 3.548 x 3.5^2

= 1.56 kNm/m

Therefore from equation 28

m_{sy} = $0.034 \times 3.548 \times 3.5^2$

= 1.47 kNm/m

From elastic bending relationships;[20],

Equation 35

$M/I = \sigma/y.$

$\sigma = My/I.$

$y = 0.1215/2$

$= 0.06$ m.

$I = bd^3/12$

$= 1x0.1215^3/12$

$= 1.5x10^{-4}$ m^4.

Therefore from substitutions

$\sigma_{sx} = 1.56x103x0.06/(1.5x10^{-4}).$

$= 624kN/m^2.$

$= 0.624$ N/mm^2.

$\sigma_{sy} = 1.47x103x0.06/(1.5x10^{-4}).$

$= 588kN/m^2.$

$= 0.588$ N/mm^2.

4.5.2. SAP 2000 finite element analysis of two story house

4.5.2.1. Surface stresses for Bed room number 5, floor slab – in main direction

Figure 37 shows the surface stresses in main direction for the bottom surface for the load combination 2.

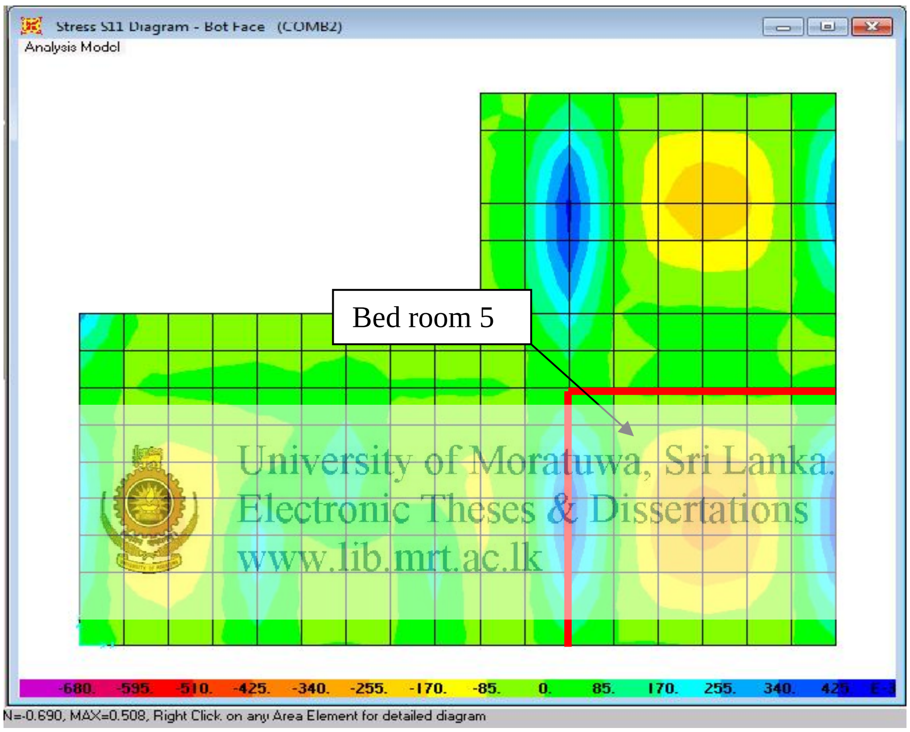


Figure 37- Surface stresses of first floor slab in main direction.

It showed minimum of 0.69 N/mm^2 and the maximum of 0.508 N/mm^2. The Slab bottom surface stress in Bedroom 5 on grid direction shows 0.42 N/mm^2.

Since Elastic analysis delivered surface stress of 0.38 N/mm^2 and Collapse mechanism delivered surface stress as 0.624 N/mm^2. It shows that, finite element elastic analysis is fairly correct. The surface stresses depend on grid spacing. For this analysis grid spacing of 0.5 m was used. Therefore if further refine analysis is made, by introducing further small grid spacing, a fair result could be achieved.

4.5.2.2. Surface stresses for Bed room number 5, floor slab – in secondary direction

Figure 38 shows the surface stresses in secondary direction for the bottom surface for the load combination 2.

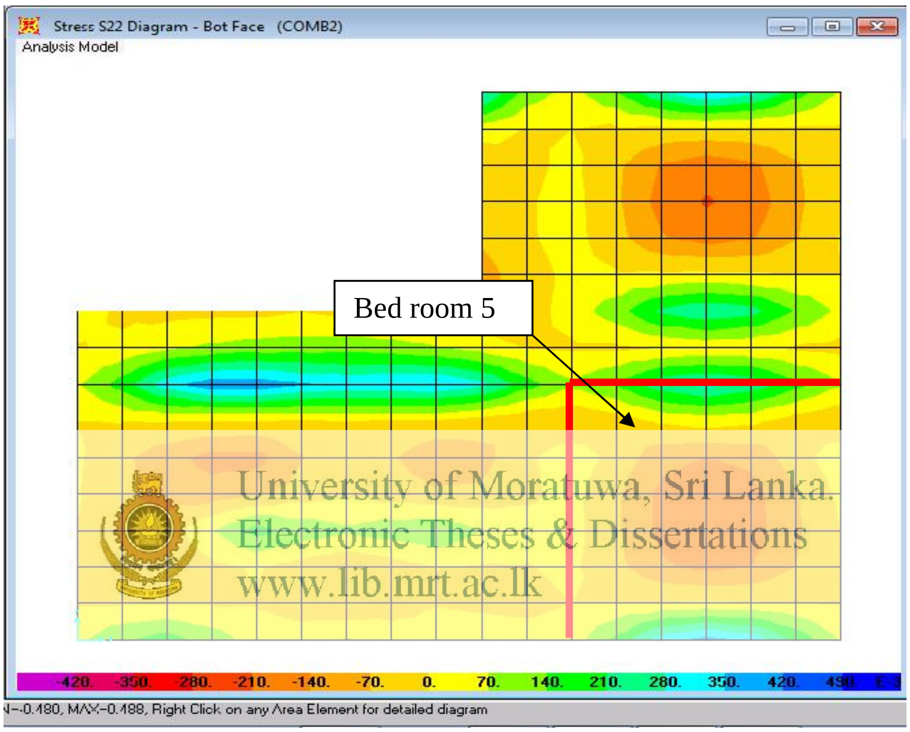


Figure 38-Surface stresses of first floor slab in secondary direction.

It showed minimum of 0.48 N/mm^2 and the maximum of 0.488 N/mm^2. The Slab bottom surface stresses in Bedroom 5 on grid direction shows 0.28 N/mm^2.

Since Elastic analysis delivered surface stress of 0.38 N/mm^2 and Collapse mechanism delivered surface stress as 0.588 N/mm^2. It shows that, finite element elastic analysis is fairly correct. The surface stresses always interconnected with grid spacing. For this analysis grid spacing of 0.5 m was used. Therefore if further refine grid spacing is used, a fair result could be achieved. The finite element analysis is correct.

4.5.3. Conclusion on SAP 2000 analysis and Manual analysis

Since the manual elastic analysis and computerised elastic analysis may use different techniques and science, the output may not be same, but fairly a reasonable solution can be achieved. The technique adopted on each analysis is a form of reputed science. But in all cases the results may not coincide. In addition, the manual analysis on Elastic theory and Failure criteria gives the minimum results and maximum results respectively according to manual calculation techniques. It does not mean that the SAP 2000 results are out of the boundaries. By experiencing the upper and lower results from manual calculations the degree of accuracy on acceptability can be derived.

4.6. Analysed results of two story house

4.6.1. Results for load combination 1

4.6.1.1.Shell stresses for load combination 1

Figure 39 shows the shell stresses for load combination 1 as below.

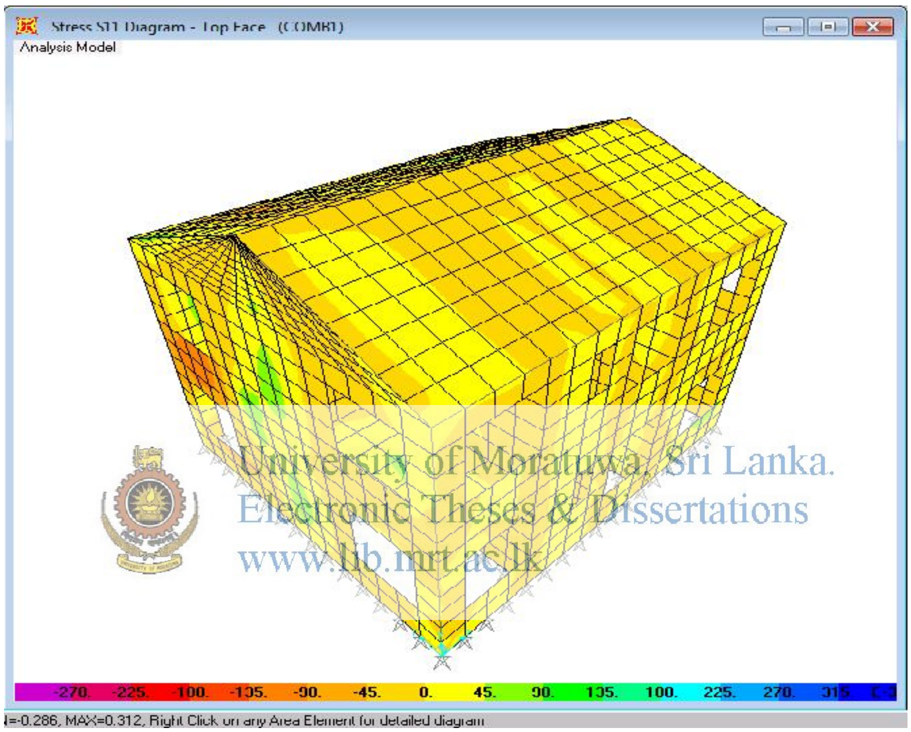


Figure 39-Shell stresses for load combination 1.

The minimum surface stress was - 0.286 N/mm^2 and the maximum surface stress was 0.312 N/mm^2.

Since compressive stress is limited to 1.6 N/mm^2 and the tensile stress is limited to 2.79 N/mm^2, this house model is safe for load combination 1 against shell stresses.

4.6.1.2.Shear stresses for load combination 1

Figure 40 shows the shear stresses for load combination 1 as below.

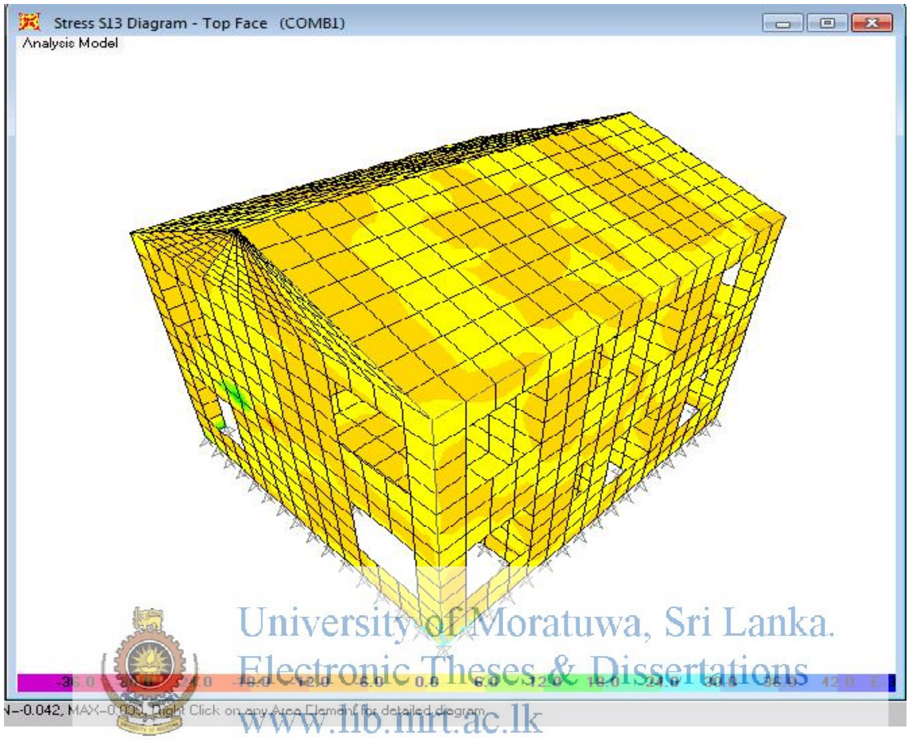


Figure 40-Shear stresses for load combination 1.

The shear stress varies from - 0.042 N/mm^2 to 0.039 N/mm^2.

Since shear stress is limited to 0.05 N/mm^2, this house model is safe for load combination 1 against shear.

4.6.2. Results for load combination 2

4.6.2.1. Shell stresses for load combination 2

Figure 41 shows the shell stresses for load combination 2 as below.

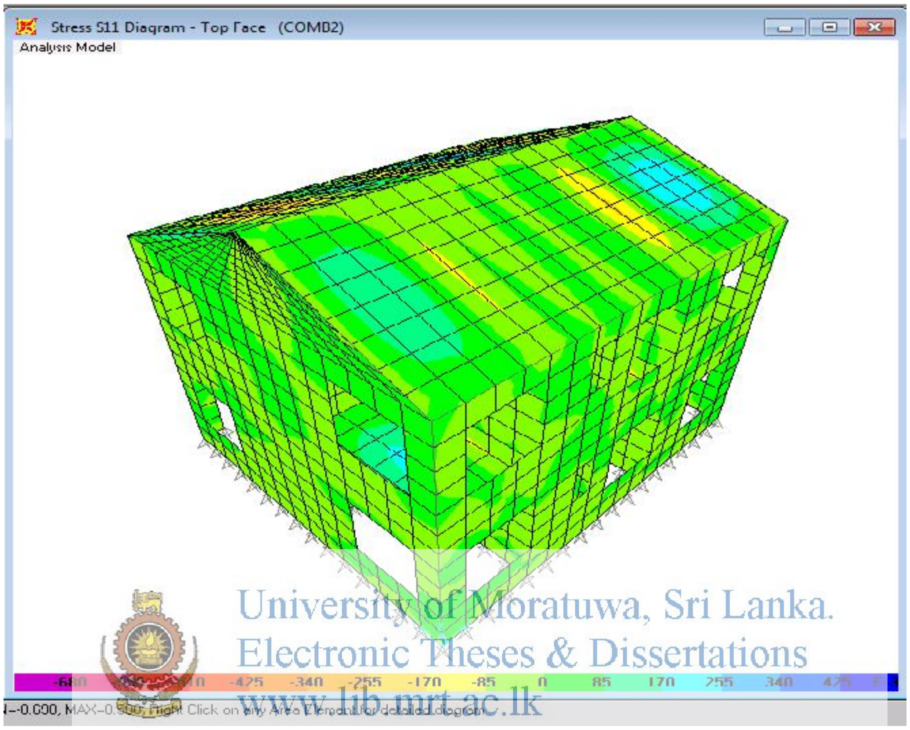


Figure 41-Shell stresses for load combination 2.

The minimum surface stress was - 0.69 N/mm^2 and the maximum surface stress was 0.508 N/mm^2.

Since compressive stress is limited to 1.6 N/mm^2 and the tensile stress is limited to 2.79 N/mm^2, this house model is safe for load combination 2 against shell stresses.

4.6.2.2. Shear stresses for load combination 2

Figure 42 shows the shear stresses for load combination 2 as below.

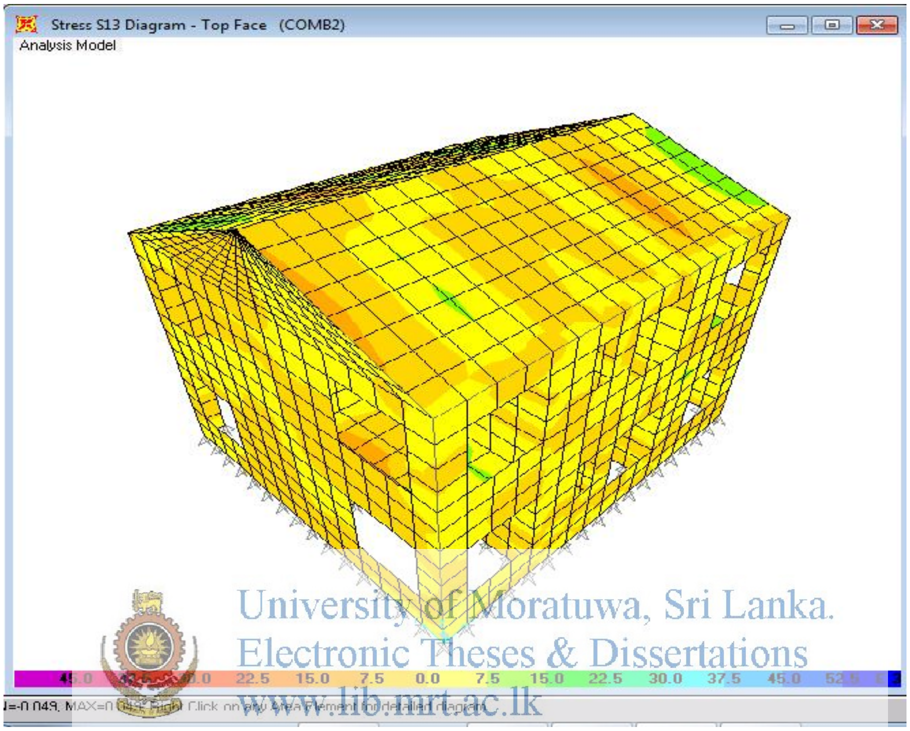


Figure 42- Shear stresses for load combination 2.

The shear stress varies from - 0.049 N/mm^2 to 0.049 N/mm^2.

Since shear stress limited is 0.05 N/mm^2, and the real stress is less than 0.049 N/mm^2 as it depends on grid pattern, this house model is safe for load combination 2 against shear.

4.6.3. Results for load combination 3

4.6.3.1. Shell stresses for load combination 3

Figure 43 shows the shell stresses for load combination 3 as below.

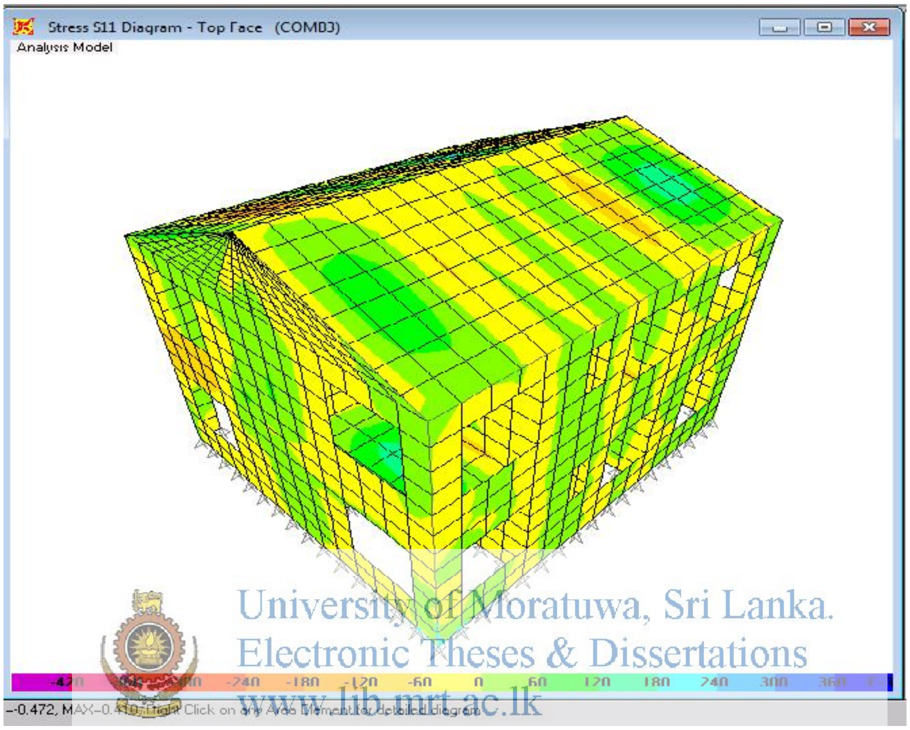


Figure 43-Shell stresses for load combination 3.

The minimum surface stress was - 0.472 N/mm^2 and the maximum surface stress was 0.41 N/mm^2.

Since compressive stress is limited to 1.6 N/mm^2 and the tensile stress is limited to 2.79 N/mm^2, this house model is safe for load combination 3 against shell stresses.

4.6.3.2. Shear stresses for load combination 3

The below Figure 44 shows the shear stresses for load combination 3.

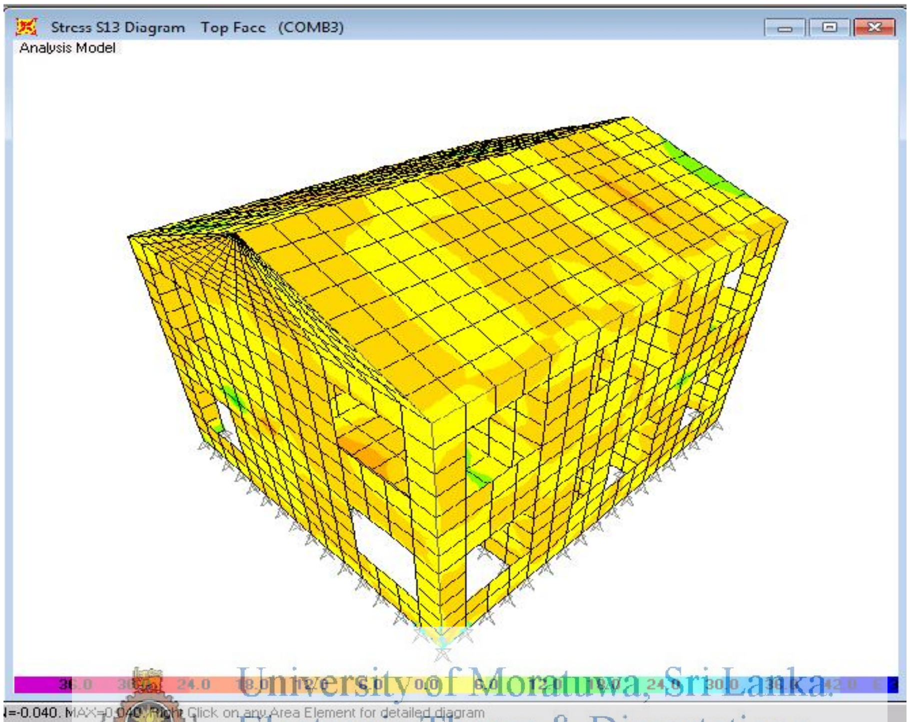


Figure 44-Shear stresses for load combination 3

The shear stress varies from - 0.04 N/mm^2 to 0.04 N/mm^2.

Since shear stress limited is 0.05 N/mm^2, this house model is safe for load combination 3 against shear.

4.6.4. Deflection check

4.6.4.1. Deflection check against lateral loads

The Figure 45 shows the deflections against lateral loading as below.

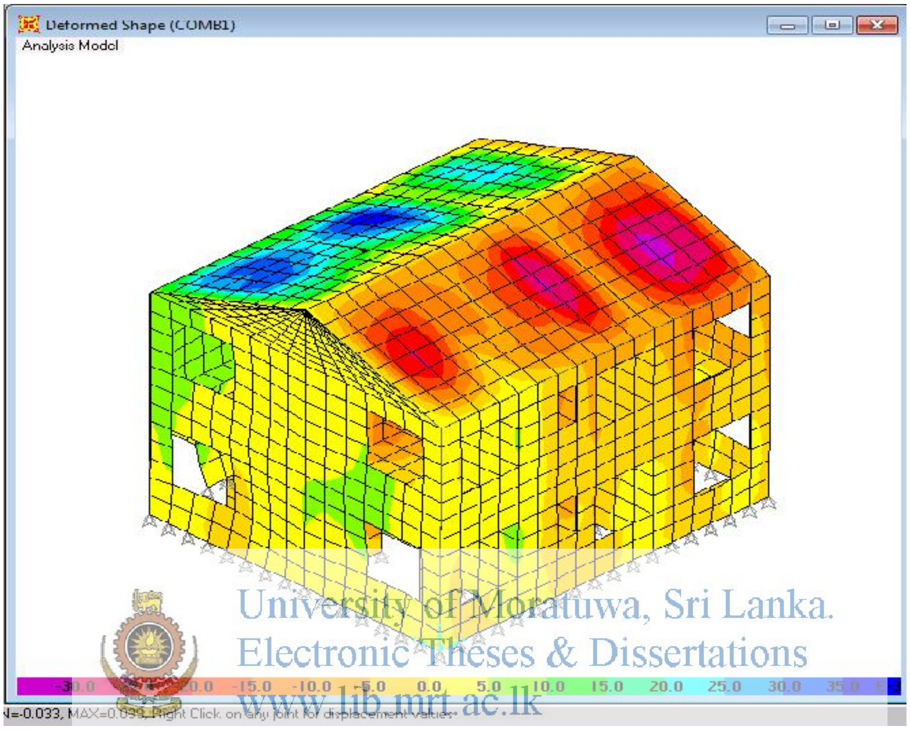


Figure 45-Deflection check against lateral loading.

It showed the minimum of -33 mm deflection and maximum of 39 mm deflection.

The deflection check against lateral loading showed that the absolute maximum deflection as 39mm.

The criteria for deflection on wall as per clause 2.10[14] = Span/100 = 6000/100

= 60 mm

Therefore the overall deflection check does comply the “European Recommendations for Sandwich Panel Part 1: Design”[14].

4.6.4.2. Deflection check against short term loading

The Figure 46 shows the deflections against short-term loadings as below.

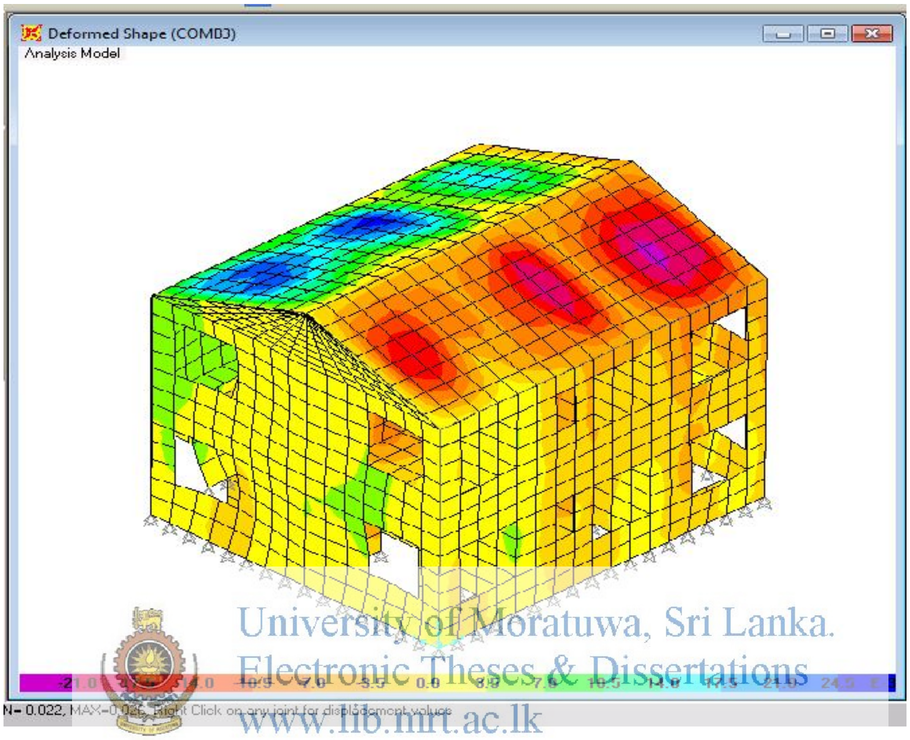


Figure 46- Deflection check against short term loading

It showed minimum of -22mm deflection and maximum of 26mm deflection.

Therefore the absolute maximum deflection showed 26 mm deflection as per above.

As per the clause 2.10 guide line on “European recommendation for sandwich panel part 1; Design” [14].

The short term deflection checks shall be less than Span / 200

Therefore the maximum deflection = 6000 /200 = 30 mm

Therefore the short term deflection check is passed.

4.6.4.3. Deflection check against long term loading

The Figure 47 shows the deflections against long-term loadings as below.

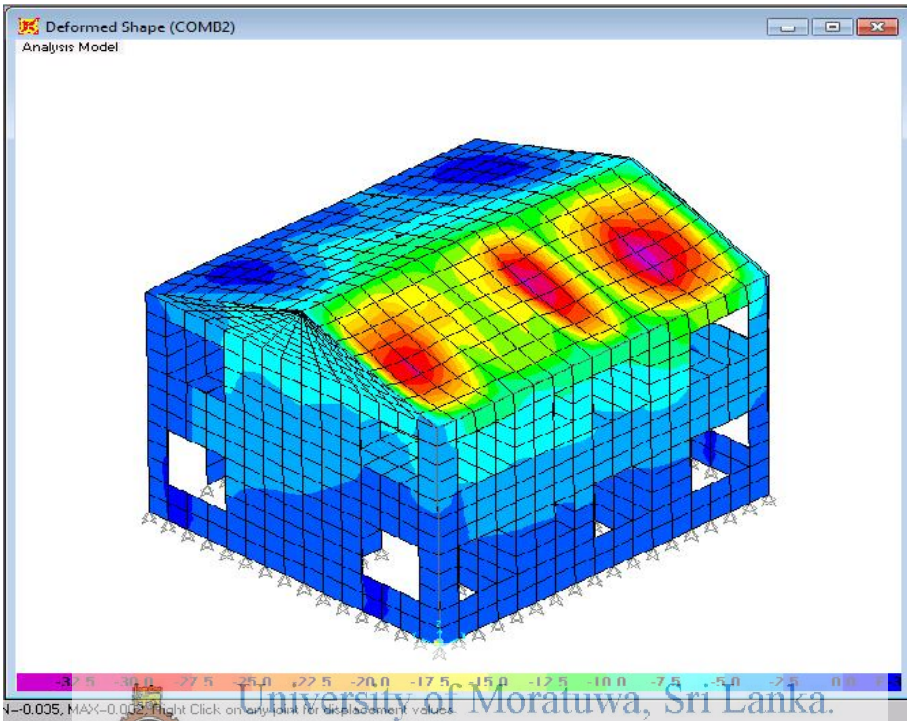


Figure 47-Deflection check against long term loading.

It showed minimum of -35 mm deflection and maximum of 2 mm deflection.

The deflection check against long term loading showed 35 mm deflection as the absolute maximum.

As per clause 2.10[14] the threshold of deflection is Span/100

Therefore the maximum deflection = 4000 / 100

= 40 mm

Therefore this design comply the “European recommendation for sandwich panel part 1;Design”[14].

4.7. Conclusions on two story house modelling and analysis results with material capacities.

Based on above analysis results in three cases, it can be concluded that the shell stresses are ranged from -0.69 N/mm^2 to 0.508 N/mm^2. This range is in the elastic limits of sandwich panel. As SIP elastic compressive stress limit is 1.6 N/mm^2. The elastic tensile stress limit is 2.79 N/mm^2

Since the shear stresses range from -0.049 N/mm^2 to 0.049 N/mm^2. The SIP material elastic shear stress limit is 0.05 N/mm^2 and the finite element gives the extreme case, the material shear capacity is adequate for safe behaviour under above three cases.

The deflection checks also displayed the deflection as 35 mm for long term deflection, 26 mm for short term deflection and 39mm for lateral deflections. The limits are 40 mm for the long term loading, 30 mm for short term loading, 60 mm for lateral loading. Therefore the two story house model comply the serviceability limit checks.

It can be noted that, two story house model comply the all design parameter checks related to SIP wall, slab and roof constructions as load bearing elements.

4.8. Modelling of two story cluster houses.

The three dimensional model of the two story cluster house is shown in the Figure 48 as below.

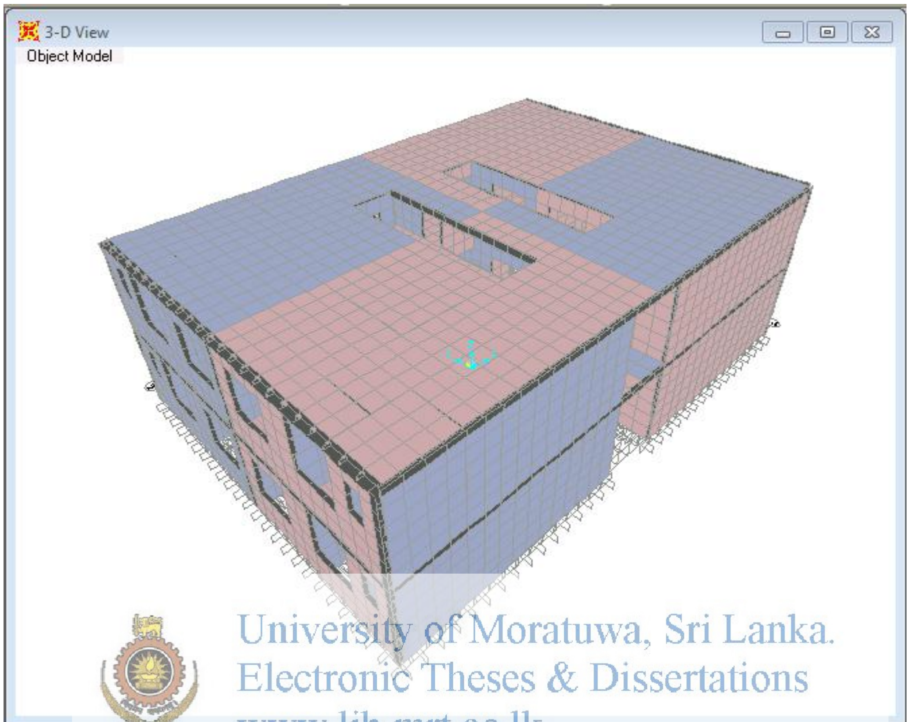


Figure 48-Model of cluster houses (8 nos)

4.8.1. Load assignments

4.8.1.1. Loading for Roof slab

The roof slab element load assignment was as below.

Live load-0.75 kN/m^2

Finishes -0.15 kN/m^2

Wind force -0.35 kN/m^2

Self load of material-4.3kN/m^3

4.8.1.2. Loading for Walls

The wall elements loading were as below.

Wind force -0.3 kN/m^2

Self load of material- 4.3kN/m^3

4.8.1.4. Loading for Slab

The slab element's load assignment was as below.

Live load – 1.5 kN/m^2

Corridor load – 3.0 kN/m^2

Finishes – 0.15 kN/m^2

Self load of material-4.3kN/m^3

4.8.2. Load combinations

4.8.2.1. Combination 1

1.6 Live load +1.4 dead load+1.4 finishing load

4.8.2.2. Load combination 2

0.9 dead load+1.4 wind load

4.8.2.3. Load Combination 3

1.2 Live load+1.2 dead load+1.2finishing load+1.2 wind load

4.8.3. Slab element load assignment.

The slab element's load assignment on the model is shown as below in Figure 49.

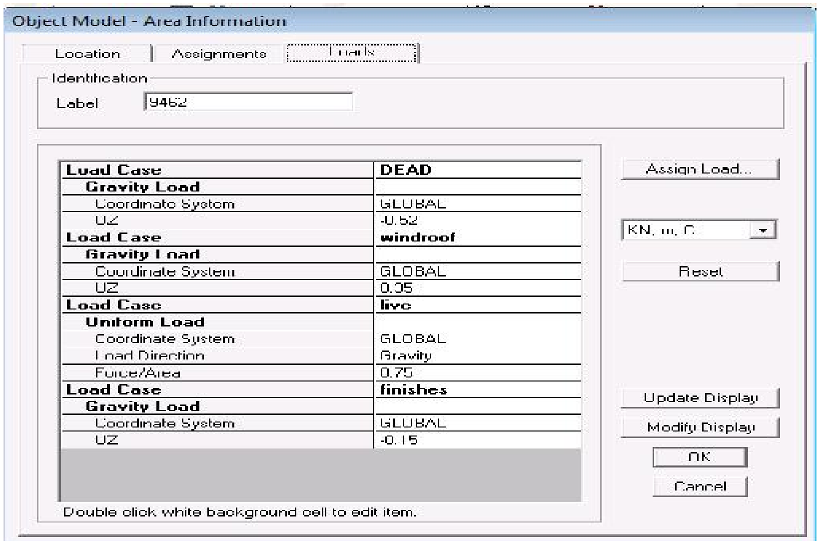


Figure 49-Slab element's load assignment.

4.9. Verification of SAP 2000 finite element analysis report for cluster house model

4.9.1. Manual calculation of surface stress

The first floor slab at Bed room number 2 is considered. It has 3 m span in both directions. Adjacent one side is discontinued.

4.9.1.1. Elastic Analysis

Since SAP 2000 is delivering elastic analysis results, to compare the results the manual analysis also shall be from the same concept. Therefore elastic analysis is made from Reinforced concrete designer's hand book;[26].

Unit load

From Equation 36

w = 1.4 x Dead loads + 1.6 x Live loads

Considered loads

Slab weight	-	0.52 kN/m^2
Live load	-	1.5 kN/m^2
Floor finishes	-	0.15 kN/m^2
Ceiling finishes and Services	-	0.15 kN/m^2

Therefore from equation 27

$$w = 1.4x(0.52+0.15+0.15) + 1.6x1.5$$

$$= 3.548 \text{ kN/m}^2$$

From table 50;[26]

α_{x3} = 0.3

α_{y3} = 0.3

From clause 14.2.2;[26]

From Equation 37

$$\xi_1 = 1\text{- }0.833k^2/(1+k^2)$$

Since k = 1

Therefore, ξ_1 = 0.5835

From clause 14.2.2;[26]

From equation 38

$$M_{dx} = \xi_1\, \alpha_{x3}(\ wl_x^{\ 2}/8\ )$$

From equation 39

$$M_{dy} = \xi_1\, \alpha_{y3}(\ wl_y^{\ 2}/8\ )$$

Therefore

$$M_{dx} = 0.5835x0.3x3.548x3^2/8$$

$$M_{dy} = 0.698 \text{ kNm/m}$$

From elastic bending relationships;[20],

Equation 40

$$M/I = \sigma/y.$$

$$\sigma = My/I.$$

$y = 0.1215/2$

$= 0.06$ m.

$I = bd^3/12$

$= 1x0.1215^3/12$

$= 1.5x10^{-4}$ m^4.

Therefore from substitutions

$\sigma_{sx} = 0.698x10^3x0.06/(1.5x10^{-4})$.

$= 0.27$ N/mm^2.

$\sigma_{sy} = 0.698$ $x10^3x0.06/(1.5x10^{-4})$.

$= 0.27$ N/mm^2.

4.9.1.2. Collapse mechanism analysis

To benchmark the SAP 2000 analysis results, collapse mechanism also used. To get the collapsed mechanism analysis, BS 8110- Part 1:1977; [25] was used. It was considered that, the slab is restrained slab.

As per Clause 3.5.3.4[25];

From Equation 41

$m_{sx} \quad = \quad \beta_{sx} n l_x^2$

From Equation 42,

$m_{sy} \quad = \quad \beta_{sy} n l_x^2$

From table 3.14[25];

Consider positive moment at mid span,

$\beta_{sx} \quad = \quad 0.03$

$\beta_{sy} \quad = \quad 0.028$

Therefore from Equation 29,

$n \quad = \quad 1.4(0.52+0.15+0.15) + 1.6x1.5$

$\quad = \quad 3.548 \text{ kN/m}^2$

By substitution,

$m_{sx} \quad = \quad 0.03 \times 3.548 \times 3^2$

$\quad = \quad 0.957 \text{ kNm/m.}$

m_{sy} = $0.028 \times 3.548 \times 3^2$

= 0.894 kNm/m.

From Equation 43,

M/I = σ/y.

σ = My/I.

y = 0.1215/2

= 0.06 m.

I = $bd^3/12$

= $1 \times 0.1215^3/12$

= 1.5×10^{-4} m^4.

Therefore from substitutions,

σ_{sx} = $0.957 \times 10^3 \times 0.06/(1.5 \times 10^{-4})$.

= 0.382 N/mm^2.

σ_{sy} = $0.894 \times 10^3 \times 0.06/(1.5 \times 10^{-4})$.

= 0.357 N/mm^2.

4.9.2. SAP 2000 finite element analysis of two story house

4.9.2.1. Surface stresses for Bed room number 2, floor slab – in main direction

The below Figure 50 shows the surface stresses in main direction of span for the bottom surface of the slab element for the load Combination 1.

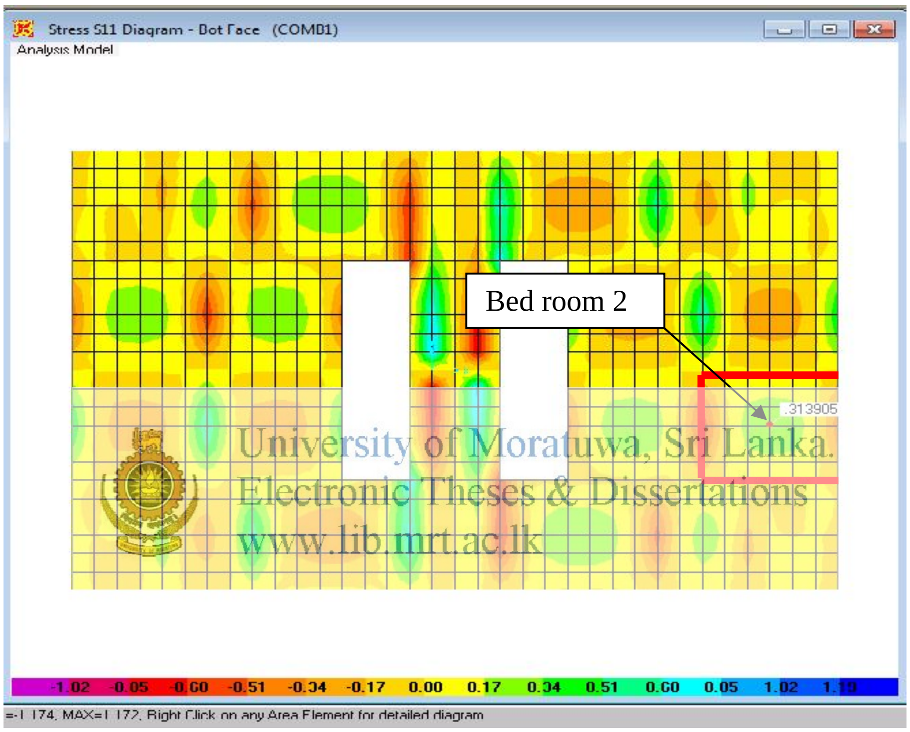


Figure 50- Surface stresses of first floor slab in main direction.

The Slab bottom surface stress in Bedroom 2 on grid direction shows 0.31 N/mm^2.

Since Elastic analysis delivered surface stress of 0.27 N/mm^2 and Collapse mechanism delivered surface stress as 0.382 N/mm^2. It shows that, finite element elastic analysis is fairly correct. The surface stresses depend on grid spacing. For this analysis grid spacing of 0.5 m was used. Therefore if further refine analysis is made, by introducing further small grid spacing, a fair result could be achieved.

4.9.2.2. Surface stresses for Bed room number 2, floor slab – in secondary direction

The below Figure 51 shows the surface stresses in secondary direction of span for the bottom surface of the slab element for the load Combination 1.

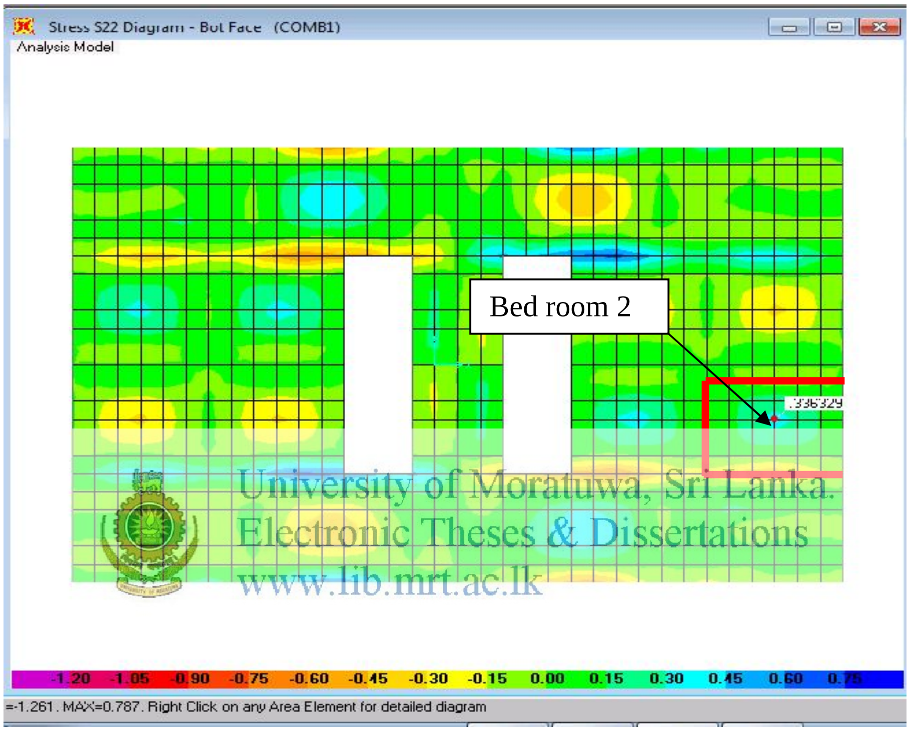


Figure 51-Surface stresses of first floor slab in secondary direction.

The Slab bottom surface stresses in Bedroom 2 on grid direction shows 0.33 N/mm^2.

Since Elastic analysis delivered surface stress of 0.27 N/mm^2 and Collapse mechanism delivered surface stress as 0.357 N/mm^2. It shows that, finite element elastic analysis is fairly correct. The surface stresses always interconnected with grid spacing. For this analysis grid spacing of 0.5 m was used. Therefore if further refine grid spacing is used, a fair result could be achieved. The finite element analysis is correct.

4.9.3. Conclusion on SAP 2000 analysis and Manual analysis

Since the manual elastic analysis and computerised elastic analysis may use different techniques and science, the output may not be same, but fairly a reasonable solution can be achieved. The technique adopted on each analysis is a form of reputed science. But in all cases the results may not coincide. In addition, the manual analysis on Elastic theory and Failure criteria gives the minimum results and maximum results respectively according to manual calculation techniques. It does not mean that the SAP 2000 results are out of the boundaries. By experiencing the upper and lower results from manual calculations the degree of accuracy and acceptability can be derived. Somehow for this instance, the principal direction stress at mid span is smaller comparative to the secondary direction mid span bottom layer stress. But for the manual calculation it delivers transverse response for collapse mechanism. Eventhough the final results slightly moved up and down the results are commonly acceptable and the finite element model is acceptable as accurate.

4.10. Analysed results of two story cluster houses

4.10.1. Shell stresses for load combination 1

The below Figure 52 shows the shell stresses for load combination 1

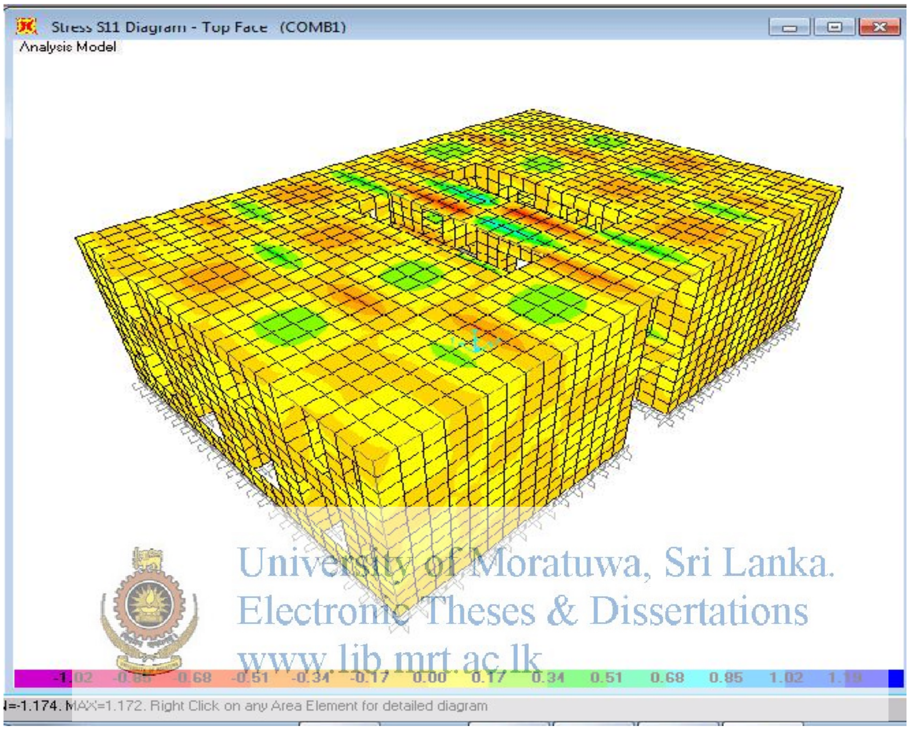


Figure 52-Shell stresses for load combination 1 of cluster houses.

The stresses in the model have range from -1.17 N/mm^2 to 1.17 N/mm^2 for load case one.

Since the compressive strength is limited to 1.6 N/mm^2 and the tensile strength is 2.79 N/mm^2, this house model is safe for load combination 1.

4.10.2. Shear stresses for load combination 1

The below Figure 53 shows the shear stress for load combination 1.

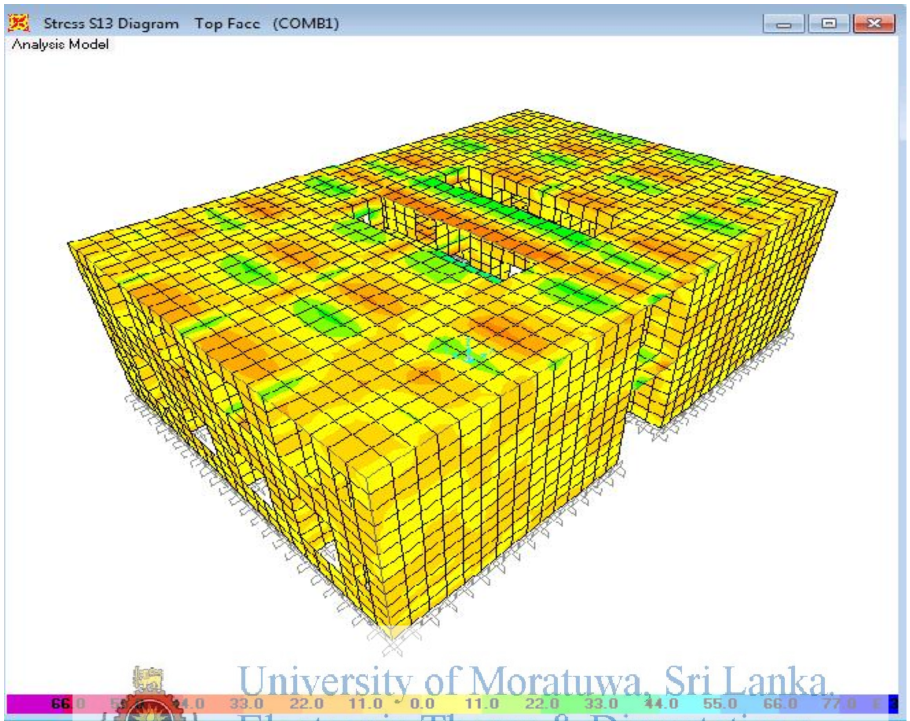


Figure 53-Shear stresses for load combination 1 of cluster houses.

The shear stresses in the model has ranged from -0.05 N/mm^2 to 0.05 N/mm^2.Since the shear stress is limited to 0.05 N/mm^2 This house model is safe for load combination 1. Even the SIP material property and the shear stress become same, the analysed shear stress is lesser as it deliver results conservatively based on grid spacing. Therefore structure still becomes safe.

4.10.3. Shell stresses for load combination 2

The below Figure 54 shows the shell stresses for load combination 2.

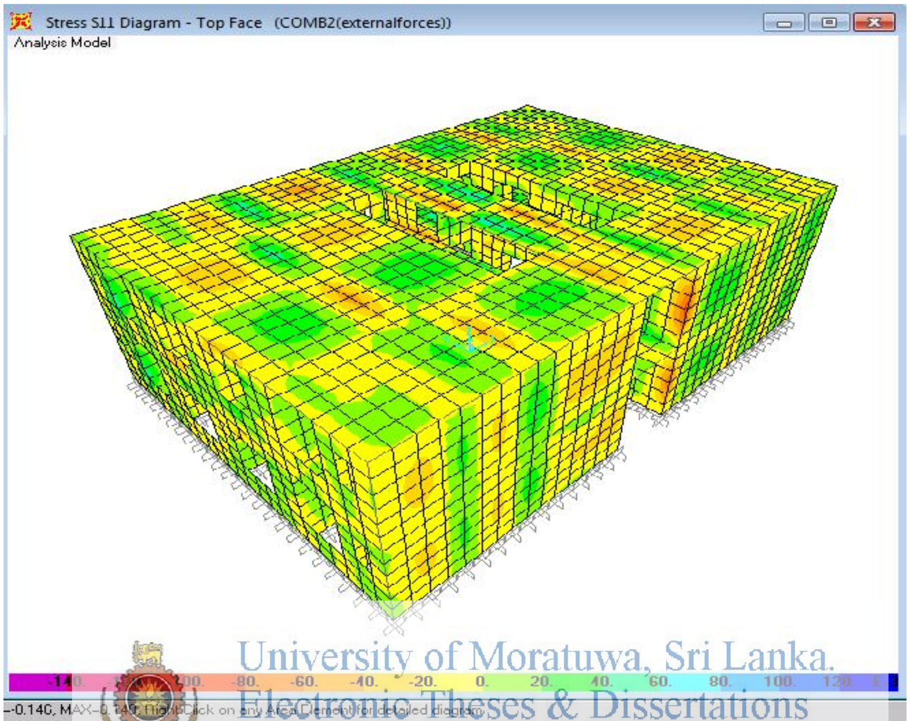


Figure 54-Shell stresses for load combination 2 on cluster houses.

The stresses in the model ranged from - 0.146 N/mm^2 to 0.140 N/mm^2 for load combination 2.

Since the elastic compressive stress is limited to -1.6 N/mm^2 and the tensile stress is limited to 2.79 N/mm^2, this house model is safe for load combination 2.

4.10.4. Shear stresses for load combination 2

The below Figure 55 shows the shear stresses for load combination 2.

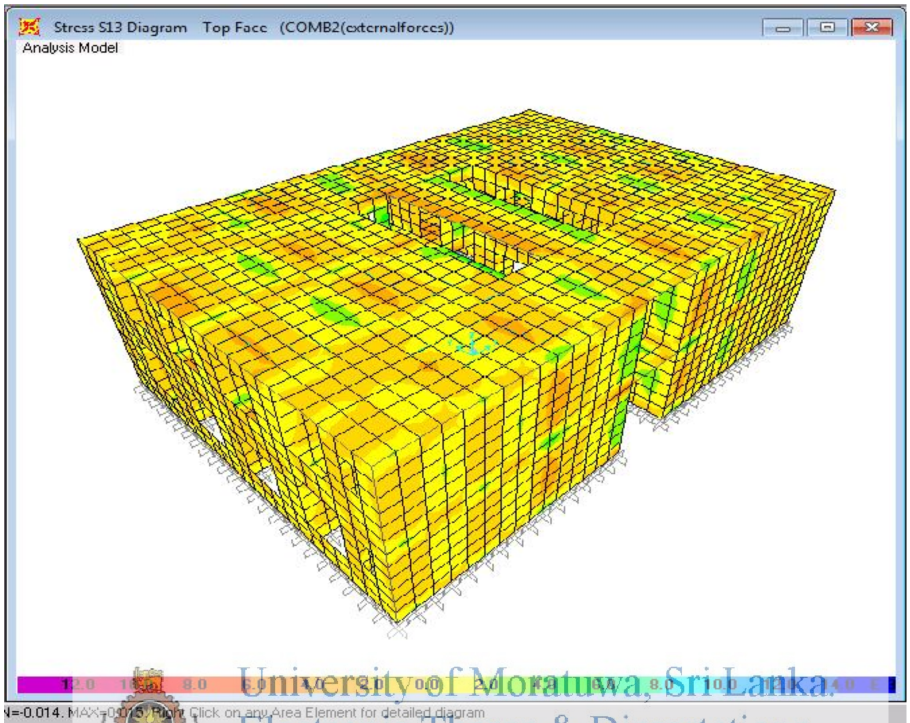


Figure 55-Shear stresses for load combination 2 of cluster houses.

The shear stresses in the model has ranged from -0.014 N/mm^2 to 0.015 N/mm^2.Since the shear stress is limited to 0.05 N/mm^2 This house model is safe for load combination 2 shear check.

4.10.5. Shell stresses for load combination 3

The below Figure 56 shows the shell stresses for load combination 3.

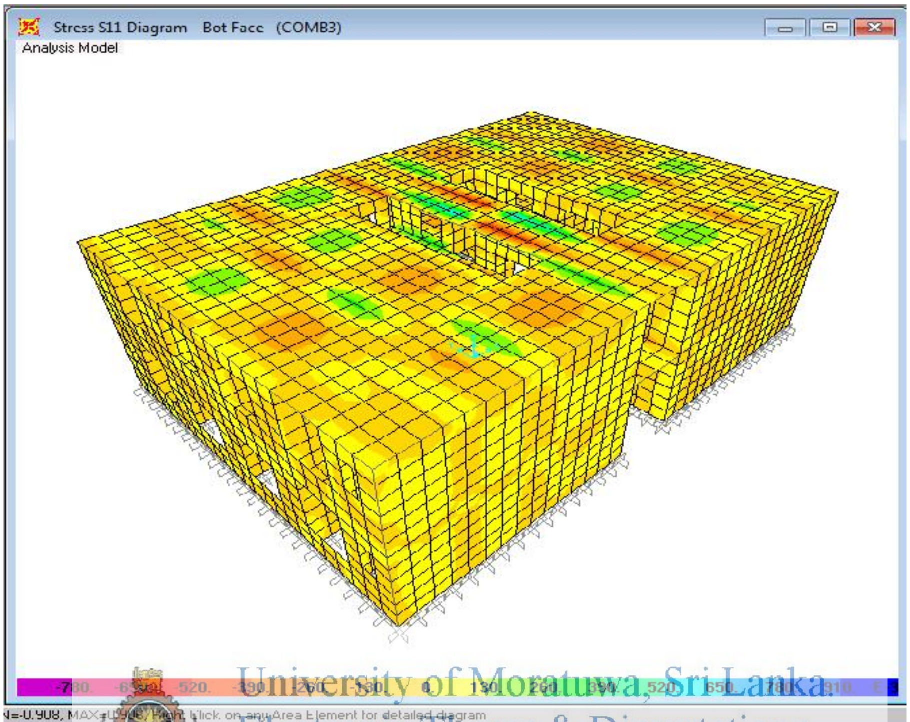


Figure 56- Shell stresses for load combination 3.

The surface stresses in the model limited from -0.908 N/mm^2 to 0.908 N/mm^2.

Since the elastic compressive stress limit is 1.6 N/mm^2. And the elastic tensile stress limit is 2.79 N/mm^2. This house model is safe for load combination 3 in surface stresses.

4.10.6. Shear stresses for load combination 3

The below Figure 57 shows the shear stresses for load combination 3.

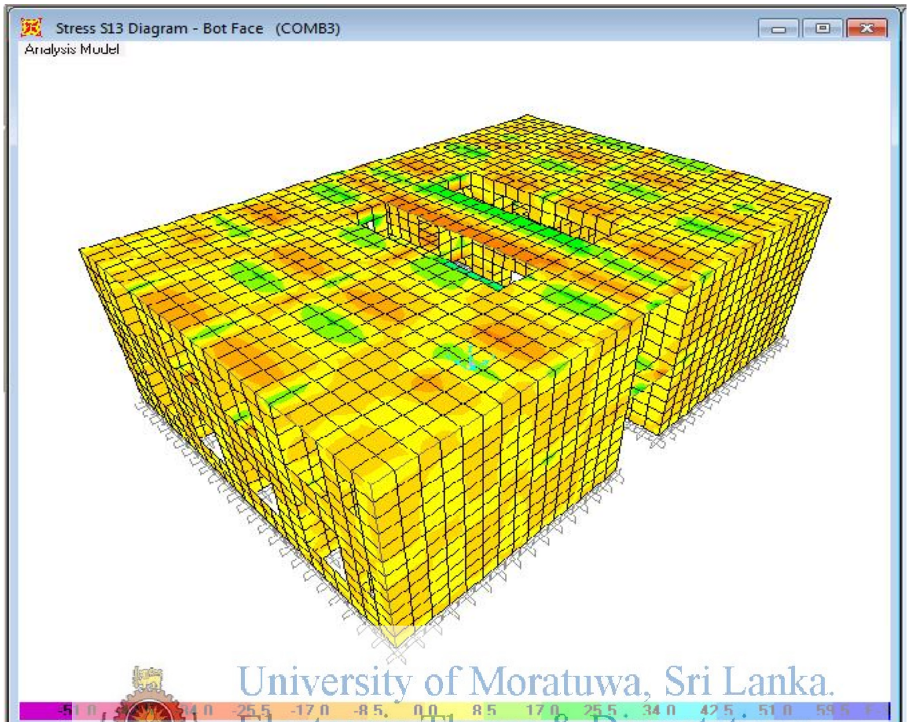


Figure 57-Shear stresses for load combination 3 on cluster houses.

The shear stresses in the model limited from - 0.49 N/mm^2 to 0.49 N/mm^2 for load combination 3. Since the shear strength is 0.05 N/mm^2. Even the SIP material property and the shear stress become same; the analysed shear stress is lesser as it delivers results conservatively based on grid spacing. Therefore structure is still safe.

4.10.7. Deflection Check

4.10.7.1.Overall deflection check

The below Figure 58 shows the deflection against later load on load Combination 3.

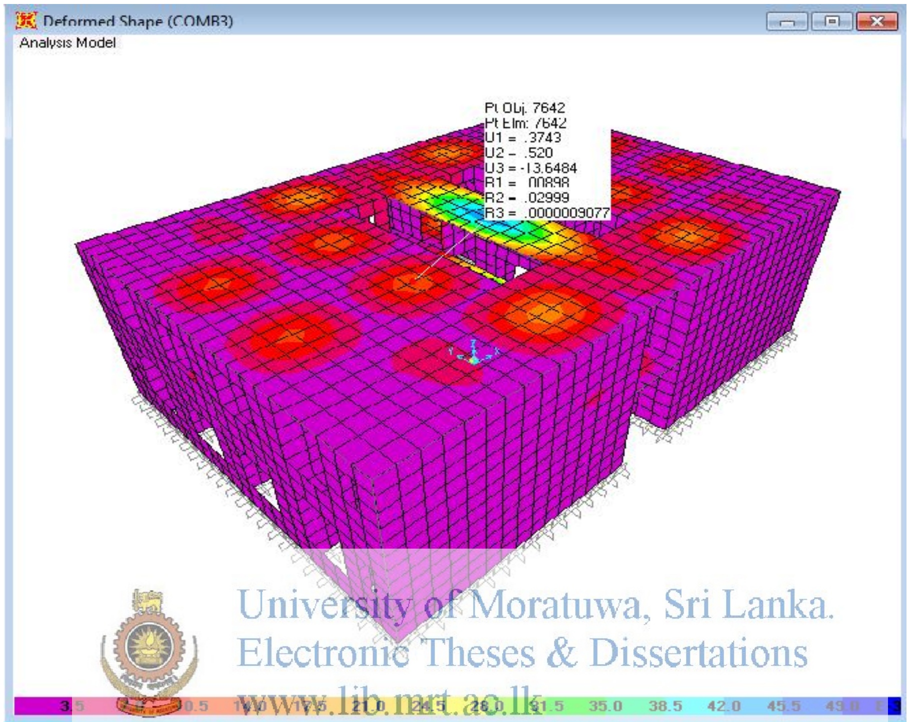


Figure 58-Overall deflection check against lateral load.

The cluster houses overall deflection check was made as above and found that the maximum deflection as 14 mm.

The deflection threshold as per clause 2.10 = 6000/100 = 60 mm

Therefore the overall deflection check comply the “European Recommendations for Sandwich Panel Part 1: Design”[14].

4.10.7.2. Deflection due to permanent loading

The below Figure 59 shows the deflection due to permanent loading in load Combination 1.

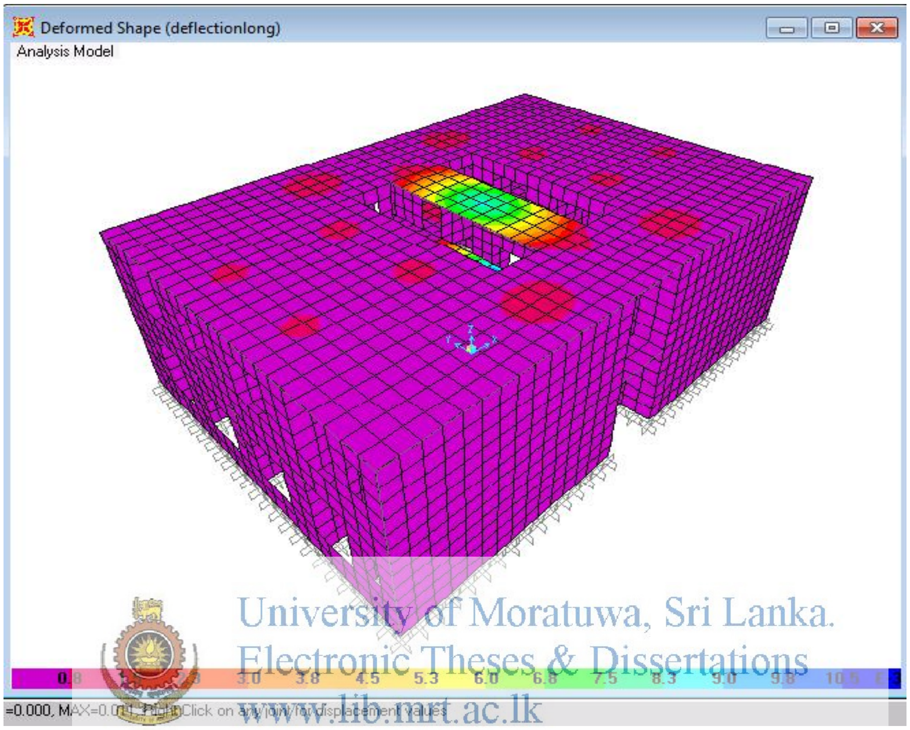


Figure 59- Deflection due to permanent loading

The deflection on slab elements has limited to 11 mm.

The deflection threshold as per clause 2.10 = 3500/100 = 35 mm.

Therefore the overall deflection check complied the “European Recommendations for Sandwich Panel Part 1: Design”[14].

4.10.7.3. Deflection due to short term loading

The below Figure 60 shows the deflection due to short term loading on load Combination 2.

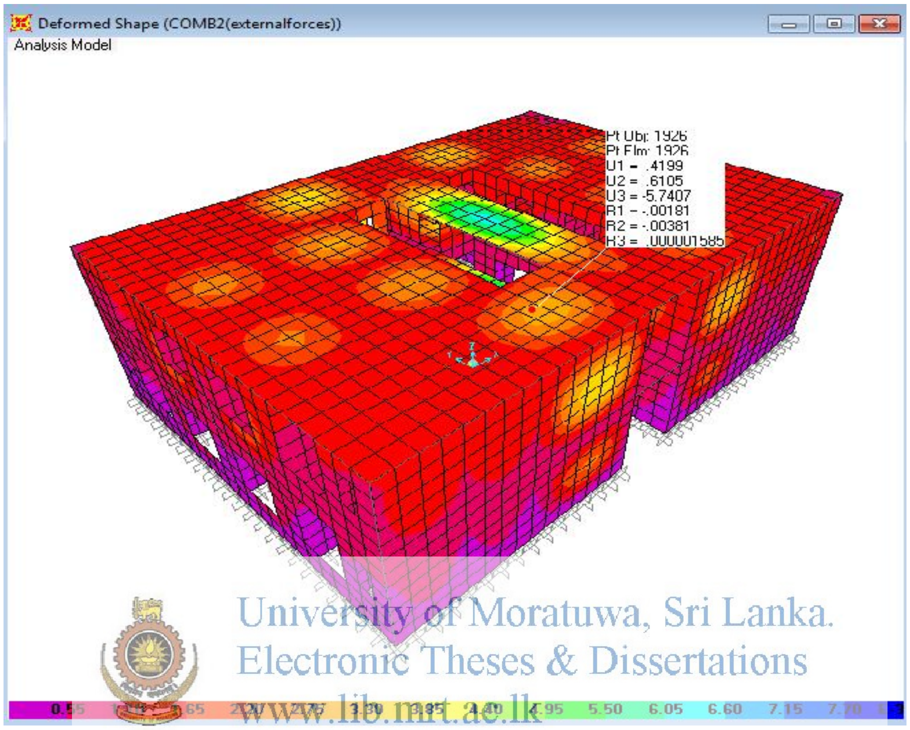


Figure 60-Deflection due to short term loading.

The deflection on slab and wall elements have limited to 6 mm.

The deflection threshold as per clause 2.10 = 3500/200 = 17.5 mm.

Therefore the overall deflection check complied the "European Recommendations for Sandwich Panel Part 1: Design"[14].

4.10.7.4.Conclusions on deflection check

As per the “European Recommendations for Sandwich Panel Part 1: Design”[14] for deflection check; it needs to check the deflection in three categories. Those are the deflection on short term loading, deflection under long term loading and the deflection due to lateral loading.

On the analysis, the outcomes were, the maximum deflection due to short term loading was 6 mm, the maximum deflection due to long term loading was 11 mm and the maximum deflection due to lateral loading was limited to 14 mm.

The limits on deflections for the model as [14] are; the maximum deflection due to short term loading is 17.5 mm, the maximum deflection due to long term loading is 35 mm and the maximum deflection due to lateral loading is limited to 60 mm.

Therefore the modelled structure is safe against deflection; it means that there is no requirement of additional introduction of surface stiffness. This design complied the “European Recommendations for Sandwich Panel Part 1: Design” [14] for deflection check.

4.11. Conclusions on two story cluster dwelling unit model analysis results

On all above three analysis cases the shell stresses were limited from -0.908 N/mm^2 to 0.908 N/mm^2 and the shear stress on the model was limited to 0.05 N/mm^2.

The row material's engineering properties were tested and established as the Shear strength is 0.05 N/mm^2, the compressive strength is 1.6 N/mm^2 and the tensile strength is 2.79 N/mm^2.

Therefore the analysis results were shown that the model is structurally safe.

The deflection was also checked against the "European Recommendations for Sandwich Panel Part 1: Design" [14]. It was found that the short term deflection occurred due to lateral loading in the model was 6 mm and the short term deflection limit is 17.5 mm. the deflection due to permanent loading in the model was 11mm and the deflection limit due to permanent loading is 35 mm. the lateral deflection on the model was 14 mm and the lateral deformation limit is 60 mm.

Somehow for the human comfort the lateral deformation limit is 12mm and the model had the deformation of 6 mm.

Therefore the model is safe against surface stresses, shear forces and deflections.

4.12. Conclusions on house analysis.

Two story house model

Based on the analysed results for the load combination 1, load combination 2 and the load combination 3; the results commonly that the maximum stress on element are limited from -0.69 N/mm^2 to 0.508 N/mm^2. This is a very low figure comparative to the material properties.

Since this material stiffness is very weak, it showed larger displacements on panels. The long term load case deflection showed 35 mm, short term load case deflection showed 26 mm and lateral load case deflection showed 39 mm displacements. Since the threshold are, as the deflection against short term loading limits 30 mm, 40 mm for long term loading deflection and 60mm for lateral loading case deflection.

The analysed model maximum shear stress was 0.049 N/mm^2. The material property showed that the shear strength is 0.05 N/mm^2. Therefore shear stress check on each model essential.

It is cleared that the thick shell element modelled on the two story house is safe against the typical loading parameters. The all element in the model behaves as two dimensional load sharing elements. Therefore on the practice of common house planning, if all structural elements are modelled as two directional load sharing elements and the structural elements can have the maximum bay size of 6 m x 6 m with possible openings to comply the architectural legislative requirements established by the Urban Development Authority of Sri Lanka [19] and the guidelines published by the Society of Structural Engineers, Sri Lanka as “Guidelines for Buildings at Risk from Natural Disasters” [17], the house model become safe.

Two story cluster house model.

The model has the common maximum structural elements of size from 3 m to 3.5 m and all element stresses were limited to -0.908 N/mm^2 to 0.908 N/mm^2. On the corridor, there is a slab element of the size of 6 m x 2 m. This element behaves as one way load sharing element. And the anticipated load on element is higher. Therefore this element was modelled with the reinforcing mean of 2 no of steel channel of 150 mm x 75 mm x 10 mm on free span along the longer direction. Even though this element was externally reinforced, it showed the largest deflection on permanent loading case.

The stresses on corridor elements rose from -0.908 N/mm^2 to 0.908 N/mm^2. The largest deflection of 11 mm was observed on this element under permanent load case. Therefore this element shall be considered as an extraordinary case and shall be prevented from planning.

The shear stress on load combination 1 and load combination 3 become critical as the material shear strength also equal to 0.05 N/mm^2. Therefore it is recommended to do the shear check first on each model.

The other elements on the cluster house model behave as the same as two story house model. Therefore by preventing the extra ordinary one way load sharing elements, it is possible to do the designs simply.

Some how the critical stresses were below the parameters established experimentally. Those are 0.05 N/mm^2 in shear strength, 1.6 N/mm^2 in compressive strength and 2.79 N/mm^2 in tensile strength, all model elements are structurally safe. The deflection checked also displayed that the model structure is safe.

The stiffness generated by structural opening reinforcing element and the stair case have not been taken in to account.

Therefore it is easily possible to express that, if one would do the design complying two ways load sharing element up to the panel size of 6 m x 6 m and comply the architectural legislative requirements established by the Urban Development Authority of Sri Lanka [19] and the guidelines published by the Society of Structural Engineers, Sri Lanka; "Guidelines for Buildings at Risk from Natural Disasters" [17]; the all designs would become safe.

In addition before material comes to its yield points, the service limits exceeds and therefore before the failure of the structure, it gives warning as excessive deflections on the building. Therefore SIP can be used for domestic constructions for Sri Lanka.

CHAPTER-5

COST ANALYSIS

5.1. General Introduction

The goal of this chapter is to find the value of construction of domestic structures from polyurethane sandwich panel material for walls, slabs and roofs. Two models were considered for the structural validity of the products. The same two models comprising of two story house and two story cluster housing structures will be economically analysed to see the possibility of economic viability of the material.

5.2. Conceptual design

All house units are assumed to be constructed with perimeter rubble wall of 1 m height, 0.5 m wide and 0.3 m elevation from the existing ground level. The proposed elevation is gained from selected earth filling and place the lean concrete for the entire floor to 75mm thick.

Walls and slab elements be constructed without openings. The architectural requirement of openings will be generated later. The all doors and windows are assumed to be from plywood panel doors and aluminium windows respectively.

The exposed roof panel joints shall be covered from plastic silicone, which shall generally be provided with the panel materials. The silicate base water proofing material will be applied with proper joint stress enhancing net to accommodate the possible thermal stresses. As per “Guidelines for Buildings at Risk from Natural

Disasters" [17] the steel elements were introduced for the strength enhancements. The cost on those materials was also considered on weight basis.

5.3. Basic cost elements for analysis

Following cost elements has incorporation for the construction cost derivations

Wall, Slab and roof panel is from 120mm thick polyurethane

Sandwich panels having the site cost as	-	1725 Rs/m^2
Panel erection cost	-	172.50 Rs/m^2
Rubble works on foundations	-	7000 Rs/m^3
Imported earth filling	-	1700 Rs/m^3
Floor carpeting	-	1000 Rs/m^2
Screed concrete 75mm thick	-	1000 Rs/m^2
Water proofing (Silicate base)	-	800 Rs/m^2
Aluminium Window	-	6750 Rs/m^2
Plywood doors	-	9000 Rs/m^2
Bathroom floor tiling	-	3500 Rs/m^2
Steel member supply & fixing with surface treatments	-	280 Rs/kg
Plumbing works for housing unit	-	Rs.50, 000
Electrical works for housing unit	-	Rs.50, 000

5.4. Costing for two story housing unit

Wall area	=	387.25 m^2
Slab area	=	56.25 m^2
Roof cover area	=	118.75 m^2
Total sandwich panel	=	553.25 m^2

Sandwich panels having the site cost as - 553.25 m^2 x 1725 Rs/m^2 = Rs. 968,190

Panel erection cost	-553.25 m^2 x 172.50 Rs/m^2	= Rs. 96,819
Rubble works on foundations	- 17m^3 x 7000 Rs/m^3	= Rs. 119,000
Imported earth filling	- 21.4m^3 x 1700 Rs/m^3	= Rs. 36,400
Floor carpeting	- 127.5 m^2x1000 Rs/m^2	= Rs. 127,500
Screed concrete 75mm thick	-71.25 m2 x 1000 Rs/m^2	= Rs. 71,250
Water proofing (Silicate base)	- 127.75 m^2 x 800 Rs/m^2	= Rs. 102,200
Aluminium Window	-32m^2 x 6750 Rs/m^2	= Rs. 216,000
Plywood doors	-18 m^2 x 9000 Rs/m^2	= Rs.162, 000
Bathroom floor tiling	-9m^2 x 3500 Rs/m^2	= Rs. 31,500
Steel angle 150 x 150 x10 mm	-137.5m x 6,594 Rs/m	= Rs.906, 675
Plumbing works for housing unit		= Rs. 50,000
Electrical works for housing unit		= Rs. 50,000

Total cost for 127.5 m^2 house = Rs. 2,937,534 therefore per floor rate =Rs23, 039/m^2

5.5. Costing for two story cluster housing units

Wall area	=	988 m^2
Slab area	=	186 m^2
Roof cover area	=	186 m^2
Total sandwich panel	=	1360 m^2

Sandwich panels having the site cost as	-1360 m^2 x 1725 Rs/m^2	= Rs.2, 380,000
Panel erection cost	-1360 m^2 x 172.50 Rs/m^2	= Rs. 238,000
Rubble works on foundations	- 29m^3 x 7000 Rs/m^3	= Rs. 203,000
Imported earth filling	- 61.m^3 x 1700 Rs/m^3	= Rs. 103,700
Floor carpeting	- 372 m^2 x 1000 Rs/m^2	= Rs. 372,000
Screed concrete 75mm thick	-204 m2 x 1000 Rs/m^2	= Rs.204, 000
Water proofing (Silicate base)	- 210 m^2 x 800 Rs/m^2	= Rs.168, 000
Aluminium Window	-58m^2 x 6750 Rs/m^2	= Rs.391, 500
Plywood doors	-98 m^2 x 9000 Rs/m^2	= Rs.885, 600
Bathroom floor tiling	-24m^2 x3500 Rs/m^2	= Rs. 84,000
Steel angle	- 116 m x6594 Rs/m	=Rs.764, 904
Steel channel	-24m x 6594n Rs/m	=Rs.158, 256
Plumbing works for housing unit	- 8 no x Rs. 50,000/each	= Rs. 400,000
Electrical works for housing unit	- 8 no x Rs. 50,000/each	= Rs. 400,000

Total cost for 8nos x 46.5 m^2 house	=	Rs. 6,752,960
Therefore per floor rate	=	Rs.18, 153/m^2

5.6. Costing conclusions

The two model deliver the average rate for the per floor as Rs.20,600/m^2 The traditional house constructed from brick works, concrete elements and roof cover all inclusively accommodate Rs. 34,970/ m^2. Therefore the domestic units constructed over polyurethane sandwich gives more than 41% saving.

CHAPTER-6

OUTLINE OF THE PROJECT

6.1. General Introduction

The goal of this chapter is to validate the project finding in terms of engineering, economics, time and quality management, saving of natural resources and reusing of resources. The new dimension on polyurethane sandwich material is a good initiation. The validations on each category will be discussed separately.

The two numbers of typical floor plans have been analysed. Those are very typical for individual house constructions and housing cluster constructions. The ultimate goal is to validate the project for realizations.

6.2. Engineering validation

The two plans have been modelled on SAP 2000 software to find the critical surface stresses, shear stresses and deflections under different load combinations under the standards published by engineering bodies. "European Recommendation for Sandwich Panel; part1, Design [14] was used for deflection checking. The additional robustness to the structures were introduced as per the published guidelines by the Society of Structural Engineers, Sri Lanka; "Guidelines for Building at Risk from Natural Disasters [17].

Since the polyurethane sandwich panel does have the low density rather than nominal construction materials, the structural criticality may arise under lateral loading conditions. Therefore, this scenario was separately analysed. The output of the analysed result shows that the critical finite elements reached the maximum of 0.508 N/mm^2 from -0.69 N/mm^2. For the individual house, which has the span up to 6 m by 6 m. The shell element panels are analysed as two way span thick panels. As per the material test results on this thesis Chapter 3 on SIP, the analysed results are below the elastic capacities. The elastic limit for SIP material is 1.6 N/mm^2 in compression, 0.05 N/mm^2 in shear and 2.79 N/mm^2 in tension. Hence the shear stresses becoming critical as the material strength is a low value.

The anticipated deflections on two story house were 35 mm for long term loading, 26 mm for short term loading and 39 mm for lateral deflection check. The thresholds were 40 mm for long term loading, 30mm for short term loading and 60 mm for lateral loading. Therefore it is clear that it is easily possible to meet the engineering parameters relevant to two story domestic constructions.

The model of cluster houses had the corridor with free span of 6 m with the width of 2 m. It was analysed as one way span element and the results show high stresses. Somehow typically when this type of element is designed, it may comprise of beams on either side of the long spans. Therefore typically reinforcing channel sections were introduced. This element is the most critical element and all the other elements of the model comply with the stresses and the deflections as above model as all elements are the two way load sharing elements of maximum 3.5 m by 3 m. The aforesaid critical corridor element delivered the critical surface stresses as from -0.908 N/mm^2 to 0.908 N/mm^2. Since the material properties identified on this thesis have the elastic shear stress of 0.05 N/mm^2, compressive stresses of 1.6 N/mm^2 and tensile stress of 2.79 N/mm^2.

On concluding, it is clear that for all two ways load sharing elements up to 6 m, the identified SIP are safe for domestic loading constructions. In addition if it would introduce a local capacity enhancement mechanism in the form of introduction of shear stud or any other form, even after the constructions completed, the local strengthening could be done easily for the future expansion works.

6.3. Economic validations

In Sri Lanka typically almost all domestic constructions are made from traditional materials. The typical construction cost also float on the typical range. Therefore the cost comparisons generally made on net area basis. The costing on traditional house under nominal finishing condition may deliver the average cost as Rs. 34,970/m^2. Comparatively the designed two house models ended up with the nominal finishing conditions as Rs.20, 500/m^2. This is 41% saving from nominal traditional material. This is a remarkable achievement.

6.4 Time Validation

To construct a sandwich structure, it may need nearly one tenth to one twentieth of the time needed comparative to the house constructed from traditional materials. Since there is a remarkable achievement it is fitted to Sri Lanka's needs. Some people may construct their houses on years. The common route cause is the shortage of skill labour in the country for construction industry. The people spent almost 30% to 60% of the construction cost as labour. But with the experiences for large construction from SIP, it needs only 3.5% of the material cost for material erections. Since the panel manufacturers generally deliver the assembly procedures and tools' needed, in time to come, dwelling unit owners might be able to construct their own houses.

6.5. Quality validations

The material comes with polythene insulations to protect the material from possible physical damages on logistics, erections and building finishing stages. Since all SIP's are automated factory products each element may hold its unique consistency. Therefore comparatively owners getting good output as no labour applied paints etc. Depending on the environmental conditions the customer can order the products to match his needs. The durability of the face material also can be differed from supplier to supplier. The surface material coatings are differing from supplier to supplier as some SIPs' are having aluminium coated, zinc coated or enamel paint coated only or a series of combinations.

6.6. Polyurethane sandwich material validation for domestic constructions in Sri Lanka

It is proven that the introduction of sandwich material would become realized and would become more popular as it validates in more parameters against traditional materials as on engineering, economical, time and quality. Since this option save the scares raw material on Mother Nature a regulatory mediation and more research on this stream is a must.

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