Advanced FRP for Flooring in Buildings

-A Low carbon material application in the construction industry

By

Yijian Gao

This thesis is submitted in partial fulfilment of the requirements for the award of the degree of Doctor in Civil Engineering, from the University of Portsmouth

June-2013
Abstract

Fibre-reinforced polymers (FRP) are building materials that permit both the improvement of long-term building performance and the simplification of the construction process, thanks to their high specific strength, low thermal conductivity, good environmental resistance, and ability to be formed into complex shapes. FRP materials are well-suited to fulfilling many building functions. By integrating traditionally separate building systems and layers into single function-integrated components, and by industrially fabricating those components, the amount of on-site labour can be greatly reduced and overall quality can be improved. The FRP materials used in the construction industry include glass fibre reinforced polymer (GFRP) and carbon fibre reinforced polymer (CFRP). Most GFRP based buildings are lacking in integration and function and only benefit for the small span, with deep beams or slabs. The CFRP based construction component has higher strength and stiffness. However, the investigation into CFRP based buildings has been lacking.

This research aims to investigate the CFRP floor panel, as a primary component in the floor system, to replace traditional concrete floor slab in large buildings. The objectives of this project include the design of CFRP floor panel system in buildings using European design codes, analysing proposed CFRP floor panel by FEA modelling, and experimentally validating design and FEA models using scaled CFRP floor samples. A scale effect of test specimen was investigated in conducting design strength check of full proposed CFRP floor panel. This project supplied design curves with dimensional parameters for practical design of CFRP floor panels, to fit the design specifications required by different buildings with varied dimensions. Design curves present the measurements of deflection and critical stresses against the variation of the proposed CFRP floor panel with different dimensions.

The proposed CFRP floor panel was designed as a pultruded beam with an open cross-section. The design was carried out using the Eurocodes and supported by the finite element analysis (FEA). Modelling results indicated that the proposed floor panel passed the tall design check, recommended by Eurocode, and the safety checks on both deflection and material strength, which are important for producing CFRP floor panel products that meet the dimensional requirements in design of buildings with different design specifications.
Experimental results of scaled CFRP floor panel samples are also presented in this thesis, which successfully validates the design and modelling analysis. The conducted scale effect was amended by a reduction factor of 0.625 for the material strength of the full CFRP panel, which passed the Hashin criteria check. This project also studied the shear effects on bending behaviour of proposed CFRP panel with open cross-section consisting of thin-walled plates. An important load-deflection correction factor was proposed, which plays an important role together with geometrical shape factor in the calculation of shear related deflection.

This novel CFRP floor panel can be easily installed in buildings because of its lightweight feature, and easily integrated with the suspending ceiling, ventilation and lighting system because of its designed shape. This investigation also provided plenty of information concerning the use of this potential building component in the low carbon construction industry, which could save up to 50% heating energy and reduce CO_{2} emissions by 40% compared to the traditional construction industry.

Key words: Composite floor panel system, pultruded CFRP slab, Low carbon construction material, Finite elements analysis.
Contents list

Abstract...........................................................................................................................................i
Contents list.......................................................................................................................................iii
Declaration .........................................................................................................................................ix
List of figures ......................................................................................................................................x
List of tables .....................................................................................................................................xiii
Acknowledgements ..........................................................................................................................xiv
Dissemination ....................................................................................................................................xv

Chapter 1- INTRODUCTION .................................................................................................................1
1.1 Background ..................................................................................................................................2
1.2 Motivations ..................................................................................................................................3
1.3 Objectives ...................................................................................................................................5
1.4 Methodology ...............................................................................................................................6
  1.4.1 Conceptual design of CFRP floor panel ...............................................................................6
  1.4.2 Structural analysis and design safety check .........................................................................6
  1.4.3 Experimental validation .......................................................................................................7
1.5 Thesis organization .......................................................................................................................7
1.6 Terminology ................................................................................................................................9

Chapter 2 - APPLICATION OF FRP IN BUILDING CONSTRUCTION ........................................13
2.1 Introduction ................................................................................................................................14
2.2 Chronology ...............................................................................................................................14
2.3 Modern FRP building and systems ..........................................................................................20
  2.3.1 GE Living Environment Concept House – USA, 1989 .......................................................20
    2.3.1.1 Structural System .........................................................................................................21
    2.3.1.2 Production and assembly .........................................................................................21
    2.3.1.3 Integration of functions ..........................................................................................21
    2.3.1.4 Discussion ..................................................................................................................22
  2.3.2 Advanced Composite Construction System- UK, 1986-96 .................................................22
    2.3.2.1 Noteworthy projects .................................................................................................23
    2.3.2.2 Structural system .......................................................................................................23
    2.3.2.3 Integration of functions ............................................................................................24
    2.3.2.4 Discussion ..................................................................................................................24
## Contents list

2.3.3 Eyecatcher Building- Switzerland, 1999 ................................................................. 25  
   2.3.3.1 Structural system ............................................................................................... 25  
   2.3.3.2 Integration of functions ..................................................................................... 26  
   2.3.3.3 Discussion.......................................................................................................... 26  
2.3.4 Startlink Composite House- UK, 2012 .................................................................... 27  
   2.3.4.1 Structural system ............................................................................................... 27  
   2.3.4.2 Discussion.......................................................................................................... 28  
2.4 Conclusion ...................................................................................................................... 29

### Chapter 3 - DESIGN GUIDELINES ............................................................................. 31

3.1 Introduction .................................................................................................................... 32  
3.2 Structural design guideline .......................................................................................... 34  
3.3 Material selection ........................................................................................................... 40  
   3.3.1 Glass fibres ............................................................................................................. 40  
   3.3.2 Carbon fibres ......................................................................................................... 41  
   3.3.3 Kevlar TM (Aramid fibres) .................................................................................... 41  
3.4 Proposed Manufacture ................................................................................................. 42  
   3.4.1 Pultrusion................................................................................................................ 42  
   3.4.2 Hot press moulding ................................................................................................. 43  
3.5 Connection design ........................................................................................................... 44  
   3.5.1 Bolted connections ................................................................................................. 45  
   3.5.2 Clamped connections ............................................................................................. 45  
   3.5.3 Bonded connections ............................................................................................... 45  
3.6 Fire design ...................................................................................................................... 46  
   3.6.1 Design requirements ............................................................................................... 46  
   3.6.2 Performance criteria ............................................................................................... 46  
3.7 Conclusion ...................................................................................................................... 48

### Chapter 4- PROPOSED FRP FLOOR PANEL SYSTEM ............................................. 49

4.1 Introduction .................................................................................................................... 50  
4.2 Conceptual design of proposed CFRP floor panel ......................................................... 51  
4.3 Design load of proposed floor panel ............................................................................ 53  
4.4 Numerical modeling analysis ....................................................................................... 54  
   4.4.1 Establishment of modeling ..................................................................................... 54  
   4.4.2 Geometrical simplifications .................................................................................. 55
4.4.3 Boundary considerations .................................................................55
  4.4.3.1 Mid-span ..................................................................................55
  4.4.3.2 End supports ...........................................................................55
4.4.4 Material properties .........................................................................55
4.4.5 Meshing ..........................................................................................56
4.4.6 Results and design check .................................................................57
  4.4.6.1 Deflection check .......................................................................57
  4.4.6.2 Hashin criteria .........................................................................58
  4.4.6.3 Effects of varied mesh densities in the model of the floor panel on critical
          stresses ..........................................................................................59
  4.4.6.4 Effects of varied mesh densities on Hashin criteria indicators ...61
  4.4.6.5 Hashin criteria check ...............................................................65
  4.4.6.6 Stress contour ..........................................................................66
  4.4.6.7 Stress in fibre direction .............................................................67
  4.4.6.8 Stress in transverse matrix direction ........................................67
  4.4.6.9 Stress in through-thickness direction .........................................68
  4.4.6.10 Shear stress in XZ plane ..........................................................69
4.4.7 Discussion of critical design parameters ..........................................69
4.4.8 Dimensional variation of floor panels .............................................70
  4.4.8.1 Span variation ..........................................................................70
  4.4.8.2 Height variation .......................................................................71
  4.4.8.3 Width variation ........................................................................72
  4.4.8.4 Bonding length variation ..........................................................73
4.4.9 Design curves ................................................................................74
4.5 Buckling and free vibration analysis ..................................................75
  4.5.1 Buckling analysis .........................................................................75
  4.5.2 Free vibration analysis .................................................................76
4.6 Discussion ........................................................................................78

Chapter 5 – EXPERIMENTAL AND MODLING WORK OF SCLED SAMPLES ..........79
5.1 Introduction .......................................................................................80
5.2 Test sample and mold description .....................................................80
  5.2.1 Materials .......................................................................................81
  5.2.2 Geometrical shape and dimensions ..............................................81
  5.2.3 Production of CFRP elements ......................................................82
  5.2.4 Adhesives .....................................................................................84
<table>
<thead>
<tr>
<th>Section</th>
<th>Page</th>
</tr>
</thead>
<tbody>
<tr>
<td>5.3 Facilities</td>
<td>85</td>
</tr>
<tr>
<td>5.3.1 Oven</td>
<td>85</td>
</tr>
<tr>
<td>5.3.2 Test machines</td>
<td>86</td>
</tr>
<tr>
<td>5.4 Experimental set-up</td>
<td>86</td>
</tr>
<tr>
<td>5.5 Description of specimens</td>
<td>86</td>
</tr>
<tr>
<td>5.6 Test procedure</td>
<td>87</td>
</tr>
<tr>
<td>5.7 Test results and discussion</td>
<td>87</td>
</tr>
<tr>
<td>5.8 Modeling of scaled floor panels</td>
<td>91</td>
</tr>
<tr>
<td>5.8.1 Effects of varied mesh densities on structural behaviour</td>
<td>92</td>
</tr>
<tr>
<td>5.8.2 Effects of varied mesh densities in span and height on Hashin criteria indicators</td>
<td>93</td>
</tr>
<tr>
<td>5.8.3 Stress contour</td>
<td>98</td>
</tr>
<tr>
<td>5.8.3.1 Stress contour in X-direction</td>
<td>98</td>
</tr>
<tr>
<td>5.8.3.2 Stress contour in transverse direction</td>
<td>98</td>
</tr>
<tr>
<td>5.8.3.3 Shear stress contour in YZ plane</td>
<td>99</td>
</tr>
<tr>
<td>5.8.4 Debonding of scaled test samples</td>
<td>100</td>
</tr>
<tr>
<td>5.9 Scale effects</td>
<td>101</td>
</tr>
<tr>
<td>5.9.1 Weakest-link theory</td>
<td>102</td>
</tr>
<tr>
<td>5.9.2 Calculation of scaling effects</td>
<td>104</td>
</tr>
<tr>
<td>5.10 Conclusion</td>
<td>104</td>
</tr>
<tr>
<td>Chapter 6 - SHEAR EFFECTS OF CFRP FLOOR PANELS</td>
<td>106</td>
</tr>
<tr>
<td>6.1 Introduction</td>
<td>107</td>
</tr>
<tr>
<td>6.2 Analytical calculation of deflection</td>
<td>107</td>
</tr>
<tr>
<td>6.2.1 Deflection of a floor beam with an open cross-section due to bending</td>
<td>107</td>
</tr>
<tr>
<td>6.2.2 Shear effects on the deflection of a CFRP floor panel</td>
<td>113</td>
</tr>
<tr>
<td>6.2.3 Analysis of scaled test panels</td>
<td>114</td>
</tr>
<tr>
<td>6.2.3.1 shear distribution</td>
<td>114</td>
</tr>
<tr>
<td>6.2.3.2 Form factor</td>
<td>117</td>
</tr>
<tr>
<td>6.2.3.3 Conduction of load-deflection correction factor</td>
<td>119</td>
</tr>
<tr>
<td>6.2.4 Theoretical calculation of deflections of full CFRP panels</td>
<td>121</td>
</tr>
<tr>
<td>6.3 Conclusion</td>
<td>122</td>
</tr>
<tr>
<td>Chapter 7 - THERMAL BEHAVIOR OF FRP COMPONENTS AND FIRE SAFETY ENGINEERING DESIGN</td>
<td>124</td>
</tr>
<tr>
<td>7.1 Introduction</td>
<td>125</td>
</tr>
</tbody>
</table>
8.5.3 Lighting system ................................................................. 153

Chapter 9 - CONCLUSION AND FUTURE WORK ...................... 154
9.1 Overview .............................................................................. 155
9.2 Proposed CFRP floor panel system ....................................... 155
9.3 Experimental investigation ..................................................... 156
9.4 Design curves ....................................................................... 157
9.5 Future work ......................................................................... 158

REFERENCE ........................................................................... 160

APPENDIX ........................................................................... 168
Declaration

Whilst registered as a candidate for the above degree, I have not been registered for any other research award. The results and conclusions embodied in this thesis are the work of the named candidate and have not been submitted for any other academic award.

Yijian Gao
LIST OF FIGURES

Figure 1-1 The structure of thesis ................................................................. 8
Figure 1-2 The element-based local coordinate systems ................................. 12
Figure 2-1 Monsanto house construction ....................................................... 15
Figure 2-2 Untitled project ........................................................................... 15
Figure 2-3 Schalenhaus doernach project ....................................................... 16
Figure 2-4 Pilyvilla project ........................................................................... 16
Figure 2-5 Telephone exchange room project ............................................... 17
Figure 2-6 All plastic house in winter ............................................................. 17
Figure 2-7 Fiber-shell project ....................................................................... 18
Figure 2-8 Futuro project ............................................................................. 19
Figure 2-9 Kunststoffhaus 2000 system ......................................................... 19
Figure 2-10 Maison en Plastique Project ....................................................... 20
Figure 2-11 GE Living Environment House .................................................. 20
Figure 2-12 Moulded baseboard raceway (left), Utilities embedded in waffle slab (middle), Integrated shelf connection system in foundation wall form work (right) .......... 21
Figure 2-13 The main ACCS components (left), Assembled corner section (right) ............. 22
Figure 2-14 Box-beam assembly (left), Aberfeldy footbridge (middle and right) ..... 23
Figure 2-15 Severn Visitor’s Center (left), Automated car wash enclosure (right) .... 23
Figure 2-16 Building applications of the ACCS proposed by Minguzzi ............. 24
Figure 2-17 The Eyecatcher: Under construction (left), Completed (middle), Translucency (right) ............................................................................................... 25
Figure 2-18 The Eyecatcher: Frame schematic (left), Built-up members (middle), Frame (right) ........................................................................................................... 26
Figure 2-19 Startlink Composite house .......................................................... 28
Figure 2-20 Startlink components ................................................................. 28
Figure 3-1 Vertical deflection ........................................................................ 35
Figure 3-2 Stress-strain properties of typical fibres ........................................ 40
Figure 3-3 Schematic showing the pultrusion manufacturing process ............... 43
Figure 3-4 Hot press moulding ..................................................................... 44
Figure 4-1 A CFRP panel and the cross-section view ....................................... 51
Figure 4-2 Bonded connection ..................................................................... 52
Figure 4-3 A CFRP floor panel systems with raised floors and suspending ceilings .... 53
Figure 4-4 LUSAS element geometry 3D Solid Continuum Element –HX8M (left), PN6 (right) .................................................................54
Figure 4-5 A FEA model of half floor panel ...............................................................57
Figure 4-6 Predicted deflection in half model ..............................................................57
Figure 4-7 A FEA model of the CFRP panel .................................................................66
Figure 4-8 Stress $\sigma_x$ distribution in fibre direction ..............................................67
Figure 4-9 Stress $\sigma_y$ distribution in transverse direction ......................................68
Figure 4-10 Stress $\sigma_z$ distribution through-thickness direction ..............................68
Figure 4-11 Shear stress $\sigma_{xz}$ distribution in XZ plane ..........................................69
Figure 4-12 Analysis of panels with varied span .........................................................71
Figure 4-13 Analysis of panels with varied height .......................................................71
Figure 4-14 Analysis of panels with varied width .......................................................72
Figure 4-15 Analysis of panels with varied bonding length .......................................73
Figure 4-16 First buckling mode predicted by FEA ....................................................76
Figure 4-17 First free vibration mode predicted by FEA ............................................77
Figure 5-1 Carbon prepreg .......................................................................................81
Figure 5-2 A cross-section view of scaled sample .......................................................81
Figure 5-3 Die for producing top plate of specimen ....................................................82
Figure 5-4 Release films A5000 (left) for top plate and the one A2000 (right) for C-section strip ..........................................................83
Figure 5-5 Cross-section view of section mould .......................................................84
Figure 5-6 Moulds and carbon prepreg .................................................................84
Figure 5-7 Bytec Hydraulic Pressure Oven ..............................................................85
Figure 5-8 Zwick/Roell Z030 test machine ...............................................................86
Figure 5-9 Experimental set-up ...............................................................................86
Figure 5-10 A scaled floor panel .............................................................................87
Figure 5-11 Load-deflection curves .........................................................................88
Figure 5-12 Histogram for 8 samples .........................................................................88
Figure 5-13 Normal distribution .............................................................................90
Figure 5-14 Cumulative distribution .......................................................................90
Figure 5-15 A half FEA model with a line load .........................................................91
Figure 5-16 A deformed mesh ...............................................................................92
Figure 5-17 A final FEA shell model .......................................................................95
Figure 5-18 Stress distribution in fibre direction .......................................................98
Figure 5-19 Stress distribution in transverse direction .............................................99
Figure 5-20 Shear stress distribution in YZ plane ....................................................99
List of Figures

Figure 5-21 Failure modes (a) Web broken (b) Debonding (c) Top plate broken ..........101
Figure 5-22 Logarithmic plot of a strength size effect .............................................103
Figure 6-1 A beam with simple supports and uniformly distributed load ..................107
Figure 6-2 Deflection and curvature of a beam due to bending .................................108
Figure 6-3 Deflection of a beam of asymmetrical cross section .................................108
Figure 6-4 Bending of a beam with an asymmetrical cross-section ............................109
Figure 6-5 Bending stress distribution .......................................................................114
Figure 6-6 Dimension of specimen’s cross-section .................................................115
Figure 6-7 Shear stress distribution .........................................................................117
Figure 7-1 Energy loss in brick block .......................................................................129
Figure 7-2 A FEA model of brick block ..................................................................130
Figure 7-3 Temperature distribution at wall cross section after 2 hour ......................131
Figure 7-4 Inside temperature against time of brick block .......................................131
Figure 7-5 The reduction of temperature at inner wall after 2 hours .........................132
Figure 7-6 Construction material thermal behaviors in 5 hours’ time .........................133
Figure 7-7 Energy loss by heat transfer though different panels ..............................135
Figure 7-8 Methods for achieving fire safety objectives .........................................139
Figure 7-9 Mechanisms involved in the thermal decomposition of polymer composites, showing feedback loops involving heat flux .........................................................140
Figure 7-10 Effects of temperature and time of a gas fire on the post-fire tensile strength of a 16-ply (1.9 mm thick) carbon/epoxy laminate ..............................................141
Figure 7-11 Approximate variation in tensile strength with temperature for concrete and reinforcing steel .................................................................142
Figure 7-12 Standard fire curve (ISO 834) for insulation materials ........................142
Figure 7-13 Fire protection for CFRP panel system .................................................143
Figure 7-14 PHI contour of Rockwool insulation with 80mm thickness under 1000°C for after 120mm burning analysis ..........................................................144
Figure 8-1 Position locaters on a steel I beam .........................................................148
Figure 8-2 CFRP slabs installation ...........................................................................148
Figure 8-3 Raised floor installation ..........................................................................149
Figure 8-4 Suspend ceiling installation .....................................................................149
Figure 8-5 Final assembled CFRP floor system ......................................................150
Figure 8-6 Illustration of HVAC system ................................................................152
Figure 8-7 Typical office lighting systems ...............................................................153
List of tables

Table 3.1 Summary of Standards Committees for FRP composites ........................................32
Table 4.1 Material Properties ..................................................................................................56
Table 4.2 Effects of varied mesh densities in span on the maximum stresses .......................59
Table 4.3 Effects of varied mesh densities in height on the maximum stresses ......................60
Table 4.4 Effects of varied mesh densities in width on the maximum stresses ......................60
Table 4.5 Hashin criteria check with varied mesh densities in span ......................................61
Table 4.6 Hashin criteria check with varied mesh densities in height ....................................63
Table 4.7 Hashin criteria check with varied mesh densities in width ....................................64
Table 4.8 Design check for deflection and strength ................................................................66
Table 4.9 Eigenvalues (load factor) of first eight buckling modes .........................................75
Table 4.10 Recommended values of parameters in Equation 4.23 and acceleration ratio limit ......................................................................................................................................77
Table 4.11 Frequencies predicted by FEA modeling ............................................................77
Table 5.1 Mechanics properties of Araldite 2015 ..................................................................85
Table 5.2 Specimens’ test results ..........................................................................................88
Table 5.3 Statistical results. ..................................................................................................89
Table 5.4 Effects of varied mesh densities in span on the maximum stresses .......................93
Table 5.5 Effects of varied mesh densities in height on the maximum stresses .....................93
Table 5.6 Hashin criteria check with varied mesh densities in span ......................................94
Table 5.7 Hashin criteria check with varied mesh densities in height ....................................94
Table 5.8 Design check for deflection and strength ............................................................97
Table 5.9 Hashin criteria check using reduced strength by the scaled effect .......................104
Table 6.1 Form factor and load-deflection correction factors conducted from a scaled panel ...........................................................................................................................................120
Table 6.2 Deflections of a full CFRP panel given by theory and FEA modeling ....................122
Table 7.1 Thermal properties of construction materials ......................................................127
Table 7.2 CO₂ emission in producing the different building materials ................................128
Table 7.3 Energy loss by absorption from different walls ....................................................134
Table 7.4 Total Energy loss by different walls per day .........................................................135
Table 7.5 Temperature against Rockwool thickness ............................................................144
Acknowledgements

I would like to sincerely thank my study director, Dr. Jiye Chen. It was he who brought me in the door and guided me throughout my research. His suggestions were always constructive and beneficial, whether regarding my research or my life. His patience and generosity made my life much easier than I thought it could be. I thank him for all of the once-in-a-lifetime opportunities that he gave to me, and I know that I could not have made a better choice, one that will forever guide my future.

I would like to express my great thanks to Dr. Zhongyi Zhang, my second supervisor. His knowledge in FRP materials helped me to complete my experimental work. I would also like to thank Dr. Dominic Fox, who gave me lots of help in administrative support. My great thanks also should be given to Dr. Hom Dhakal, who provided me with lots of help for the manufacturing test samples. Thanks to the Gurit Company, for supplying all of the CFRP materials in this investigation.

I am grateful for my family and for their constant mental support, no matter how far away they are. I especially thank my wife. She made great sacrifices in her career to support my completion of my PhD degree study. Without her love and support, it would be impossible for me to complete this PhD project.
Dissemination


Gao Y (2011), Network Group for Composites in Construction Young’s Research Poster Competition, Advanced Composites in Construction, University of Warwick
INTRODUCTION
1.1 Background

There are four main types of materials used in the construction industry: stone, timber, concrete, and steel. In the past, stone and timber were the key materials used for building. As a result of development and the industrial revolution, steel entered into the equation, followed later on by concrete, the use of which made improving the standard of living as far as property is concerned a natural progression. These two materials gradually dominated the construction industry and have made a great contribution to the lives of those who live or work in towns and cities.

For over a century, steel and concrete have been used in the constructing of almost every building, however, these traditional materials are heavy, quick to corrode or break down so do not have a great lifespan, while from an ecological point of view, their production produces a lot of carbon dioxide (CO$_2$), e.g. 90.2 kg CO$_2$ per ton of concrete, 2200kg CO$_2$ per ton of steel (Buenett 2006). This release of carbon dioxide is now proven to have a great effect on the environment, and society has become increasingly aware of the potential risks this poses. To solve this problem, we must develop a construction industry which, put simply, has a low carbon footprint. Efficient design and the manufacture of new construction materials and components, which are structurally effective and environmentally green, is now becoming essential in the reduction of carbon emissions where the construction industry is concerned. Fibre reinforced polymer (FRP) is corrosion resistant, lightweight, high in strength, has good thermal conductive and anti-fire performance and not only has a long lifespan, but its production can greatly reduce CO$_2$ emissions both through its method of manufacture, plus its effective thermal insulation qualities. As a consequence, FRP building elements could prove to be one of the newer types of material being used in the construction industry in the near future.

For the past 50 years, FRP materials have been mainly used by the automotive and aerospace industries, both of whom were aiming to utilise its versatility as a material. Just as its name implies, FRP consists of fibres which are embedded in a polymer matrix to provide added strength. These fibres bind together to enhance the performance of FRP.

As manufacturing techniques have developed over the last two decades, such as pultrusion, filament winding and injection molding, which made the cost of FRP profiles has fallen
dramatically (Burgoyne, 2004 and Lux Research 2012). As a result, FRP has become more and more widely used in the construction industry, particularly in relation to commercial buildings. Compared to more traditional materials, FRP products are beneficial and advantageous to use owing to:

- High strength to weight ratio
- Durability in all environments
- Ease of use and time saving in construction
- Electromagnetic neutrality - ideal for certain specific requirements
- Adaptability and versatility of component manufacture
- High durability, low thermal conductivity and lightweight.

However, the use of FRP does have certain shortcomings, the most obvious being cost, which is still double that of steel or concrete based on today’s cost of materials. Though many of today’s manufacturing processes provide for the future ability to recycle products, FRP does not fall in to this category – however its lifespan offsets this drawback to what is felt is an acceptable degree.

1.2 Motivations

In order to reduce the effects of CO\textsubscript{2} on global warming, the CO\textsubscript{2} emission rate is expected to be reduced by 26% by 2020 when compared to emission rates measured in 1990, and reduced by 60% by 2050 (HM Government, 2008). Therefore, all new residential properties are required to be classed as ‘zero carbon’ from the year 2016 on, and non-domestic buildings by 2019 according to the new policy issued by the UK government, though at present there is no clear definition of what ‘zero carbon’ actually means. Carbon fibre reinforced polymer (CFRP) has been used in many situations to build up primary structures because of its high strength, light weight and durability. CFRP also has another outstanding feature, which is that of low thermal conductivity (Mutnuri, 2006). As a consequence, CFRP and its use in the construction industry can have a positive effect on CO\textsubscript{2} emissions. In the last decade, apart from the application of CFRP in strengthening damaged structures, a number of bridges incorporating CFRP have been built worldwide, residential properties have been being built using CFRP in Europe and piers have recently been built in the USA using CFRP (Bank, 2006). We are now at the stage where the CFRP based floor panel, as a new building
component, has the potential for mass use as its production and use all assist in reaching the required CO$_2$ emissions targets.

For speeding up the development of a reduced carbon emission construction industry, structural CFRP components such as pultruded CFRP beams and composite sandwich decks are now being utilised in bridge engineering (Bakis, et al., 2002). In building construction, the low thermal conductivity of CFRP enables load-carrying components to act as insulating in addition to their structural function. A complete glass fibre reinforced polymer (GFRP) house was manufactured by Start link in the UK, in 2011 (Hutchinson, Singleton, 2007). Compared with traditional houses, GFRP houses could be economic to both build and maintain, thermally efficient and have long-term sustainability. Existing GFRP houses were designed with a short life span, as single or two storey residential properties. However, GFRP is restricted in its use in construction owing to the materials lack of fire resistance owing to the polymer resin used. Therefore, application of CFRP materials was increased greatly in large constructions, e.g., the AIT bridge. The span of AIT Bridge can be up to 25m (Nakamura et al. 2011). This proved the CFRP is feasible to construct large buildings because of its high strength, rigidity (Bank, 2006) and similar thermal conductivity compared to GFRP (Mutnuri, 2006). The potential applications of CFRP components could include schools, offices, hospitals, etc. As a consequence, an advanced CFRP floor panel system for large buildings was investigated in this PhD study. A design approach and technique for producing a novel CFRP floor panel which would replace traditional heavy and thick concrete was proposed. The economic benefits of building with CFRP components can be obtained both through energy saving and reduction in costs where installation and transportation etc. are concerned. There is also a significant environmental benefit through CO$_2$ emission reduction when considering the reduction in heating needs within buildings which use CFRP components. In conclusion, the CFRP floor panel should be given serious consideration as a new construction component as it could bring long term benefits to a construction industry required to reduce its carbon footprint.

The benefits from the application of a CFRP floor system include the following:

1) The weight of a CFRP floor panel is only about 8–10% that of reinforced concrete. Therefore a CFRP floor panel replacing concrete reduces the dead load significantly. (Gao, Chen, etc., 2013)
2) Swift and ease of installation can save considerably on labour costs, making any project more economic. Pultruded CFRP panels offer advantages over cast-in-situ concrete slabs as the quality of CFRP products can be accurately monitored in the factory (Bank, 2006).

3) High strength and rigidity of carbon fibre gives CFRP panels a high safety margin.

4) Significant energy saving can be presented by the following points. Large buildings with CFRP floor panels have reduced heating demands in the winter, so reducing corresponding CO$_2$ emissions (Hutchinson, Singleton, 2007). Ready Homes Enterprises (n.d) summarized that a CFRP domes house could save 50 -70% on heating energy when compared to a traditionally built house, and could reduce the emission of CO$_2$ by at least 50%. Compared with conventional materials, new buildings with a CFRP floor system will require fewer raw materials and consume less energy in their overall construction.

5) Building construction is a complicated and often wasteful process as waste and off-cuts from materials used on site are seldom recycled (Hutchinson, Singleton, 2007). However, pre-pultruded CFRP floor components can certainly cut this waste down. A previous investigator, Keoleian G. A. (2005) indicated that assuming the life span of FRP bridge decks over a 60 year period, the CFRP deck could be 37% less costly to build in the long run consume less total primary energy in overall construction by 40% and reduce the emission of carbon dioxide by 39% (Keoleia, et al., 2005) from a materials point of view. Taking in to account the total benefits from all aspects, as in heating costs (Wilkinson, n.d) and corresponding CO$_2$ emission reduction, foundations, swift installation, transportation costs, and reduced maintenance costs (Keller, Haas and Vallee, 2008), etc., the initial problem of higher material costs (estimated double cost of traditional materials (Bakis, et al.,2002, Keller, Haas and Vallee, 2008, Keller, 2007)) of the CFPR components, when compared to conventional costs, can be justified.

1.3 Objectives
The objective of the research program is to investigate a CFRP floor panel system in buildings, and it consists of the following sub objects:

- Conceptual design of a CFRP floor system
- Determination of the geometry of the CFRP panel
• FEA analysis for supporting initial design
• Design safety check using Eurocode
• Investigation of mechanical behaviour and influencing parameters
• Design of CFRP floor panels with varied dimensions
• Validation using scaled test samples
• Design of connections between composite floor slabs and steel or concrete beams
• Fire safety design of CFRP floor panel system
• Raised floor, suspended ceiling and lighting systems, and surface treatment
• Installation and assembling approach

1.4 Methodology
The methodology used in this project was that of comparative case studies for the conceptual design, Eurocode based design check, numerical modeling verification and experimental validation. Firstly, the conceptual design was based on the previous application of FRP materials in buildings. Secondly, the design was carried out using Eurocodes and was supported by the finite element analysis (FEA). The commercial software package, LUSAS, was chosen in the investigation. Finally, initial design was validated by experimental test work of scaled CFRP panel samples.

1.4.1 Conceptual design of CFRP floor panel
In order to make the most efficient use of energy and resources, strong attention was paid to resolving the problems that hinder the use of FRP materials in buildings. The first step was to identify these problems through a review of the history of building materials. Secondly, the current methods used to resolve these problems were studied. Wherever possible, the most appropriate and effective existing solutions were selected and presented as references for creating a new design procedure of the proposed CFRP floor panel system, which includes general design of the geometry of the CFRP floor panel and techniques for both production and connection.

1.4.2 Structural analysis and design safety check
A number of numerical models were employed to investigate deflection and stress distribution of the CFRP panel, with different geometrical parameters. Modelling results were used to carry out design safety checks, including deflection checks and strength checks, and
to optimise the initial design. According to the failure criterion recommended by Eurocode, the design would reflect the concerns over and effects of different design parameters on the predicted behaviour of the proposed CFRP floor panel. The thermal performance of the proposed CFRP floor panel was also investigated by FEA modelling work, to approve the application of CFRP as an insulation material that could bring benefits through reducing heating requirements.

1.4.3 Experimental validation
The design work on the proposed CFRP floor panel was validated by scaled test samples. Validation included three-point bending tests and corresponding FEA modelling simulation. Through validation, structural rigidity, failure mechanism, scale effects and crucial stresses of the proposed floor panel were identified using design failure criteria. Also, a manufacturing technique for the proposed new CFRP floor panel was suggested.

1.5 Thesis organization
The research work presented in this thesis is divided into four main sections: review of building materials, design of the CFRP floor panel, FEA modelling analysis and experimental validation. The structure of this thesis can be seen in Figure 1-1. A summary of each chapter is given as follows:

Chapter 1: An introduction to the project is given, followed by a statement on the background of this research subject and a listing of the objectives and research methods, which have been presented above.

Chapter 2: Review of the general design principles of FRP structures, the shapes of different cross-sections, connections, etc. The conclusion provides justification for investigating the CFRP floor panel when compared to steel or concrete beams. A review of FRP building in the construction industry - definition, characteristic and design is stated in this chapter.

Chapter 3: A general design guideline of FRP composite structures is given in this chapter. A discussion of design criteria used in this investigation is also presented in this chapter.
Chapter 4: The conceptual design of the CFRP floor panel system is formed. A basic model of a proposed floor panel is designed using Eurocodes and supported by the FEA modeling analysis. The detailed design curves and modelling simulation are presented in this chapter.

Chapter 5: An experimental investigation of scaled CFRP floor panels is presented. A comparison of experimental results with predictions from FEA simulation is made for validating the proposed CFRP floor panels. The scale effect with material strength reduction is investigated to verify the proposed full panel in satisfying the design criteria.

Figure 1-1: The structure of thesis
Chapter 6: The shear effect of proposed open cross-section of CFRP floor panel on the deflection is investigated in this chapter. The formulations to calculate deflection with shear effects are conducted using experimental results, and applied in the prediction of deflection of full floor panels. Validation by FEA modelling is also given in this chapter.

Chapter 7: Based on Eurocode and previous investigations (EC2, 1992), the fire safety engineering design for the CFRP floor system is established and presented. Furthermore, the FEA modeling for thermal behaviours of the CFRP floor system is also presented in this chapter to investigate the energy saving effects in buildings with CFRP components.

Chapter 8: In terms of the outcomes from the initial design, an approach for installation and assembling of a CFRP floor panel system is suggested in this chapter.

Chapter 9: The conclusions for this research project are summarised, and future research work, in the application of CFRP materials in the construction industry, is suggested.

1.6 Terminology
Unfortunately, many scientific and engineering terms are also used in common language, with different or less specific meanings. Even within the scientific community, there are sometimes several different conventions followed due to lack of consensus. Therefore, in the pursuit of clarity, some of these somewhat ambiguous and often misunderstood terms, related to the topics in this thesis, are defined below.

Matrix / Resin:
The matrix is the binding material that envelopes the reinforcement in a composite material (see definition below). Resins are one of the many ingredients in matrix materials (along with fillers, curing agents, mould release agents, stabilizers, etc.) and are composed of polymers.

Polymer / Plastic:
A polymer is a broader category that contains a wide range of materials, all of which have a long chain structure consisting of millions of repeated molecules. Carbohydrates, rubbers, and DNA are all polymers. The precise definition of the term plastic, however, is less clear. Some chemistry dictionaries indicate that plastics are only the thermoplastic sub-category of polymers, while others indicate that they must be organic. The only consensus seems to be
that plastics can only be synthetic polymers, i.e. not naturally occurring. In general, the confusion in the scientific community is magnified tenfold in the general population, thus the term plastic has been avoided in this text wherever possible.

**FRP / PMC**

These abbreviations refer to the same material: composites consisting of fibre reinforcements and polymer matrices. FRP stands for fibre-reinforced polymer (also fibre-reinforced plastic – see the previous heading for the explanation of why this term is avoided), and PMC stands for polymer-matrix composite. No one abbreviation is more correct than the other. Usage appears to be a matter of tradition and preference. Additional variations of the terms may specify the exact reinforcement, such as GFRP or CFRP to specify glass or carbon fibre, or to indicate the production method, such as PFRP to indicate composites produced through pultrusion. Beyond the series of abbreviated terms, there is a vast array of strings that mean essentially the same thing: fibre-reinforced polymer, fibre-reinforced plastic, polymer-matrix composite, fibre-matrix composite, organic-matrix composite, and synthetic-resin composite, etc. The term FRP, for fibre-reinforced polymer, will be used in this thesis.

**Composite / Advanced Composite**

Composites include an enormous range of materials, though the word is often used to specify the subcategory of FRP composites. Composites are substances composed of multiple materials of which large units (on a molecular scale) remain chemically separate, but act in many ways as a single material. FRP materials are only one of many materials that fit this description. The term ‘composites’ can apply to carbon nano-tube reinforced epoxy resin, straw reinforced mud bricks, or simply wood. For this reason, the term ‘advanced composites’ is sometimes used to exclude other materials. In this thesis, ‘composite’ means fibre reinforced polymers.

**Thermal related terms**

We quite often see terms such as Thermal Conductivity / Thermal Conductance / Heat Transfer Coefficient / Thermal Resistance / Thermal Transmittance / R-Value / U-Value. These terms are confusing, because they all refer to the transference of heat through materials, and the definitions of these terms, given by different countries and industries, vary slightly. In general, thermal conductivity is a material property that describes the rate of heat flowing through a unit area over a unit of time, when subjected to a 1°C temperature gradient.
It is represented by the Greek symbol $\lambda$ (or k) and given in the units W/m·K. Thermal conductance is very similar to $\lambda$, but it is used for a finite area and finite thickness of a material. Thus, it is a system property rather than a material property (units W/K). The heat transfer coefficient refers to the radiation and convection conditions at the surface of a structure. It is represented by the letters $h_r$ or $h_c$, and is given in the units W/K.

The previous terms are primarily used in science and engineering, while the following terms are most often used in the building industry. They are linked to specific arrangements of materials, including all layers, air gaps, and surface conditions. Therefore, they are system properties. The thermal transmittance, or U-value, incorporates both the thermal conductance and heat transfer coefficients of a particular structure, such as a wall or a window frame. The terms ‘thermal resistance’ and ‘R-Value’ are synonymous and are both the reciprocal of thermal transmittance.

As stated, these terms are easily confused and their definitions vary by location and field. Thus, wherever possible, descriptions will be made in terms of the basic values of thermal conductivity, $\lambda$, and the heat transfer coefficient, $h$.

**Heat Flux / Heat Flow Rate / Total Heat / Heat Generation Rate**

These terms are confusing because they are all represented by the letter Q (or q) and refer to the transfer of energy as heat. Heat flux is the amount of energy being transmitted as heat per unit area. It is represented by the symbol $q$ and given in the units W/m$^2$. The heat flow rate is the rate of energy being transmitted as heat through a finite area. It is represented by the symbol $Q$ and given in the units W. The total heat is the total amount of energy transferred to a system, which is represented by the symbol $Q$ and given in the units J. Finally, the symbols $q_{sd}$, $q_{w}$, and $q_{gen}$ represent the total amount of heat generated or consumed per unit mass (units kJ/kg) throughout a reaction. The first two symbols respectively refer to the decomposition of polymers and the vaporization of water, while the third symbol is a general term for any heat generation or consumption reaction.

**Longitudinal / Transversal / Thickness / 1-1 / 2-2 / 3-3**

All of these terms refer to directions based on the element based local coordinate systems shown in Figure 1-2. Two systems are considered: the assembly/product and the material. On the assembly/product scale, the longest dimension is termed the longitudinal direction. The
second largest dimension is termed the transversal, and the shortest dimension is defined as thickness (see Figure 1-2). In the material system for FRP, the first direction, 1-1 (shown in Fig. 1-2), is the direction in which most reinforcement is oriented. The second direction, 2-2, is the direction in which some reinforcement is oriented. Finally, the third direction, 3-3, is the direction in which the least reinforcement is oriented. For unidirectional materials, there are no reinforcements in both 2-2 and 3-3 directions.

Figure 1-2: The element-based local coordinate system
LITERATURE REVIEW
2.1 Introduction
The earliest use of FRP materials in building construction dates back to the 1950s with the construction of some single storey properties. The application of composite materials has been largely developed within the past 20 years; not merely intermediate technology systems with hand lay-up fabrication processes which is labour intensive, but also automatic high-tech manufacturing means such as pultrusion, hot press and filament winding have been widely used (Hollaway, 1994). However, there are limitations for the use of FRP composites; for example, the pultrusion of fibre-reinforced polymers in building construction. When compared with the use of concrete and steel, it has restrictions and the use of FRP is only suitable in certain circumstances. Only in the following applications can FRP materials be employed: resistance to corrosive environments, electromagnetic transparency and low weight to strength ratio (Bruneau, 1994). Considering these restrictions of FRP composites, it is quite important for researchers to expand the range of use of these materials. Employing adhesive joint methods for load-bearing parts is a method rarely used in construction and no former design standards of this kind have existed.

From the 1950s to the 1970s, FRP materials began to be used in the construction of commercial buildings. Unfortunately, due to the oil crisis of 1973, construction projects using FRP materials had to stop as oil is the source of polymers and FRP materials became too costly. However, this problem took place in the early days of FRP application, but was solved in the 1980s when oil price started to fall and stabilise. At this point the principle of performance-base was accepted. The public’s comprehension of synthetic polymer materials improved due to the application of carbon-fibre materials in luxury and high-tech products. Finally, FRP materials began to be extensively used in construction again from 1989. Section 2.3 will give a brief description of some of the typical buildings in which FRP materials were used.

2.2 Chronology (From 1950s to 1970s)
This section reflects back on some key projects that used load-bearing FRP elements from the 1950s to the 1970s. These projects are listed by year, project title, architect/engineer and location.

1956 Monsanto House of the Future – Richard Hamilton, Marvin Goody – USA
Named the “Monsanto House of the Future”, this project was Monsanto’s attempt to create a lucrative engineering industry for synthetic polymers. No one had ever thought that the design of the project could become a model for mass production. The cost of the prototype was almost $1,000,000 (one million dollars), but Monsanto assumed that mass production would be much cheaper, pricing each unit at a more reasonable and affordable $20,000 (twenty thousand dollars) (Meikle, 1995). As seen in Figure 2-1, there was a concrete core in the centre, from where four FRP cantilevers, arranged in a C-shape, radiated outwards. Every cantilever included two winding shells or bents, which formed the bottom chord and the outside wall, then the roof. The cantilever at the bottom chord, which is supported by the base, generated pressing forces.

![Figure 2-1: Monsanto house construction (Meikle, 1995)]

1957 Untitled-Cesare Pea-Italy

The design of this project was for the 6th Milan Exhibition (Triennale at Milan). As illustrated in Figure 2-2, the materials of the 4.8m\(^2\) square, 2.7m tall boxes were glass-reinforced polyester and a honeycomb-shaped paper core. Different arrangements, as well as additional units, were considered for the design. The thermal integration, air-conditioning and ventilating system were exceptionally good. The surface areas of the walls and floors included graphite covered glass fibre cloth; electrical heating elements were included as well (Makowski, 1964).

![Figure 2-2: Untitled project (Makowski, 1964)]
1958 Schalenhaus Doernach – Rudolf Doernach – Germany
The first German FRP system was exhibited at the Stuttgart Plastics Show. The modular approach was also employed in this design; a hexagonal space which was formed by four panels. As illustrated in Figure 2-3, the early model included a foam core laminated to aluminium face sheets, but materials of the latter models included GFRP laminate face sheets with a covered paper honeycomb core (Makowski, 1964). A 50m² floor was created by the 9m span and, by connecting other hexagonal parts, larger structures were made. By inserting a set of tubes through the honeycomb core, which had a liquid inside, some models’ fire resistance was improved, together with improved thermal dissipation and sound insulation. A disturbing noise was caused by the differential solar heating system, when the pop-riveted joints moved. In addition, the steel pop-rivets corroded over time and the white external surfaces became spotted (Fire, 1991).

![Figure 2-3: Schalenhaus doernach project (Fire, 1991)](image)

1960 Polyvilla – J. Ladyjenski, S. A. Sodibat – Belgium
As shown in Fig. 2-4, the largest production of FRP buildings in the world appeared between the years of 1960 and 1973 when approximately 250 types of this building were constructed. PVC tubes filled with concrete were included in the skeleton of the structure. 2.0 m tall and 2.6 wide wall panels merged with glass-reinforced polyester face sheets and a phenolic foam core to make a 50 mm thick sandwich structure (Schein, 1971).

![Figure 2-4: Pilyvilla project (Schein, 1971)](image)
1963 Telephone Exchange room- Mickleover Ltd. – UK
A two-storey system was designed by Quarm and Mickleover for the Bakelite Corporation to improve the design of a Signal Relay Room System. A thicker-walled version of the system, illustrated in Fig. 2-5, was made in the same year. It was built as a biological research laboratory for the British Antarctic Survey (Makowski, 1964). By using phenolic foam and fire-proof polyester resins, according to British Standard 476, the relay room system and the two-storey system were both rated as Class 1 surface spread of flame.

![Figure 2-5: Telephone exchange room project (Makowski, 1964)](image)

1965 All Plastic House – Dieter Schmid – Italy
This design was made with FRP sandwich boards. Steel columns and a concrete automatic carport were built into the foundation (Doernach et al., 1974). As seen in Figure 2-6, the FRP house was constructed on top of the concrete, automatic carport.

![Figure 2-6: All plastic house, in winter (Doernach, et al., 1974)](image)

1968 Fibre-Shell – Ezra Ehrenkrantz, TRW Systems Corporation – USA
In this design, there were two different kinds of fully-developed systems: one of sections made in flat boards which connected to tubular forms, and one of tube filaments curved on an enormous, house-sized, mandrel, as shown in Fig. 2-7 right. The former approach was developed because it allowed for a more economical use of materials, less expensive tooling costs, more flexible dimensions and ease of transport. The latter approach created extremely
rigid structures which had no flexible dimensions and were more difficult to transport. As shown in Fig. 2-7 left, over 1,800 Fibre-Shell houses had been constructed across the USA by 1973 (Ehrenkrantz, 1989).

1968 Futuro – Matti Suuronen, Yjrö Ronkka – Finland

The Futuro, displayed in Figure 2-8, was, perhaps, one of the most famous examples of an FRP structure. It was a modular structure, originally created for meeting a very specific niche. The purpose of the project was to design a ski lodge with good thermal properties. In addition, the ski lodge needed to be easy to assemble on a rough floor and capable of being heated in a short period of time. The final version was obviously influenced by Suuronen’s experience with FRP structures and it was compared to a flying saucer. The walls were constructed of glass-reinforced polyester face sheets bonded to a polyurethane foam core. The method of assembling the structure required bolting the eight lower and eight upper boards on to its tubular steel foundation. To cope with the cold temperature in a Finnish winter, an efficient electrical heating system was installed making it possible to accurately adjust the temperature to an acceptable ambient temperature in less than 30 minutes (Home and Taanila, 2003).
Companies from 20 different countries bought the prototype, which became known worldwide with many types of expositions and construction exhibitions. In the coming year, the areas of commerce, residential, medical, and military forces witnessed the construction of sixty Futuros. Though the design was incredibly popular, the project was an economic failure because of the high costs, poor business management and its quirky appearance, which was hard for consumers to accept. Also, though it was significantly modified when it was used in the military, its fire resistance was still unspecified (Ludwig, 1998).

1968 Kunststoffhaus 2000 System – Wolfgang Feierbach – Germany
This project was one of the speediest to assemble, with workers connecting the 26 wall and roof parts within one day and without the use of any heavy lifting equipment. Instead of using the modular approach, as with previous FRP buildings, the segmental method was employed to achieve more flexibility. As shown in Fig. 2-9, the materials of the components were glass reinforced polyester face sheets of 5mm thickness combined with a polyurethane foam core of 70mm (Audouin, 1969). Also, in order to cover the bolted joints of the parts, flame-retardant sheets were placed in the interior wall.

In the following ten years, over seventy structures were constructed using this system in residential, commercial, industrial situations (Feierbach, 2003). The design of the initial house, with highly stylised inner walls, went out of fashion very quickly, though later on the design was used for creating offices. Discontent still existed regarding the connection details and surface finishes (Ludwig, 1998).

![Figure 2-9: Kunststoffhaus 2000 system (Audouin, 1969)](image)

1971 Maison en Plastique – Jean Prouvé – France
This FRP concept house constructed in St. Gobain, France and displayed in Fig. 2-10, was a representation of Jean Prouvé’s preassembled house structure. Having great similarity with
the Konig system, standardised roof and wall panels, integrated by universal linking matters were employed in the house. The panels’ materials - glass-reinforced polyester face sheets and polyurethane foam cores - enabled almost infinite length production. Neoprene gaskets were used to seal the joints but the production of the elements was the main concern, so ultimately there was little which could be called ‘innovative’ in either engineering or architectural aspects (Huber and Steinegger, 1971).

![Figure 2-10: Maison en plastique project (Huber and Steinegger, 1971)](image)

2.3 Modern FRP building and systems

2.3.1 GE Living Environment Concept House – USA, 1989

FRP buildings were developed significantly by a plastics manufacturer looking to expand their market. The plastic producer, General Electric (GE), initially decided to redesign family housing. As shown in Fig. 2-11, the GE Living Environment concept building was constructed near the headquarters of the company in Pittsfield, Massachusetts, in 1989.

![Figure 2-11: GE living environment house (Wilson, 1990)](image)

With David George, of Richard-Nagy-Martin, as the designer, almost 5 million dollars was spent in the design and construction, while 45 different manufacturers were involved in the collaboration (Wilson, 1990). Described by GE as a “living laboratory”, modern systems and
products were employed in the house, from washing machines to wall cladding. Thanks to the use of recycled products from the automotive and aircraft industries, thirty percent of the house was built from polymeric materials, the largest portion being unreinforced thermoplastics.

2.3.1.1 Structural system
The structure was two storeys tall and set on a concrete foundation. The foundation was poured by polymer form work and, when the insulation and interior surface were finished, it remained in place. For the engineered-wood, corrugated wall and floor panels, conventional timber members were covered for the load-bearing members. Bolts and adhesive bonds were used to connect the parts (Chevin, 1989).

2.3.1.2 Production and assembly
Little automation was employed in the prototype. For future aspects, GE considered a two-phase industrial producing system. Basic panels would be assembled in a large middle factory in the first phase, while these panels would then be transported to smaller factories to add the final finishes and detailed parts in the second stage. Thus, the transportation of the finished panels would be only a short distance from the manufacturing site (Chevin, 1990).

2.3.1.3 Integration of functions
Both the erect structural system and the building shape were supported by the completed wall panels. Weather protection was achieved via thermoset cladding on the exterior and fire resistance, via gypsum, on the interior.

A network of pipes, hidden behind the gypsum layer, had the dual role of heating and cooling, while electrical cables passed through hollow baseboards and door frames. As illustrated in

Figure 2-12: Moulded baseboard raceway (left), utilities embedded in waffle slab (middle), integrated shelf connection system in foundation wall form work (right)
Fig. 2-12, high-density foam waffle slabs were adopted for plumbing below the finished floor with additional plumbing in the walls. Heat recovery, grey water utilisation and many other eco-friendly and energy saving concepts were incorporated in the structure.

2.3.1.4 Discussion
When it was constructed, GE planned to build a second unit by 2000, in which 75% of the materials would be polymeric plastics (Teti, 1989). However, this blueprint was never realised, with interest in the Living Environment house dying out. Consumers were fond of the traditional products which were already available. A single building design must connect many components; though the concept is important, it is far more important to be able to create a real product. Though clearly resembling the Monsanto House of 1956, the methods of integrating building functions were quite successful. In summary, the thought that an FRP building system could be industrially produced was again brought into people’s minds in a conventional way and with an environmentally friendly approach.

2.3.2 Advanced Composite Construction System – UK, 1986-96
Developed by Maunsell Structural Plastics, from 1986 to 1996, the Advanced Composite Construction System (ACCS) is a plain, interlocking FRP panel structure. Pultruded flat panels, which are connected by slotted ties, are contained in the panels. The thickness of the panel is 8cm and the width is 61cm; a cellular cross section, in the shape of a box, is attached. There is a bone-shaped cross-section in the slotted connectors. As shown in Fig. 2-13, single celled elements are used for making corners of 90° and 45°. According to different applications, the components are made by the pultrusion process, with either glass fibre or fire-retardant polyester and vinyl ester.

![Figure 2-13: The main ACCS components (left) and assembled corner section (right)](image-url)
2.3.2.1 Noteworthy projects
The ACCS, developed from an earlier bridge envelopment system by Maunsell, has been used mostly in steel-framed bridges, for protection and to create an inspection platform system. As illustrated in Fig. 2-14, the system’s first applications in structures were on the Aberfeldy footbridge in Scotland and the Bond Mills drawbridge in England.

Figure 2-14: Box-beam assembly (left), Aberfeldy footbridge (middle and right)

The extensive use of the system began when the eight bridges, which form the Second Severn Crossing between England and Wales, were constructed. Looking at Fig. 2-15, it is easy to see that the on-site construction management offices were also built with the ACCS, including the walls, ceiling panels and floor. These two-storey offices, the first application of this system in civil engineering, were transformed into a service centre after the bridges were completed.

Figure 2-15: Severn visitor’s centre (left) and automated car wash enclosure (right)

2.3.2.2 Structural system
Though thin panels are appropriate for a single layer over short spans and for small loads, for larger tasks, dual-wall rigid sections are needed. To prevent buckling, local cells can be filled with expanded foam, as they were in the Bond Mills drawbridge. A conventional beam and slab preparation can be used for large spans in houses too. Connectors and structural
adhesives are used for connecting the elements. Lee et al. (1995) and Duthinh et al. (2001) have performed a series of structural evaluations.

### 2.3.2.3 Integration of functions

This integration of functions regarding the several examples of this system in engineering centred only on the use of panels to bear loads and envelop a building. In 1998, in his book Fiber Reinforced Plastics (Minguzzi, 1998), Minguzzi studied the application of the system in large multiple-storey houses. He planned to include the implementation of the system to invent vacant wall and floor parts, adopting ACCS panels on all of the faces, as displayed in Fig. 2-16. The building facilities were routed in the large space. However, only one direction of transmission of utilities was proposed, and no solutions for entry and exit utilities were proposed. Moreover, the floor panels interrupted the erection pits at each storey. In the end, structural fire endurance received no consideration, though the materials are produced using fire resistant formulations. However, those formulations only prevent fires from being ignited; they do not prevent structural collapse, which may still happen even at low temperature conditions.

![Figure 2-16: Building applications of the ACCS proposed by Minguzzi (1998)](image)

### 2.3.2.4 Discussion

As time passed, in the small group of single and two-storey buildings, problems appeared, with water leakage through the slotted ties being the biggest problem. Resulting from the toggle knots, which are slid into the tracks of the panels, most of the adhesive is pushed out the other end, and the bond effect is poor. Since pultruded components are not always suitable, producing tolerance of elements also creates a problem. Multiple-storey buildings are not a future prospect for ACCS. Supported by the British highway authority, many new
bridge envelopment projects are being carried out with this system; however, only a few small-niche applications, in industrial factories and chemical treatment centres, use it. It is almost impossible for the system to be applied in future primary, load-bearing buildings, as it lacks structural fire endurance capability.

2.3.3 Eyecatcher Building - Switzerland, 1999

Illustrated in Fig. 2-17, the Eyecatcher was developed in 1999 and it displays numerous modern construction materials and techniques, as was seen at the Swissbau, or Swiss Building Fair, in Basel. Being five storeys high, the Eyecatcher is the tallest load-bearing GRP construction that has ever existed. It forms no cold or warm bridges, it has a low possibility of conducting electricity between its members and its skeleton of FRP has to be naked on both the outside and the inside of the building. Cellular FRP sandwich panels make up the building’s outside frame, as a surrounding wall to protect it. The wall is full of aerogel beads and gives one the opportunity to see the inside. It serves a dual purpose, including insulating fire. Since more than 20,000 visitors visited it during the fair, authorities made the decision to disassemble it and rebuild it at its final location, a few blocks away.

![Figure 2-17: The Eyecatcher: under construction (left), completed (middle), translucency (right)](image)

2.3.3.1 Structural System

Both vertical and lateral loads were supported by three trapezoidal trusses composed of FRP components, shown in Figure 2-18 left, which were produced by sticks bound together to single pultruded members. The vertical members were formed of two channel parts, which were adhered to an I-beam, producing a column space with lightweight resistance to buckling. The parallel members were comprised of two sections attached to their flanges, so as to create a rigid box section. Extra sections above the flanges improve the connection and reinforce the stiffness of the members, some of which should be adhered with web plates. After being bonded and bolted, the permanent connections finally connected with each other, while
bolting alone was used for the transit connections, allowing easy disassembly and reconstruction of the structure after the building fair finished.

![Figure 2-18: The Eyecatcher: frame schematic (left), built-up members (middle), frame (right)](image)

### 2.3.3.2 Integration of functions

The following projects demonstrate that panels can offer a strong shear resistance for the lateral solidness of the building, which allow space for the vertical members. When the building was designed, designers made numerous tests on the use of the translucent sandwich panels as a structural instrument and as a result the surrounding of the building cannot be used with the load-bearing system.

### 2.3.3.3 Discussion

In the construction of an active suppression system, the problem of fire safety must be addressed. As soon as a fire breaks out, water sprinklers are activated and the fire department is automatically called. Because the pultruded FRP columns heat up and burn easily, the only possible way to get permission for their use is from a relevant building authority, which may offer a special permit for demonstration structures. The Eyecatcher was destined to be a demonstration project and the objective was impossible to mass produce, which is the point to be remembered. In the same way, the costs and the speed of building seem to be of little consequence beside the fact that a five-flight FRP building is now finished and is, like any other active office building, occupied. So, the building represented a wonderful experiment and plays a fundamental role in paving the way for the popularity of FRP materials in buildings. After all, the project demonstrates that FRP materials can be applied in a material-adapted way, ignoring the flying saucers and loose foam style of the 1950s and 60s.
2.3.4 Startlink Composite House – UK, 2012

As shown in Fig. 2-19, the Startlink lightweight construction structure was backed by the UK Government Technology Strategy Board and commercial partners, covering Larkfleet House, Exel Composites, Costain and OCS Structural Plastics. By building a home of lightweight pultruded fibre reinforced polymer with incorporating off-site fabrication skills, the syndicate created energy-saving and economical housing. To cut the cost of materials, thin or centre-stuffed objects were used by the syndicate, which also developed a system which obviated the need for traditional building materials. An economical two-storey, three-bed roomed house was built by the consortium in June 2012, and the Passivhaus standards were matched automatically. The goal of the Startlink system is to improve construction ability and adjustment range; thus, site waste is avoided and it calls for no skilled workers, wet trades or concrete. The building design has included and incorporated of water saving measures and flood resistance. The Startlink composite building is a fair resolution to the problems of CO², energy waste and high cost.

2.3.4.1 Structural System

Without using concrete or structural steel, Startlink uses ten new glass fibre pultruded profiles, together with several normal profiles, to build a modular system, shown in figure 2-19. The fundamental parts are a ground panel, 6mm thick perimeter profile and flat panel with lightweight cores. Except for the 5mm timber, other profiles are 3mm thick or less. Each side of the flat panel has short return legs, which support 6mm round extruded gaskets. Pairs of lined branches are inserted into a single pillar and each side of the deeper quadrate columns, to complete water-resistant seals in the walls and the roof, of 240mm and 120mm thick. They are locked in place and connected with the floor panels by a buckle set channel, 50mm x 25mm. The goals of the timber profile are to secure the floor panel and then divide it into two parts, to develop the upper and lower strings of a roof bunch. Attached to floor panels at 600mm cores, it enables a range of 5m at 1.5 kN/m² burdening and deviation constrained to L/360. The panel is so light that it can be carried by two people and the lower part of the timber, together with the flat panels, forms a ceiling.

The distance from floor to the ceiling is 240mm and ample room is left for sound proofing and insulation. To make a building which is well insulated, 225mm of insulation is placed within the 240mm thick exterior walls, and even thicker insulation installed in the roof as well. Moisture and air permeability will be resisted by sealing gaskets. The walls are
connected by two remaining small profiles, at an angle of 90 degrees or other various angles, shown in figure 2-20. Integral modular plumbs, beneath buckle set passages, enable easy setting-up of click-fix modular pipes, and cable systems are put into walls and floors. By embedding Unistrut nuts, which are limited to passages in struts and floor panels, the same pipes allow blind fixing.

Figure 2- 19: Startlink composite house (Hutchinson and Singleton, 2007)

Figure 2- 20: Startlink components (Hutchinson and Singleton, 2007)

2.3.4.2 Discussion

Founded on 10 pultruded profiles that connect together, and using bolts and buckles to build houses quickly, the Startlink system is a modular building system. The materials have remarkable characteristics that are almost unknown in the industry, but which are incredibly useful for construction. These include lower heat transmission and expansion when compared to steel, brick and concrete. It is stronger than steel, with good fire-proofing and sound insulation. Moreover, it is steady, inactive and impervious to water, and is the only insulation
needed when building a house. A Startlink building, with proper insulation, has lower embedded energy than a timber framed house. Site waste and the costs of transporting and assembly are avoided because of the light weight and prefabrication of sections. Though lightweight buildings are easier to build, thermal mass and heat are hard to balance in the summer time. Startlink proposes a resolution, which designs a “green” roof for the system and retains water for evaporative cooling and the thermal mass of the earth. The low-preserving system is simple to change and re-utilise, and increases the probability through significant energy saving. Because the Startlink system does not use steel or brick in building, it is rapid to construct, 25% cheaper than conventional houses and, also, environmentally friendly.

2.4 Conclusions
Four modern construction systems using FRP materials have been studied: ACCS, GE The Eyecatcher and Startlink. The ACCS is more suitable for bridge enveloping than for houses because of the difficulties stated in Section 2.3.2 related to link performance, although it was successful in the resurrection of the plastic concept of the 1980s. The GE house system does not include the structural application of FRP materials and is not wholly relevant to this programme. Consequently, only two systems are left: the Eyecatcher building and the Startlink construction system. The whole FRP building system, the Eyecatcher building, was constructed easily and swiftly. The project demonstrates that FRP materials can be applied in a material-adapted way, ignoring the flying saucers and loose foam style of the 1950s and 60s. However, the major problem is a fire safety issue. Load-bearing systems, with GFRP column and beams, have poor fire resistance properties. The last system, Startlink, seems to enable more rapid and easy fabrication, with standard pultruded profiles. These enable the building cost to be reduced greatly, while good insulation and zero waste make this building 25% cheaper than conventional houses. It is also environmentally friendly. However, the main problem is that the building’s overall dimensions are restricted when made with glass fibre material.

For the construction of load-bearing FRP structures, several building systems can be employed, though very few can prop over one storey and none can prop more than three storeys or higher. The ability for building taller houses with FRP materials was illustrated with the five-storey Eyecatcher Building, though the Eyecatcher requires structural fire
resistance. Although many single and two-storey FRP systems exist, there is no available manufacturing system, no structure that has been manufactured, and no positive opinions on methods that would enable multiple-storey FRP houses to be built. The reason for this is mainly that no material-adjusted proposal has been conceived to produce the necessary structural fire resistance to FRP parts that is imposed under the pressure of a multiple-storey house. Methods which use the preservation of FRP parts via superficial sheets neutralise the benefits demonstrated by the material and are neither material-suitable nor economical. The development of FRP materials for ease of construction and to promote the long-time service of multiple storey buildings relies on the potential of economic approaches to produce enough structural fire endurance.

In summary, carbon fibre FRP could become a suitable construction material because of its high strength for large buildings and good thermal behaviour for developing a low carbon construction industry. Pultruded panels are suitable for making floor slabs, wall panels and claddings etc. in buildings considering manufacture quality, material saving and easy installation and having standard construction components. Therefore, a CFRP pultruded panel system is proposed in this investigation for flooring in buildings.
3

DESIGN GUIDELINES
3.1 Introduction

By the late 1960s and early 1970s a number of pultrusion companies were producing ‘standard’ I-shaped and tubular profiles. The Structural Plastics Research Council (SPRC) was established in 1971 by the American Society of Civil Engineers (ASCE). A manual, called the Structural Plastics Design Manual (SPDM), was published in 1979 as an FHWA report and, subsequently, by the ASCE in 1984 (ASCE, 1984), for the design of structural plastics (McCormick, 1988). This guide was not restricted to pultruded profiles. In 1996, a European design guide for polymer composite structures was published (Clarke, 1996), and, in 2002, the European Union published the first standard specifications for pultruded profiles (CEN, 2002a). In order to study the design and application of the early 1980’s composites, and to address those FRP composites made over the past few years, the technical committee on Structural Composites and Plastics (SCAP), of the American Society of Civil Engineers (ASCE), published a guide book to deal with those issues. The ASTM D30.30.01 (Composites for Civil Engineering) solved the problems of FRP composites which have been used in the engineering process. There is a subcommittee, named “T-21 Composites”, set up by the American Association of State Highway and Transportation Officials (AASHTO) Bridge Committee, which is studying and forming a manual for composite usage in bridge construction, covering concrete patching, FRP concrete reinforcement and vehicular path deck plates. There are also a number of different professional institutes which have established standards, codes, test means and specific regulations. To summarise the standards and specifications of FRP composites, table 3.1 is created (MDA, 2000).

Table 3.1 Summary of Standards Committees for FRP composites (MDA, 2000).

<table>
<thead>
<tr>
<th>Organisation</th>
<th>Committee</th>
</tr>
</thead>
</table>
| American Concrete Institute (ACI) | 440 – Composites for Concrete;  
440C – State-of-the-art Report;  
440D – Research;  
410E – Professional Educations;  
440F – Repair;  
440G – Student Education;  
400H – Reinforced Concrete (rebar);  
440I – Pre-stressed Concrete (tendons);  
440J – Structural Stay-in-Place Formwork; |
<table>
<thead>
<tr>
<th>Organization</th>
<th>Guidelines</th>
</tr>
</thead>
<tbody>
<tr>
<td>American Society of Civil Engineers (ASCE)</td>
<td>440K – Material Characterisation; 400L - Durability</td>
</tr>
<tr>
<td>American Society of Testing and Materials (ASTM)</td>
<td>Structural Composites and Plastics</td>
</tr>
<tr>
<td>American Association of State Highway and Transportation officials (AASHTO) Bridge Subcommittee</td>
<td>D20.18.01 – FRP Materials for Concrete; D20.18.02 – Pultruded Profiles; D30.30.01 – Composites for Civil Engineering</td>
</tr>
<tr>
<td>International Federation of Structural Concrete (FIB)</td>
<td>T-21 - FRP Composites</td>
</tr>
<tr>
<td>Canadian Society of Civil Engineers (CSCE)</td>
<td>Task group on FRP</td>
</tr>
<tr>
<td>Japanese Society of Civil Engineers</td>
<td>ACMBS – Advanced Composite Materials for Bridges and Structures</td>
</tr>
<tr>
<td>Transportation Research Board</td>
<td>Research Committee on Concrete Structures with Externally Bonded Continuous Fibre Reinforcing materials</td>
</tr>
</tbody>
</table>
3.2 Structural design guideline

The development of standards and regulations for FRP usages in the construction field is ongoing, for the moment. Some international organizations, which are connected to the composite business, have been doing research and analysis of the structure of composite, by both micromechanics and Finite Element Analysis (FEA) (Seracino, R., 2005). Positive results have been achieved from this research and have been used for developing design standards and methodology specifications for producing FRP composites in the future. The consensus is that in order to ensure good performance and safety within the life span of the design under the defined conditions, FRP systems must have adequate rigidity and strength. More specifically, when designing the deck systems, external forces, such as collision or fire hazard, have to be considered to avoid disastrous failure (GangaRao, et al., 1999). All in all, the FRP composites design methods can be divided into two categories: working-stress design and load-resistance factor design (GangaRao, and Siva, 2002).

For the working stress design, many safety factors are considered to achieve long-term durability, temperature degrees, diffusion effects, edge effects, ignitability and so on. And, to control the pressure level at the laminate parts, proper failure standards are used together with safety margins. For instance, the stress of the fibre direction under full load circumstances must be within 20% of the minimum strain capacity. To get a precise test result, all failure tests, including compression, pressing, bending and other forces for the laminate, shape and
system aspects, should be carried out. What’s more, some tests have to be conducted at the system level for:

**Deflection criteria**

Appropriate limiting values of deflection, taking into account the nature of the structure, finishes, partitions and fixings, and the function of the structure, shall be agreed by the client. EUROCOMP Handbook (Clarke, 1996) recommends deflection use L/250 of the span of the poling length.

\[ \delta_{max} = \text{the sagging in the final state relative to the straight line} \]
\[ \delta_1 = \text{the variation of the deflection of the beam due to the permanent loads immediately after loading} \]
\[ \delta_2 = \text{the variation of the deflection of the beam due to the variable loading, plus any time dependent deformations due to the permanent load} \]
\[ \delta_0 = \text{the pre-camber (hogging) of the beam in the unloaded state} \]

**Strength criteria for FRP laminates**

In general 3-D states of stress, the Tsai-Wu criterion is proposed to predict that failure will not occur if the following inequality is satisfied (Tuttle, 2004).

\[
X_1\sigma_{11} + X_2\sigma_{22} + X_3\sigma_{33} + X_1\sigma_{11}^2 + X_2\sigma_{22}^2 + X_3\sigma_{33}^2 \\
+ X_4\tau_{23}^2 + X_5\tau_{13}^2 + X_6\tau_{12}^2 + 2X_{12}\sigma_{11}\sigma_{22} + 2X_{13}\sigma_{11}\sigma_{33} + 2X_{23}\sigma_{22}\sigma_{33} \leq 1
\]  
(3.1)
where $\sigma_{11}$, $\sigma_{22}$, and $\sigma_{33}$ are the stress components in longitudinal, transverse and through thickness direction respectively.

The strength related constants $X_1$, $X_2$, $X_3$, $X_{11}$, etc. can be determined based upon individual strength measurements, i.e., $\sigma_{11}^{fl}$, $\sigma_{11}^{rc}$, $\sigma_{22}^{fl}$, $\sigma_{22}^{rc}$, as shown below.

\[
X_1 = \frac{1}{\sigma_{11}^{fl}} - \frac{1}{\sigma_{11}^{rc}}; X_2 = \frac{1}{\sigma_{22}^{fl}} - \frac{1}{\sigma_{22}^{rc}}; X_3 = \frac{1}{\sigma_{33}^{fl}} - \frac{1}{\sigma_{33}^{rc}};
X_{11} = \frac{1}{\sigma_{11}^{fl}} - \frac{1}{\sigma_{11}^{rc}}; X_{22} = \frac{1}{\sigma_{22}^{fl}} - \frac{1}{\sigma_{22}^{rc}}
\]

\[
X_{33} = \frac{1}{\sigma_{33}^{fl} \sigma_{33}^{rc}}; X_{44} = \left(\frac{1}{\tau_{23}}\right)^2; X_{55} = \left(\frac{1}{\tau_{31}}\right)^2; X_{66} = \left(\frac{1}{\tau_{12}}\right)^2 \tag{3.2}
\]

The coefficients $X_{12}$, $X_{13}$, $X_{23}$ can be determined using equi-biaxial tests. Also, $X_{12}$, $X_{13}$, $X_{23}$ can be calculated using the following equations.

\[
X_{12} = -\frac{1}{2} \sqrt{X_{11}X_{22}} = \frac{-1}{2\sqrt{\sigma_{11}^{fl} \sigma_{11}^{rc} \sigma_{22}^{fl} \sigma_{22}^{rc}}} \tag{3.3}
\]

\[
X_{13} = -\frac{1}{2} \sqrt{X_{11}X_{33}} = \frac{-1}{2\sqrt{\sigma_{11}^{fl} \sigma_{11}^{rc} \sigma_{33}^{fl} \sigma_{33}^{rc}}} \tag{3.4}
\]

\[
X_{23} = -\frac{1}{2} \sqrt{X_{22}X_{33}} = \frac{-1}{2\sqrt{\sigma_{22}^{fl} \sigma_{22}^{rc} \sigma_{33}^{fl} \sigma_{33}^{rc}}} \tag{3.5}
\]

### Strength criteria for FRP pultrusion components

1. **Hashin failure criteria**

Hashin failure criteria were originally developed for unidirectional polymeric composites, and hence their applications to other type of laminates and non-polymeric composites are approximated. Failure indices for Hashin criteria are related to fibre and matrix failures and they involve four failure modes. The criteria are extended to three dimensional problems, in which the criteria for both tension and compression are considered. (Hashin, 1980).

2. **Joints and fasteners with adhesive bonding design**

   (a) Adhesive selection should be based on previous experience or on a specific selection process.

   (b) Preliminary adhesive selection should be performed using any unbiased method
which includes all of the factors required for a reliable selection procedure. Guidelines of applicable selection procedures are given in the EUROCOMP Handbook (Clarke, 1996)

(c) Currently, any selection process can do no more than suggest one or more generic types of adhesives that are worthy of more detailed examination.

(d) A detailed selection, within the most promising groups of adhesives, can be chosen based on information provided in this code, in published adhesive data or in data sheets of adhesive manufacturers.

(e) Factors to be considered in adhesive selection are:

- adherent materials
- environmental factors
- applied loading
- joint geometry restrictions
- bonding and curing processes
- cost
- special requirements, including health and safety.

(f) The material compatibility between the adhesive and the adherents shall be checked, in cooperation with the adhesive manufacturer or by testing, as the possible material incompatibility may significantly reduce adhesion.

(3) Allowable stress design

Allowable stress design (ASD) is based on the philosophy that the safety of a structure is obtained by ensuring that no structural member reaches its ultimate strength under nominal service loads. This is accomplished by ensuring that the design stress (representing any stress eg. Shear stress, tensile stress, bearing stress) in every member, or the required stress, \( \sigma_{\text{reqd}} \), obtained from calculations using elastic theory and using the nominal service loads on the structure, be less than the ultimate strength, \( \sigma_{\text{ult}} \), of the material used in the structural member, divided by a factor of safety, SF. The ultimate strength divided by the safety factor is termed the allowable stress, \( \sigma_{\text{allow}} \), and the fundamental ASD equations are given as

\[
\sigma_{\text{reqd}} \leq \sigma_{\text{allow}} \quad (3.19)
\]

\[
\sigma_{\text{allow}} \leq \frac{\sigma_{\text{ult}}}{SF} \quad (3.20)
\]

\[
\sigma_{\text{reqd}} \leq f(\text{loads, geometry, elastic}) \quad (3.21)
\]
With respect to serviceability, ASD philosophy is to assign different factors (analogous to strength safety factors) to the material moduli, to determine deflections and displacements under the nominal service loads. The nominal service loads used in design are obtained from building codes or model load codes (ASCE (2002). Allowable stress design is also recommended by EUROCOMP Handbook (Clarke, 1996), for designing FRP laminates and sign support structures.

Thinking of those actions which are likely to happen during service, the design of load-resistance factor approach is applied in the structure. With this design employed, adequate durability is accomplished while accepting the probable failure. Multiple elements should be considered while calculating the probable failure, including randomness effect, analytical uncertainties of material and the variability that may occur during the process. Apart from that, when ensuring the serviceability limit of the deck design, ductile failure should be accounted for by the design, as well. The ultimate limit state is quite important for the approach to act successfully, since that error of component level could be vital for the whole. For example, referring to FRP coupon and deck test data, the following limit states should be identified and quantified while the design method is employed:

- external stress, rotation effects, deflection or vibration, and human effects;
- damage
- lateral-torsion, buckling forces

In addition, opinions suggest that the internal force effects should be modified depending on different levels, such as ductility at shape or system level, redundancy, as well as operational performance. Likely, power equations, suitable width of the deck and the distribution of load on the transverse level and knock-down influences are advised in aspects of shape, temperature, material and so on.

The suggestions of the Federal Highway Administration (2006) are found in the following guidelines for design and usage of FRP materials in bridge construction. Many of these advices have already been explained above.

1. For long term durability’s sake, the preset strains with full load in design case will be no more than 20% of the minimum extreme strength of FRP composites and the final strength level depends on the test experiment and stated in the authorised plans.
(2) To avoid degradation of the product over time, environmental factors, i.e. knock-down effects of 0.65, shall be employed in the materials.

(3) Since the elasticity of the material is quite low, a large part of the design will perform through the driving of deflection limitations rather than strength means.

(4) When the deck fails in a way other than the laminate’s tension, safety factors need to be employed in the design. For factors of reliability with FRP composites, the AASHTO will develop a load and resistance factor design (LRFD).

(5) Sustainable paint should be used to give the exposed surfaces protection from ultraviolet.

(6) To install an FRP deck on a steel superstructure, more attention must be paid to the attachment method. The composite of the FRP and steel cannot be easily calculated in capacity. However, when load is unavoidably to be transited between the two, the conjunction itself must be more careful in case of failure. When the interface is designed, two aspects should be considered: to ensure the connection action and to provide a slip mechanism.

(7) Once exposed to direct sunshine, FRP material can rapidly heat up, while a dark paint on the surface will make the phenomenon more serious. We’ve learned from experience that when the temperatures of the surface rapidly vary while the heat cannot be dissipated by the FRP promptly, a thermal gradient will appear from the bottom of the deck to the top. This problem has a seasonal nature, in that it is more likely to happen at the beginning of spring and the end of autumn. Therefore, a 100 °F temperature gradient is assumed between the top and the bottom parts of the deck, in case of any internal thermal issues. Moreover, the temperature difference will result in the top part emitting heat faster than the bottom does, so the selection of supporting and anchorage materials may also be influenced. To understand that the thermal pressure occurring may equal the stress produced from live loads is very important for safety in application.

In summary, EUROCOMP Handbook was used as a main design guideline in this project, which includes strength check (ultimate limited state) and deflection check (serviceability limited state). In strength check, Hashin failure criteria was employed to control maximum stresses in all directions within limitation. In deflection check, unfactored design load was used to carry out practical deformation under allowable values.
3.3 Material selection

There are many different kinds of fibres which can be used, and each has its own merits and drawbacks. However, the three most used fibres in civil building and construction are glass, Kevlar TM and graphite. To choose the right fibre for certain uses, one needs to think about many different factors, including the characteristics of the fibre itself, such as strength, stiffness, durability and external factors, such as its cost limitations and whether it’s easy to get the component parts. Figure 3-2 shows the currently available fibres’ stress-strain curves. It’s necessary to mention that these curves are for pure fibres, without any influences of the polymer matrix considered. Three main different fibres, i.e. glass fibre, carbon fibre and Aramid fibre will be introduced in the following sections.

![Stress-strain properties of typical fibres](image)

Figure 3-2: Stress-strain properties of typical fibres

3.3.1 Glass fibres

Glass fibres are produced in a process known as direct melt. By drawing rapidly and continuously from a glass melt, fibres are formed, with a diameter of 3 to 25 microns. Since glass fibres are cheapest, they are the most often used in engineering. Several different models of glass fibres are available, with the most basic being E-glass. R-glass costs are higher but it is more durable. The characteristics of glass fibres are high strength, low thermal conductivity and moderate flexibility and density. For those structural applications which are not concerned with the weight and deflections caused by low elastic capacity, glass fibres are a good choice, such as the production of FRP-reinforcing bars, pultruded fields, FRP packing materials and so on. Dating back to the early application of glass fibres, referring to previously mentioned FRP buildings, the buildings are small because of the lack of rigidity.
and the flexibility of glass fibres, even though the cost is low.

3.3.2 Carbon fibres
Carbon fibres are produced through a process of controlled pyrolysis, in which the potential precursor fibre will receive a complicated set of thermal treatments, including stabilisation, surface treatment, carbonisation, and so on, and then carbon fibrils are produced. Moreover, these fibrils are 5-8 microns in diameter. The fibres produced have various properties and they can be divided into several categories according to their elasticity strength.

- standard, 250-300 GPa
- intermediate, 300-350 GPa
- high, 350-550 GPa
- ultra-high, 550-1000 GPa

Carbon fibres are more expensive than glass fibres, but they are beginning to become the most commonly used material in structural construction currently. For example, they are used in concrete pre-stressing, FRP packages, repair, slabs, and so on. Carbon fibres are becoming popular because their price is decreasing and, compared to glass fibres, they are of higher in elasticity and strength, lower in weight, and excellent in thermal proof and anti-jamming ability to chemical or environmental influences. For those applications which are sensitive to density and deflection, carbon fibres are definitely the perfect choice. Due to its high grade of stiffness and good strength, carbon fibre is the ideal material for large building engineering and was selected for making proposed floor panels in this investigation.

3.3.3 Kevlar TM (Aramid fibres)
Aromatic polymerisation is the process used in the creation of aramid fibres, also called extrusion and spinning. There are two different gradients of stiffness available for aramid fibres, i.e. 60 GPa and 120 GPa. Kevlar TM has the following characteristics: high strength, low density and general elasticity. Moreover, due to the special anisotropic characteristic of aramid fibres, the FRP composites produced from Kevlar TM are of low compression. However, ultraviolet and moisture exposure can damage aramid fibres and lead to degradation.
3.4 Proposed Manufacture

FRP components can be produced in many different ways, though only those methods which come into force promptly will be discussed in the following paragraphs. Pull-winding, vacuum bag moulding, resin transfer moulding and injection moulding will be studied in the texts of special composite matters. However, two common manufacture techniques, i.e. pultrusion and hot press moulding will be discussed in detail below. Because pultrusion was chosen as a manufacture technique to conduct proposed CFRP floor panel, and hot press moulding was used in conducting scaled floor panels for experimental test.

3.4.1 Pultrusion

Pultrusion is often employed in the manufacturing of FRP bars, rods, plates, tendons and other structural parts. This method is totally automatic and, consequently, highly convenient. In some aspects, the extrusion technique, which fabricates metal parts, has similar properties with pultrusion. As indicated in Figures 3-3, the steps of the pultrusion method are, firstly, pulling raw materials through a resin bath and then through a warmed die. A structural component is produced by impregnating the resin with fibres and passing it through the die. Finally, the polymer matrix hardens into the form of the die. The cured end is where the FRP components are manufactured. The good point about the process is that it can fabricate any length of component. Note that a unidirectional FRP is created by aligning all of the fibres along with the component’s length.

The size of the pultruded profiles can range from the minimum size of 3mm diameter to the maximum size of 1m wide and 250mm deep, and the speed can range from 12m to 180m per hour, depending upon the size and complexity. The preferred minimum corner radius for internal radii is 1.5mm, and so to keep a unified thickness of the wall throughout the corner, the equivalent should be achieved between the external radius and that of the internal part plus the wall thickness. The process speed refers to how quickly the resin can be heated. For the thick part, it will take a longer time to reduce heat and trigger responses and it will operate more slowly than the thin parts. When it starts curing, it will result in exothermic response, which rapidly builds up the internal temperature, and may cause overwhelming stress and cracking. So, to reduce the thickness of the wall, as much as possible, it is very necessary to use the pultrusion method.
Because the thickness issue is inevitable, schedule, which is less sensitive to temperature, is applied to guarantee the control of the process. The range of thickness in practice is a minimum thickness 2 mm and maximum thickness 20 mm (Clarke, 1996). But, for special situations, 1-50mm is possible, too, while a bar with a diameter of 75mm has been manufactured.

Figure 3-3: Schematic showing the pultrusion manufacturing process (Strongwell, n.d)

3.4.2 Hot press moulding
As illustrated in figure 3-4, hot press moulding is created and used in the manufacturing of composite material in mass production. A hydraulic press is strong enough to be applied in a platen area, which has a pressure of about 100 tonnes/m$^2$. Between the platens, a matched metal instrument is placed, and the cavity between the two halves needs to be adjusted in order to produce the required shape. The heat of the tool should always reach about 130-170°C, no matter whether the heat is from the heated platens or produced directly by cartridge heaters.

After the cure, lasting about 2-3 minutes, is finished, the tool, which is closed with the polymer mix and reinforcement, is opened, and the component is removed. With a maximum of about 3m$^2$, the size of the mould, which can be compressed at will, is adjusted according to the size of press, whose typical size is 0.5 m$^2$. Compression moulding is only applicable for high volume requirements, whose typical components number 10000 or more, on the condition of the high, major investment in the press and instrument.

As the cure is taking place, the applied pressure at the closure of the die is restricted and stopped. Hence, the volume of the resin in the die changes because of the chemistry of the
cure and the thermal expansion, or the contraction effects. The shape of the component, during the transition from liquid to solid, does not change because of the existence of the applied pressure.

The type of resin, the level of curing agent and the depth of the component decides the time that the cure will take. As usual, thicker parts need a long time to heat through and they can create extra exothermic temperature. Hence die temperature is always lower for thick sections, which are, therefore, more difficult to mould. Generally, it takes two minutes for the cure to take place, after which the press is released and the components open with the help of a mechanical ejector mechanism found within the tool.

![Figure 3-4: Hot press moulding (Clarke, 1996)](image)

3.5 Connection design

The connection between FRP panels and supporting beams can be performed in many ways, such as mechanical connection, adhesive reinforcement or using both. No matter which is chosen, design codes, environment and the conditions during construction are all elements that can prevent the two parts from connecting. We can divide ways of connection into two categories: those with composite action and those without. Owing to composite action, most systems do not change in-plane shear when they choose mechanical connection and the composite action between the steel girder and FPR panel can function well when adhesive bonded connections are used. Whichever connection is chosen, the parallel in-plane and upright pressures between the deck and girders must be changed.
3.5.1 Bolted connections

It is believed that holes and cut-outs can weaken FRP components. This contributes to the production of large stress concentrations because of discontinuities and the lack of plasticity. With the large, uni-directional reinforced sheet, the stress concentration at a circular can be as large as 8, while isotropic materials need a value of 3. Thereafter, the connection between FRP panels and girders cannot use bolted connections.

3.5.2 Clamped connections

To take advantage of the open section of the FRP panel, clamped connections need to be used. The clamped bonding is a steel plate with a cue end, which is only suitable for the bottom flange of the panel. As the steel beam and steel bolts have lots of holes, one needs to make sure that a clamping connection is installed on the top flange of the beam. One important function of the clamping device is in providing force against horizontal and vertical movement of the panel. The clamped device has three massive benefits: it is easy to install, disassemble and maintain.

Compared to mechanical connection, adhesive-bonded connections have the advantage of eliminating stress concentration. This point is significant when brittle materials, such as FRPS, are used. What's more, no one needs to worry about the composite action between the deck and the girder.

3.5.3 Bonded connections

Compared to the widespread use of adhesive devices in many fields of industry, such as the automotive industry, its application in civil engineering is rare. The most common applications are the reinforcement of concrete columns and beams with carbon fibre bound to polymer strips, the bonding of facade elements to steel formation and adhesive fillings for sealing up fugues. However, as a way of connecting load-bearing elements, bonding is not usually used.

AASHTO found bonded connections in FRP and in steel girders used in bridge engineering. Being an adhesive tool, the polyester plays a role in the construction of a 56-m-span steel bridge in Marl, an example of using bonded connections. Because of the incomparable function that hybrid connections showed in joining structural parts, we can see how important the adhesive is in improving joint stiffness.
While the infrastructure has many thick parts to be connected and there is difficulty in controlling the quality of the machine’s assembly, the adhesive joints cannot be the single method used. In this case, adhesive film needs to be added to this system. At first, the adhesive sheet will be cut and put under the bonding platen. Then, it must be softened and heated with a fire gun. The second step involves placing the panel on top of the adhesive film and waiting for them to become bonded fully. It is the adhesive bonded joint which was chosen by this investigation to form a connection between CFRP floor panel and supporting steel beams.

3.6 Fire design

3.6.1 Design requirements

Classifying of details about the design of fire safety engineering (Clarke, 1996):

- Building code requirements, or building regulations.
- On the basis of the minimum number of suggestions, the designer should consult with the client, just as a way of explaining the rules, codes or competent authorities indicated.

When deciding the fundamental requirements, the engineer needs to take the following elements into consideration: the resins compounds of which the FRP composite are to be made and the fire resistant properties.

- The application of the structure, whose parts include the components.
- The direct consequence of fire about such components, including flammability, heat generation, smoke emission, toxic and noxious fumes.
- Other effects of fire destruction: fixings other attachments to the structure, and the results of failing in jointing components, punching or loss of strength, stiffness of components.
- Fire is not the direct cause of making damage of the components of FRP composites on parts of the structure.
- The acceptability level of destruction is another factor that should be taken into account, relating to the repair or replacement of fire-damaged components and the life span when the remedial works have been carried out.

3.6.2 Performance Criteria

The following elements shall be taken into consideration in the fire design of components
made of FRP composites (Clarke, 1996):
- The ability to ignite
- The difficulty to ignite
- Restrictions to flame spreading
- Owing to increase in temperature, the degree of strength loss.

The following factors, regarding the safety of not using structural performance, should also be taken into account:
- The emission of smoke
- Emission of toxic and noxious fumes

Fire resistance of the structural elements made from FRP composites, is decided by methods of structural fire safety engineering, which need to be accepted by the relating authority on the condition that the fire load is low, and that the possible sources of combustion should be kept with enough distance from the FRP-composite components, so as not to ignite the material within the period of fire resistance.

There are three types of methods to use in designing components made of FRP composites for fire conditions, the active one and passive one, or or combination of these two. The following method belongs to the active method category (Clarke, 1996):
- Fire observation and alarm systems
- Fire suppression systems
- Supply of adequate ways of escape, compartmentalisation of buildings, fire stops and fire doors.
- Restrictions on the use of combustible and flammable materials.

Passive methods include:
- Using non-combustible or low fire-hazard materials or constituents or additives to protect structural members.
- use of surface coatings
- Intumescent surface coatings.

FRP composite is not the only active method to be applied, but it is the principal one among the active methods relating to fire safety. Passive protection systems can be divided into the
composite type and the fire barrier type. Low fire-hazard resins and additives of the composite type are assembled in the gel-coat, or gel-coat and lay-up resins, which are used in the FRP composite. In order to reach the ignition temperature, the temperature of the mechanical properties of the composite are degraded, and the fire retardant coatings or casings, accompanied with the fire barrier type, need to be used to lengthen the time to reach the degree of temperature desired, and to control the spread of flame. Heat resistance, flame retarding and insulation, or even a combination of these, will be relevant.

The fire design of the system of fire penetration includes systems and methods to offer adequate fire barriers around pipes, services and other perforations, through the FRP composite components. Special consideration shall be taken of the behaviour of joints, which may be included in the design of structure to change pressure from one component to another. When the fire has burnt out, or is completely extinguished, an assessment should be made to check and analyse the condition of the composite, with particular attention paid to the effects at any connection point to metal. In structurally indeterminate systems, fire damage may alter the relatively sturdy parts of the structure and may redistribute forces. Because of this action the buildings overall structure may change.

3.7 Conclusion

In this Chapter, material selection, design criteria, production manufacturing and joint connections are reviewed. In this investigation, carbon fibre reinforcement polymer (CFRP) was selected due to the excellent strength-weight ratio. As seen in the design guidelines above, the CFRP advanced structure should be checked for maximum stress, deflection and, also, the Hashin criteria. All guidelines for checking are from Eurocomp. The pultrusion manufacturing method would be the best option, with its high quality, high amount of production, and easy producing aspects. An adhesively bonded joint was firstly selected to connect CFRP floor panels with supporting steel or concrete beams.
PROPOSED FRP FLOOR PANEL SYSTEM
4.1 Introduction

The first step in the conceptual design of new buildings with FRP components is to review the contemporary and historical construction projects that have involved the use of load-bearing FRP elements. Section 2 in Chapter 2 presented a historical review of the relevant projects from the early phase of FRP use in construction, which took place between the 1950s and the mid-1970s. Section 3 in Chapter 2 highlighted the relevant projects from the second phase, which began in the late 1980s and has been gaining momentum ever since.

Through this review, it was determined that the lack of success of such systems was directly linked to the use of FRP in manners that were not material-adapted or appropriate to the material. This is not surprising as the introduction of new building materials is usually followed by an initial material substitution phase, a phase in which the methods and details developed from traditional materials are applied to the new building components (Dooley, 2004). In addition, most of the designs were only for small buildings or residential buildings, which were not good models for integration. A strong emphasis was placed on employing FRP materials in manners that did not demonstrate the full recognition of their unique characteristics.

It is important here to define the term “material-adapted,” with respect to FRP materials. This understanding was achieved by compiling lists of the strengths and weaknesses of FRP materials in comparison to traditional building materials. The key advantages of the materials were found to be:

- Excellent strength-to-weight ratio
- Good environmental resistance
- Low thermal conductivity
- Low permeability to air and water
- Low thermal mass
- Facilitated part-count reduction by integration of components during fabrication
- Easy to produce in complex shapes, textures, and through-thickness colours
- Allows transmission of light

There are some characteristics, however, that place FRP materials at a disadvantage when comparing them to traditional building materials:
Chapter-4: Proposed FRP Floor Panel System

- Resins are combustible and have low maximum operating temperatures
- High unit cost
- Low stiffness (glass fibre compared to structural steel)
- Low hardness and, thus, low resistance to cosmetic damage
- Must be treated differently than the materials that have been in the hands of builders for hundreds of years; there is no inherited tradition.

Wherever possible, conventional design concepts were adapted to overcome the weaknesses listed above. Most notably, the issue of low operating temperatures was resolved through the adaptation of a solution that has been in use in other fields of engineering for decades but is unprecedented in the field of load-bearing FRP structures, internal liquid cooling. A well-established design principle, parts/systems integration, was also adopted, to minimise the issue of high unit cost. In the proposed system, the fire protection, climate control, and thermal storage systems were combined into a single system. Further, the structural system and building envelope were merged to reduce components and achieve greater overall efficiency. As such, the proposed floor system reflects the innovative application of well-established design elements that are both effective and appropriate to the material.

4.2 Conceptual design of proposed CFRP floor panel

Based on the review of building materials given in Chapter 2 and design guidelines presented in Chapter 3, the investigated CFRP floor panel is to be made from carbon fibres and epoxy polymers, and to be produced using pultrusion technology. The CFRP panel, as a proposed standard component, will be pre-manufactured in factories; the installation of the panel system will take place on site. The proposed CFRP floor panel, shown in Fig. 4-1, is designed as a pultruded, one-way spanning beam with an open cross-section.

![Figure 4-1: A CFRP panel and the cross-section view](image)

The fibres are orientated along the length of panel, to take action against bending over the span. The floor panel is supported by steel or concrete beams, and it can be connected by
either adhesive joints or mechanical joints with assistance from the longitudinal stoppers for location. Fig. 4-2 shows, schematically, the adhesive connection between a CFRP panel and a supporting steel beam. The overall size of the basic floor panel investigated is 6m in length, 0.5m in width, and 0.13m in height. The design of a two-cell, rectangular, open cellular-section has two advantages. The first is the integration with heating, cooling, fire sprinkler, ventilation and lighting systems by a suspended ceiling system. The second is the low production costs associated with pultrusion. Fig. 4-1 also shows the cross-section view of the CFRP panel with varied thickness. The notable feature of the cross-section is a varied inner arc; a small inner arc was given at the corners, and a large inner arc was given at the middle of each segment of the cross-section. This design aims to reduce the stress concentration at the corners. Two short flanges, on each side of the panel, were designed for connection between the panels, which are adhesively joined together to achieve structural integrity.

However, the CFRP composite material has two major disadvantages in engineering structures. Firstly, the material is brittle and has low impact resistance. Secondly, its fire resistance is not high enough to meet the criteria required by fire safety engineering design. These problems can be solved by the raised floor and suspending ceiling systems. The raised floor system is designed to sit on the top of the CFRP panels, as shown in Fig. 4-2. It consists of a gridded aluminium frame that provides support for a removable square, which could be 60×60cm² and made from timber or polymer plates. The height of the legs/pedestals can be
adjusted to the size of the cables and the other services required underneath, but they are typically arranged for a height clearance of at least 15cm (Access Floor Corp, n.d.). The raised floor system can provide some benefits, such as the reduction of vibration movement and impact effect. Additionally, the room between the CFRP floor panels and the raised systems can be filled with anti-fire foams, to improve fire performance.

The CFRP panel, with an open channel, enables the installation of the suspended ceiling system, shown in Fig. 4-3, which integrates the lighting, sprinkler and ventilation systems. It is proposed that the ceiling tiles be made from mineral fibres or fire-rated wood panels so that an acceptable level of fire standards/ratings is met. These tiles can also provide additional resistance to satisfy the "time rating" required for various fire safety engineering codes and to improve the fire performance of buildings with CFRP floor panels.

4.3 Design load of proposed floor panel

The design of the FRP floor panel was supported by finite element analysis. The self-weight of the floor panel, raised floor, services (heating, ventilation and cable trunk) and suspended ceiling were considered cumulatively a dead load. The area of cross-section (0.0135m$^2$) of the CFRP panel was measured using the designing tool, AutoCAD. The volume (0.081m$^3$) of the panel was calculated by multiplying the cross-section by the span 6m. The CFRP density (1502Kg/m$^3$) was taken from the data base of composites supplied by Gurit Ltd (www.gurit.com/guide-to-composites.aspx). Thus, the surface load from the panel’s self-weight was calculated as 0.397kN/m$^2$. Apart from the panel’s self-weight, the total dead load
accounted for the weight from the raised floor (0.361kN/m²), which was taken from previous work (Ready Homes Enterprises, 2011), services (including heating, ventilation and cable trunk) (0.3kN/m²) (Brooker, 2006) and the suspended ceiling (0.192kN/m²) (KNAUFDRYWALL, 2009). In total, the dead load was 1.25kN/m².

In terms of Eurocodes and references (Brooker, 2006), the total live load was taken as 6.0kN/m², including the imposed load of 5.0kN/m² and the partitions of 1.0kN/m². Considering the ultimate limit state, the partial safety factors, $\gamma_G = 1.35$ and $\gamma_Q = 1.50$, were used in the calculation of the statistically applied load, which was computed as 0.0106N/mm². Considering serviceability limit states, the design load was calculated as an un-factored load of 0.0075 N/mm².

### 4.4 Numerical modeling analysis

#### 4.4.1 Establishment of modeling

A finite element model was developed using a commercial software package, LUSAS (LUASA, 2011), to carry out the design check of the proposed CFRP floor panel. Considering the symmetry of the designed CFRP panel, a half model relating to the longitudinal direction was generated.

![LUSAS element geometry 3-D Solid Continuum Element--HX8M (left) and PN6 (right) (LUASA, 2011)](image)

This is shown in Fig. 4-4. It consists of 90,000 solid elements in total; the solid element, HX8M multi-physical 8-node 3-D element (Fig. 4-4 left), was used in the body of the panel, and PN6 multi-physical 6-node 3-D element (Fig. 4-4 right) was used in the sharp, geometrical area for simulating adhesives. Two different types of material models, isotropic and orthotropic, were used in the structural analysis for the modeling of the adhesives and
unidirectional, carbon fibre composite panel. A uniformly distributed load, obtained from section 4.3, was applied on the top surface of the panel. A fixed condition was applied at the end of the panel and over the supporting area. A symmetric condition was applied on the middle cross-section of the half model.

4.4.2 Geometrical simplifications

The floor panel was designed to have a constant cross-section along the length and is expected to be loaded symmetrically. Thus, only half of the length’s body was included in the model. Considering the worst case scenario, the end section and middle section of panel will be subjected to higher stresses. The FEA model is, therefore, half of the panel, which is believed to be higher in mesh refinement and CPU time.

4.4.3 Boundary Considerations

4.4.3.1 Mid-span

As described in section 4.4.2, the mid-span was given as a symmetrical boundary condition, meaning that translation is allowed in the vertical and transversal directions, not in the longitudinal direction; there is no rotation allowed about the transversal or z-axes.

4.4.3.2 End supports

The designed panel is proposed to be continuously supported by steel or concrete beams across its width. The adhesive will also be applied on the contact area between the panel and the steel beams. The connection was assumed to be perfectly bonded on the beam. Therefore, there are no movements in the “X, Y, Z” directions in the bonded area. Thus, the boundary condition of the supporting area in the FEA model is fully fixed. Because of solid elements used in the modeling, it is not necessary to add the rotational restraints in the supporting area.

4.4.4 Material Properties

The properties of CFRP (WE91 HSC 100) are provided by Composite Company Gurit (2001). The WE91 HSC is unidirectional, very strong and stiff in fibre direction. The WE91 HSC is a type of prepeg, which is very thin, with 0.1mm thickness. The component consists of multiple
layers, adding up to meet the designed thickness. The material is orthotropic, meaning it has two directional mechanic properties. One is in fibre direction (X direction), and the other is in transverse direction and through thickness (same properties at the Y and Z axes). Table 4.1 presents the properties of material WE91 HSC. It can be seen that the stiffness in fibre direction is about 18 times larger than that in transverse direction. The tensile and compressive strength in the fibre direction are around 30 times and 9 times the strength of the transverse direction, respectively.

Table 4.1: Material Properties

<table>
<thead>
<tr>
<th>CFRP (WE91 HSC 100)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Elastic modulus</td>
</tr>
<tr>
<td>E11 130.33 GPa</td>
</tr>
<tr>
<td>E22 7.22 GPa</td>
</tr>
<tr>
<td>E33 7.22 GPa</td>
</tr>
<tr>
<td>Shear modulus</td>
</tr>
<tr>
<td>G12 4.23 GPa</td>
</tr>
<tr>
<td>G13 4.23 GPa</td>
</tr>
<tr>
<td>G23 3.59 GPa</td>
</tr>
<tr>
<td>Poisson ratio (XY)</td>
</tr>
<tr>
<td>ν12 0.34</td>
</tr>
<tr>
<td>Poisson ratio (XZ)</td>
</tr>
<tr>
<td>ν13 0.34</td>
</tr>
<tr>
<td>Poisson ratio (YZ)</td>
</tr>
<tr>
<td>ν23 0.02</td>
</tr>
<tr>
<td>Mass Density</td>
</tr>
<tr>
<td>ρ 1502 kg/m³</td>
</tr>
<tr>
<td>Longitudinal Tensile Strength</td>
</tr>
<tr>
<td>Ux(t) 1433.6 MPa</td>
</tr>
<tr>
<td>Longitudinal Compressive Strength</td>
</tr>
<tr>
<td>Ux(c) 984.2 MPa</td>
</tr>
<tr>
<td>Transverse Tensile Strength</td>
</tr>
<tr>
<td>Uy(t) 32.5 MPa</td>
</tr>
<tr>
<td>Transverse Compressive Strength</td>
</tr>
<tr>
<td>Uy(c) 108.3 MPa</td>
</tr>
<tr>
<td>Through Thickness Tensile Strength</td>
</tr>
<tr>
<td>Uy(t) 32.5 MPa</td>
</tr>
<tr>
<td>Through Thickness Compressive Strength</td>
</tr>
<tr>
<td>Uy(c) 108.3 MPa</td>
</tr>
<tr>
<td>In-plan shear strength</td>
</tr>
<tr>
<td>Uxy 72.5 MPa</td>
</tr>
</tbody>
</table>

4.4.5 Meshing

A two-dimensional cross-section was created by AutoCAD and saved in the dfx file format, which was used by LUSAS for importing into a 2-D model. The model was first meshed using the line mesh unsolved elements. The top face and webs were meshed using linear elements. In order to reduce the total number of elements used, the regular elements were meshed using coarse meshing. A maximum edge length of 8.5mm was permitted for the top face; 4mm was permitted for the web face sheet. Once the 2-D cross-section was completed,
volume sweepings were used to transfer this pattern into the third dimension in the model. Two volumes were generated; one from the edges to the middle of the panel and another for the bonding portion. In total, 83,860 elements, incorporating 106,772 nodes, were used in the solid panel modeling, as shown in Fig. 4-5. Since the designed FRP slab is symmetrical in terms of its geometry and loading conditions, a half geometry of the actual model, cut longitudinally, was generated. This FEA model was run as an elastic linear analysis to analyse basic mechanical behaviour regarding deflection and to fulfil the strength check required by design guidelines.

![Figure 4-5: A FEA model of half floor panel](image)

### 4.4.6 Results and design check

#### 4.4.6.1 Deflection check

The deflection criteria given by design guideline is $L/250 = 24$mm. The maximum deflection calculated by FEA under un-factored design load is 7.90mm given in Table 4.11. Therefore, there is no doubt deflection check was passed. Deflection contour of the floor panel is shown in Fig. 4-6.

![Figure 4-6: Predicted deflection in a half model](image)
4.4.6.2 Hashin criteria

Hashin failure criteria were originally developed for strength check of unidirectional polymeric composites, and hence, its applications to other type of laminates and non-polymeric composites are approximated. Failure indices for Hashin criteria are related to fibre and matrix failures, and they involve four failure modes. The criteria are extended to three dimensional problems, in which the criteria for both tension and compression are considered.

Longitudinal tension

\[
\left(\frac{\sigma_{11}}{X_T}\right)^2 + \frac{\sigma_{12}^2 + \sigma_{13}^2}{S_{12}^2} = 1 \quad (4.1)
\]

Longitudinal compression

\[
\left(\frac{\sigma_{11}}{X_C}\right)^2 = 1 \quad (4.2)
\]

Transverse tension

\[
\left(\frac{\sigma_{22} + \sigma_{33}}{Y_T}\right)^2 + \frac{\sigma_{23}^2 - \sigma_{22}\sigma_{33}}{S_{23}^2} + \frac{\sigma_{12}^2 + \sigma_{13}^2}{S_{12}^2} = 1 \quad (4.3)
\]

Transverse compression

\[
\left(\frac{Y}{2S_{23}}\right)^2 - 1 + \left(\frac{\sigma_{22} + \sigma_{33}}{4S_{23}^2}\right) + \left(\frac{\sigma_{22} + \sigma_{33}}{4S_{23}^2}\right) + \frac{\sigma_{23}^2 - \sigma_{22}\sigma_{33}}{S_{23}^2} + \frac{\sigma_{12}^2 + \sigma_{13}^2}{S_{12}^2} = 1 \quad (4.4)
\]

Through thickness tension

\[
\left(\frac{\sigma_{33}}{Z_T}\right)^2 = 1 \quad (4.5)
\]

Through thickness compression

\[
\left(\frac{\sigma_{33}}{Z_C}\right)^2 = 1 \quad (4.6)
\]
It should be noted that \( \sigma_{ij} \) refers to a stress component in any direction and that \( i,j=1,2,3 \) (or \( X,Y,Z \)). When \( i=j \), \( \sigma_{ij} \) is normal stress, otherwise it is shear stress. The tensile and compressive allowable strengths for lamina are denoted by subscripts \( T \) and \( C \), respectively. \( X_T, Y_T, Z_T \) denote the allowable tensile strengths in three respective material directions. Similarly, \( X_C, Y_C, Z_C \) denote the allowable compressive strengths in three respective material directions. Further, \( S_{12}, S_{13} \) and \( S_{23} \) denote allowable shear strengths in the respective principal material directions (Hashin, 1980).

4.4.6.3 Effects of varied mesh densities in the model of the floor panel on critical stresses

Variation of maximum stresses reflecting the effects of different mesh densities in span, height and width were investigated to consider the convergent problems in FEM analysis. Tables 4.2 through 4.4 present all results of these investigations.

Table 4.2: Effects of varied mesh densities in span on the maximum stresses

<table>
<thead>
<tr>
<th>Stress component</th>
<th>20</th>
<th>40</th>
<th>60</th>
<th>80</th>
</tr>
</thead>
<tbody>
<tr>
<td>( \sigma_{xt} )</td>
<td>24.6</td>
<td>28.8</td>
<td>31.5</td>
<td>32.7</td>
</tr>
<tr>
<td>( \sigma_{xc} )</td>
<td>-90.5</td>
<td>-111.7</td>
<td>-124.3</td>
<td>-133.1</td>
</tr>
<tr>
<td>( \sigma_{yt} )</td>
<td>25.4</td>
<td>25.2</td>
<td>25.1</td>
<td>25.1</td>
</tr>
<tr>
<td>( \sigma_{yc} )</td>
<td>-13.4</td>
<td>-16.8</td>
<td>-18.2</td>
<td>-18.9</td>
</tr>
<tr>
<td>( \sigma_{zt} )</td>
<td>22.2</td>
<td>22.3</td>
<td>22.3</td>
<td>22.3</td>
</tr>
<tr>
<td>( \sigma_{zc} )</td>
<td>-16.2</td>
<td>-20.6</td>
<td>-22.9</td>
<td>-24.4</td>
</tr>
<tr>
<td>( \sigma_{xy} )</td>
<td>8.8</td>
<td>8.5</td>
<td>8.4</td>
<td>8.4</td>
</tr>
<tr>
<td>( \sigma_{yz} )</td>
<td>-8.8</td>
<td>-8.7</td>
<td>-8.7</td>
<td>-8.7</td>
</tr>
<tr>
<td>( \sigma_{zx} )</td>
<td>-9.3</td>
<td>-11.4</td>
<td>-13</td>
<td>-14.3</td>
</tr>
</tbody>
</table>

Table 4.2 shows the investigation on maximum stresses varied with different mesh densities along the span. The mesh density increased by presentation of number of divisions along the span from 20 to 80. There are nine total stress components investigated; \( \sigma_{xt} \) and \( \sigma_{xc} \) are the maximum tensile and compressive stress in \( x \) direction, \( \sigma_{yc} \) and \( \sigma_{yt} \) are the maximum tensile and compressive stress in \( y \) direction, \( \sigma_{zt} \) and \( \sigma_{zc} \) are the maximum tensile and compressive stress in \( z \) direction. \( \sigma_{xy}, \sigma_{xz}, \sigma_{yz} \) are the three maximum shear stresses in three orthogonal planes, respectively. It can be seen from Table 4.2 that most maximum stresses converged very well. The stresses in transverse and through-thickness direction, \( \sigma_{yt} \) and \( \sigma_{zt} \), were perfectly convergent in this investigation. It seems some stresses, \( \sigma_{xt} \) and \( \sigma_{xc} \), had low
convergence and need more divisions in span. This investigation stopped at division 80 due to greatly increased CPU time. Actually, the maximum values of stresses $\sigma_{xt}$, $\sigma_{xc}$, $\sigma_{yc}$, $\sigma_{zc}$ and $\sigma_{zx}$ are much less than their allowable values given in Table 4.1. The most critical stresses in this investigation are $\sigma_{yt}$ and $\sigma_{zt}$ because their allowable values are much lower than the others. All maximum stresses given by the final mesh density were used in the strength check of the design.

Table 4.3: Effects of varied mesh densities in height on the maximum stresses

<table>
<thead>
<tr>
<th>Stress component</th>
<th>32</th>
<th>40</th>
<th>48</th>
<th>56</th>
<th>64</th>
<th>72</th>
</tr>
</thead>
<tbody>
<tr>
<td>$\sigma_{xt}$</td>
<td>31.5</td>
<td>31.5</td>
<td>31.5</td>
<td>31.5</td>
<td>31.5</td>
<td>31.5</td>
</tr>
<tr>
<td>$\sigma_{xc}$</td>
<td>-124.4</td>
<td>-124.5</td>
<td>-124.5</td>
<td>-124.5</td>
<td>-124.5</td>
<td>-124.5</td>
</tr>
<tr>
<td>$\sigma_{yt}$</td>
<td>26.1</td>
<td>26.8</td>
<td>27.4</td>
<td>27.8</td>
<td>28.1</td>
<td>28.4</td>
</tr>
<tr>
<td>$\sigma_{yc}$</td>
<td>-19.4</td>
<td>-20.3</td>
<td>-21</td>
<td>-21.5</td>
<td>-21.9</td>
<td>-22.3</td>
</tr>
<tr>
<td>$\sigma_{zt}$</td>
<td>22.3</td>
<td>22.3</td>
<td>22.3</td>
<td>22.3</td>
<td>22.3</td>
<td>22.3</td>
</tr>
<tr>
<td>$\sigma_{zc}$</td>
<td>-23</td>
<td>-23</td>
<td>-23</td>
<td>-23</td>
<td>-23</td>
<td>-23</td>
</tr>
<tr>
<td>$\sigma_{xy}$</td>
<td>8.5</td>
<td>8.5</td>
<td>8.5</td>
<td>8.4</td>
<td>8.4</td>
<td>8.4</td>
</tr>
<tr>
<td>$\sigma_{yz}$</td>
<td>-8.6</td>
<td>-8.6</td>
<td>-8.5</td>
<td>-8.5</td>
<td>-8.6</td>
<td>-8.6</td>
</tr>
<tr>
<td>$\sigma_{zx}$</td>
<td>-13</td>
<td>-13</td>
<td>-13</td>
<td>-13</td>
<td>-13</td>
<td>-13</td>
</tr>
</tbody>
</table>

Table 4.3 presents the investigation of effects of varied mesh densities in height on the maximum stresses. Mesh density varied by number of divisions in height from 32 to 72. It can be seen from Table 4.3 that all stresses converged very well in this investigation. This means that the prediction given by the FEA model was rather stable when the number of divisions in height was increased. Similar to the investigation with mesh density in span, all maximum stresses given by the final mesh density were used in the strength check to support the design.

Table 4.4: Effects of varied mesh densities in width on the maximum stresses

<table>
<thead>
<tr>
<th>Stress component</th>
<th>44</th>
<th>66</th>
<th>88</th>
<th>110</th>
<th>132</th>
<th>154</th>
</tr>
</thead>
<tbody>
<tr>
<td>$\sigma_{xt}$</td>
<td>31.4</td>
<td>31.5</td>
<td>31.5</td>
<td>31.5</td>
<td>31.5</td>
<td>31.5</td>
</tr>
<tr>
<td>$\sigma_{xc}$</td>
<td>-124.3</td>
<td>-124.3</td>
<td>-124.3</td>
<td>124.3</td>
<td>124.3</td>
<td>124.3</td>
</tr>
<tr>
<td>$\sigma_{yt}$</td>
<td>25.1</td>
<td>25.1</td>
<td>25.1</td>
<td>25.1</td>
<td>25.1</td>
<td>25.1</td>
</tr>
<tr>
<td>$\sigma_{yc}$</td>
<td>-18.2</td>
<td>-18.2</td>
<td>-18.2</td>
<td>-18.2</td>
<td>-18.2</td>
<td>-18.2</td>
</tr>
<tr>
<td>$\sigma_{zt}$</td>
<td>22.3</td>
<td>22.3</td>
<td>22.3</td>
<td>22.3</td>
<td>22.3</td>
<td>22.3</td>
</tr>
<tr>
<td>$\sigma_{zc}$</td>
<td>-22.9</td>
<td>-22.9</td>
<td>-22.9</td>
<td>-22.9</td>
<td>-22.9</td>
<td>-22.9</td>
</tr>
<tr>
<td>$\sigma_{xy}$</td>
<td>8.4</td>
<td>8.4</td>
<td>8.4</td>
<td>8.4</td>
<td>8.4</td>
<td>8.4</td>
</tr>
<tr>
<td>$\sigma_{yz}$</td>
<td>-8.7</td>
<td>-8.7</td>
<td>-8.7</td>
<td>-8.7</td>
<td>-8.7</td>
<td>-8.7</td>
</tr>
<tr>
<td>$\sigma_{zx}$</td>
<td>-13</td>
<td>-13</td>
<td>-13</td>
<td>-13</td>
<td>-13</td>
<td>-13</td>
</tr>
</tbody>
</table>
Table 4.4 shows the results from the analysis of varied mesh densities in width. The number of divisions in width varied from 44 to 154. It can be seen from Table 4.4 that all stresses were quickly convergent. It means that variation of mesh density has no significant effects on stress analysis. There is no doubt that each maximum stress given by final mesh density was used in the strength check of the design.

4.4.6.4 Effects of varied mesh densities on Hashin criteria indicators

The effects of varied mesh densities in span, height and width on the Hashin failure criteria check are investigated in this section. According to the FEA modeling analysis in last section, the highest stresses for $\sigma_{yc}, \sigma_{yt}, \sigma_{xt}, \sigma_{xc}, \sigma_{zt}, \sigma_{yz}, \sigma_{zx}, \sigma_{xy}$ were taken at the bottom corner of the end of the panel. It can be seen from Table 4.1 that the calculated values of $\sigma_{zt}$ and $\sigma_{yt}$ are closer to the individual material strength than other stress components. Tables 4.5 through 4.7 present the highest tensile, compressive and shear stresses in the fibre, matrix and through-thickness directions considering varied mesh densities in span, height and width, respectively. It can be seen from these tables that the most critical stresses are the tensile stresses in the transverse matrix and through-thickness directions. Therefore, these two stresses played an important role in this investigation. Hashin failure criteria have six equations to present the criteria in three material directions, each material has tensile and compressive criterion respectively. Once the highest value of stress in a specific direction was selected for application in the corresponding criterion equation, all other values of stress components were chosen from the same point as that in which the highest value of stress in a specific direction was taken from.

Table 4.5: Hashin criteria check with varied mesh densities in span

<table>
<thead>
<tr>
<th>Mesh Density in span</th>
<th>20</th>
<th>40</th>
<th>60</th>
<th>80</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Stresses at the lower left corner of the end section for fibre tensile failure check</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>$\sigma_{xt}$</td>
<td>24.6</td>
<td>28.8</td>
<td>31.5</td>
<td>32.7</td>
</tr>
<tr>
<td>$\sigma_{xy}$</td>
<td>-0.8</td>
<td>-0.6</td>
<td>-0.8</td>
<td>-0.9</td>
</tr>
<tr>
<td>$\sigma_{zx}$</td>
<td>4.4</td>
<td>1.1</td>
<td>1.4</td>
<td>1.5</td>
</tr>
<tr>
<td>Stresses at the lower left corner of the end section for fibre compressive failure check</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>$\sigma_{xc}$</td>
<td>-90.5</td>
<td>111.7</td>
<td>124.3</td>
<td>133.1</td>
</tr>
<tr>
<td>Stresses at the lower left corner of the end section for through-thickness tensile failure check</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
### Stresses at the lower left corner of the end section for through-thickness compressive failure check

<table>
<thead>
<tr>
<th>( \sigma_{zt} )</th>
<th>22.2</th>
<th>22.3</th>
<th>22.3</th>
<th>22.3</th>
</tr>
</thead>
</table>

### Stresses at the lower left corner of the end section for transverse matrix tensile failure check

<table>
<thead>
<tr>
<th>( \sigma_{yt} )</th>
<th>25.4</th>
<th>25.2</th>
<th>25.1</th>
<th>25.1</th>
</tr>
</thead>
<tbody>
<tr>
<td>( \sigma_{zt} )</td>
<td>0.1</td>
<td>0.1</td>
<td>0.1</td>
<td>0.1</td>
</tr>
<tr>
<td>( \sigma_{xy} )</td>
<td>1.2</td>
<td>1.3</td>
<td>1.3</td>
<td>1.3</td>
</tr>
<tr>
<td>( \sigma_{yz} )</td>
<td>0.7</td>
<td>0.7</td>
<td>0.7</td>
<td>0.7</td>
</tr>
<tr>
<td>( \sigma_{zx} )</td>
<td>0.1</td>
<td>0.1</td>
<td>0.1</td>
<td>0.1</td>
</tr>
</tbody>
</table>

### Stresses at the lower left corner of the end section for transverse matrix compressive failure check

<table>
<thead>
<tr>
<th>( \sigma_{yc} )</th>
<th>-13.4</th>
<th>-16.8</th>
<th>-18.2</th>
<th>-18.9</th>
</tr>
</thead>
<tbody>
<tr>
<td>( \sigma_{zc} )</td>
<td>-0.4</td>
<td>-0.5</td>
<td>-0.5</td>
<td>-0.5</td>
</tr>
<tr>
<td>( \sigma_{xy} )</td>
<td>-1</td>
<td>-2.2</td>
<td>-2.8</td>
<td>-3.1</td>
</tr>
<tr>
<td>( \sigma_{yz} )</td>
<td>-0.5</td>
<td>-0.6</td>
<td>-0.7</td>
<td>-0.7</td>
</tr>
<tr>
<td>( \sigma_{zx} )</td>
<td>-0.1</td>
<td>-0.1</td>
<td>-0.1</td>
<td>-0.1</td>
</tr>
</tbody>
</table>

### Hashin Criteria Check (failure indicator: 1)

<table>
<thead>
<tr>
<th>Fibre tensile failure indicator</th>
<th>0.004</th>
<th>0.001</th>
<th>0.001</th>
<th>0.001</th>
</tr>
</thead>
<tbody>
<tr>
<td>Fibre compressive failure indicator</td>
<td>0.01</td>
<td>0.01</td>
<td>0.02</td>
<td>0.02</td>
</tr>
<tr>
<td>Through-thickness tensile failure indicator</td>
<td>0.47</td>
<td>0.47</td>
<td>0.47</td>
<td>0.47</td>
</tr>
<tr>
<td>Through-thickness compressive failure indicator</td>
<td>0.02</td>
<td>0.04</td>
<td>0.04</td>
<td>0.05</td>
</tr>
<tr>
<td>Transverse matrix tensile failure indicator</td>
<td>0.48</td>
<td>0.49</td>
<td>0.49</td>
<td>0.49</td>
</tr>
<tr>
<td>Transverse matrix compressive failure indicator</td>
<td>0.09</td>
<td>0.12</td>
<td>0.15</td>
<td>0.16</td>
</tr>
</tbody>
</table>

Table 4.5 shows the results of the Hashin failure criteria check considering varied mesh densities in span. The mesh density varied from 20 to 80 along the span. The Hashin criteria were used to check the failure in fibre tension/compression, transverse matrix tension/compression and through-thickness tension/compression, respectively. According to the results from Table 4.5, failure indicators for fibre tension/compression are very small because of high tensile/compressive strength in fibres. The failure indicators for tension/compression in matrix and through-thickness directions are considerable values in strength check. These indicators implied that transverse matrix and through-thickness directions are the weak aspects and need much more attention in strength check. The biggest indicator is the matrix tensile check, with almost 0.49 given varied mesh densities. There was an exception of 0.48 yielded from mesh density 20. In through-thickness tensile failure check, the indicator of 0.47 was a constant value resulting from all mesh densities. The rest of the
indicators are very small, which means the possibility of failure in all other directions was very low.

Table 4.6: Hashin criteria check with varied mesh densities in height

<table>
<thead>
<tr>
<th>Mesh Density in height</th>
<th>32</th>
<th>40</th>
<th>48</th>
<th>56</th>
<th>64</th>
<th>72</th>
</tr>
</thead>
<tbody>
<tr>
<td>Stresses at the lower left corner of the end section for fibre tensile failure check</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>$\sigma_{xt}$</td>
<td>31.5</td>
<td>31.5</td>
<td>31.5</td>
<td>31.5</td>
<td>31.5</td>
<td>31.5</td>
</tr>
<tr>
<td>$\sigma_{xy}$</td>
<td>-0.8</td>
<td>-0.8</td>
<td>-0.8</td>
<td>-0.8</td>
<td>-0.8</td>
<td>-0.8</td>
</tr>
<tr>
<td>$\sigma_{zx}$</td>
<td>1.4</td>
<td>1.4</td>
<td>1.4</td>
<td>1.4</td>
<td>1.4</td>
<td>1.4</td>
</tr>
<tr>
<td>Stresses at the lower left corner of the end section for fibre compressive failure check</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>$\sigma_{xc}$</td>
<td>124.4</td>
<td>124.5</td>
<td>124.5</td>
<td>124.5</td>
<td>124.5</td>
<td>124.5</td>
</tr>
<tr>
<td>Stresses at the lower left corner of the end section for through-thickness tensile failure check</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>$\sigma_{zt}$</td>
<td>22.3</td>
<td>22.3</td>
<td>22.3</td>
<td>22.3</td>
<td>22.3</td>
<td>22.3</td>
</tr>
<tr>
<td>Stresses at the lower left corner of the end section for through-thickness compressive failure check</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>$\sigma_{zc}$</td>
<td>-23</td>
<td>-23</td>
<td>-23</td>
<td>-23</td>
<td>-23</td>
<td>-23</td>
</tr>
<tr>
<td>Stresses at the lower left corner of the end section for transverse matrix tensile failure check</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>$\sigma_{yt}$</td>
<td>26.1</td>
<td>26.8</td>
<td>27.4</td>
<td>27.8</td>
<td>28.1</td>
<td>28.4</td>
</tr>
<tr>
<td>$\sigma_{zt}$</td>
<td>0.1</td>
<td>0.2</td>
<td>0.2</td>
<td>0.3</td>
<td>0.4</td>
<td>0.4</td>
</tr>
<tr>
<td>$\sigma_{xy}$</td>
<td>1.4</td>
<td>1.5</td>
<td>1.5</td>
<td>1.6</td>
<td>1.6</td>
<td>1.6</td>
</tr>
<tr>
<td>$\sigma_{yz}$</td>
<td>0.8</td>
<td>0.8</td>
<td>0.9</td>
<td>0.9</td>
<td>0.9</td>
<td>0.9</td>
</tr>
<tr>
<td>$\sigma_{zx}$</td>
<td>0.1</td>
<td>0.1</td>
<td>0.1</td>
<td>0.1</td>
<td>1.0</td>
<td>1.0</td>
</tr>
<tr>
<td>Stresses at the lower left corner of the end section for transverse matrix compressive failure check</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>$\sigma_{yc}$</td>
<td>26.1</td>
<td>26.8</td>
<td>27.4</td>
<td>27.8</td>
<td>28.1</td>
<td>28.4</td>
</tr>
<tr>
<td>$\sigma_{zc}$</td>
<td>0.1</td>
<td>0.2</td>
<td>0.2</td>
<td>0.3</td>
<td>0.4</td>
<td>0.4</td>
</tr>
<tr>
<td>$\sigma_{xy}$</td>
<td>1.4</td>
<td>1.5</td>
<td>1.5</td>
<td>1.6</td>
<td>1.6</td>
<td>1.6</td>
</tr>
<tr>
<td>$\sigma_{yz}$</td>
<td>0.8</td>
<td>0.8</td>
<td>0.9</td>
<td>0.9</td>
<td>0.9</td>
<td>0.9</td>
</tr>
<tr>
<td>$\sigma_{zx}$</td>
<td>0.1</td>
<td>0.1</td>
<td>0.1</td>
<td>0.1</td>
<td>1.0</td>
<td>1.0</td>
</tr>
</tbody>
</table>

Hashin Criteria Check (failure indicator: 1)

<table>
<thead>
<tr>
<th></th>
<th>0.001</th>
<th>0.001</th>
<th>0.001</th>
<th>0.001</th>
<th>0.001</th>
<th>0.001</th>
</tr>
</thead>
<tbody>
<tr>
<td>Fibre tensile failure indicator</td>
<td>0.02</td>
<td>0.02</td>
<td>0.02</td>
<td>0.02</td>
<td>0.02</td>
<td>0.02</td>
</tr>
<tr>
<td>Through-thickness tensile failure indicator</td>
<td>0.47</td>
<td>0.47</td>
<td>0.47</td>
<td>0.47</td>
<td>0.47</td>
<td>0.47</td>
</tr>
<tr>
<td>Through-thickness compressive failure indicator</td>
<td>0.45</td>
<td>0.45</td>
<td>0.45</td>
<td>0.45</td>
<td>0.45</td>
<td>0.45</td>
</tr>
<tr>
<td>Transverse matrix tensile failure indicator</td>
<td>0.65</td>
<td>0.69</td>
<td>0.72</td>
<td>0.75</td>
<td>0.77</td>
<td>0.78</td>
</tr>
<tr>
<td>Transverse matrix compressive failure indicator</td>
<td>0.10</td>
<td>0.10</td>
<td>0.11</td>
<td>0.11</td>
<td>0.11</td>
<td>0.12</td>
</tr>
</tbody>
</table>
Table 4.6 shows the results of the Hashin failure criteria check considering varied mesh densities in height. The mesh density varied from 32 to 72 along the height. Similar to Table 4.5, relatively bigger values of failure indicators were given in the transverse and through-thickness directions. Failure indicators in transverse matrix direction varied from 0.65 to 0.78 as mesh density changed from 32 to 72. The value of these indicators was convergent as mesh densities increased. Bigger values of indicators were also given for through-thickness tension/compression check. The indicators stayed constant at 0.47 and 0.45 for the tensile and compressive case, respectively, regardless of what mesh density was. All others indicators were very small. This again indicated that more attention should be paid to the transverse and through-thickness directions in strength check.

Table 4.7: Hashin criteria check with varied mesh densities in width

<table>
<thead>
<tr>
<th>Mesh Density in width</th>
<th>44</th>
<th>66</th>
<th>88</th>
<th>110</th>
<th>132</th>
<th>154</th>
</tr>
</thead>
<tbody>
<tr>
<td>Stresses at the lower left corner of the end section for fibre tensile failure check</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>$\sigma_{xt}$</td>
<td>32.1</td>
<td>31.5</td>
<td>31.5</td>
<td>31.5</td>
<td>31.5</td>
<td>31.5</td>
</tr>
<tr>
<td>$\sigma_{xy}$</td>
<td>0.4</td>
<td>0.8</td>
<td>0.8</td>
<td>0.8</td>
<td>0.8</td>
<td>0.8</td>
</tr>
<tr>
<td>$\sigma_{zx}$</td>
<td>3.6</td>
<td>1.4</td>
<td>1.4</td>
<td>1.4</td>
<td>1.4</td>
<td>1.4</td>
</tr>
<tr>
<td>Stresses at the lower left corner of the end section for fibre compressive failure check</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>$\sigma_{xc}$</td>
<td>-116.7</td>
<td>-124.3</td>
<td>-124.3</td>
<td>124.3</td>
<td>124.3</td>
<td>124.3</td>
</tr>
<tr>
<td>Stresses at the lower left corner of the end section for through-thickness tensile failure check</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>$\sigma_{zt}$</td>
<td>22.3</td>
<td>22.3</td>
<td>22.3</td>
<td>22.3</td>
<td>22.3</td>
<td>22.3</td>
</tr>
<tr>
<td>Stresses at the lower left corner of the end section for through-thickness compressive failure check</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>$\sigma_{zc}$</td>
<td>-22.9</td>
<td>-22.9</td>
<td>-22.9</td>
<td>-22.9</td>
<td>-22.9</td>
<td>-22.9</td>
</tr>
<tr>
<td>Stresses at the lower left corner of the end section for transverse matrix tensile failure check</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>$\sigma_{yt}$</td>
<td>24.8</td>
<td>25.1</td>
<td>25.1</td>
<td>25.1</td>
<td>25.1</td>
<td>25.1</td>
</tr>
<tr>
<td>$\sigma_{zt}$</td>
<td>0.5</td>
<td>0.1</td>
<td>0.1</td>
<td>0.1</td>
<td>0.1</td>
<td>0.1</td>
</tr>
<tr>
<td>$\sigma_{xy}$</td>
<td>1.4</td>
<td>1.3</td>
<td>1.3</td>
<td>1.3</td>
<td>1.3</td>
<td>1.3</td>
</tr>
<tr>
<td>$\sigma_{yz}$</td>
<td>0.7</td>
<td>0.7</td>
<td>0.7</td>
<td>0.7</td>
<td>0.7</td>
<td>0.7</td>
</tr>
<tr>
<td>$\sigma_{zx}$</td>
<td>0.1</td>
<td>0.1</td>
<td>0.1</td>
<td>0.1</td>
<td>0.1</td>
<td>0.1</td>
</tr>
<tr>
<td>Stresses at the lower left corner of the end section for transverse matrix compressive failure check</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>$\sigma_{yc}$</td>
<td>-17.7</td>
<td>-18.2</td>
<td>-18.2</td>
<td>-18.2</td>
<td>-18.2</td>
<td>-18.2</td>
</tr>
<tr>
<td>$\sigma_{zc}$</td>
<td>-0.9</td>
<td>-0.5</td>
<td>-0.5</td>
<td>-0.5</td>
<td>-0.5</td>
<td>-0.5</td>
</tr>
<tr>
<td>$\sigma_{xy}$</td>
<td>-2.5</td>
<td>-2.8</td>
<td>-2.8</td>
<td>-2.8</td>
<td>-2.8</td>
<td>-2.8</td>
</tr>
<tr>
<td>$\sigma_{yz}$</td>
<td>-0.7</td>
<td>-0.7</td>
<td>-0.7</td>
<td>-0.7</td>
<td>-0.7</td>
<td>-0.7</td>
</tr>
<tr>
<td>$\sigma_{zx}$</td>
<td>0.1</td>
<td>0.1</td>
<td>0.1</td>
<td>0.1</td>
<td>0.1</td>
<td>0.1</td>
</tr>
</tbody>
</table>
Table 4.7 shows the results of the Hashin failure criteria check considering varied mesh densities in width. The mesh density varied from 44 to 154 along the width. All indicators were perfectly convergent through mesh density changes. Similar to Tables 4.5 and 4.6, relatively bigger values of failure indicators were again found in the transverse and through-thickness directions. The failure indicator 0.6 was given in transverse matrix tensile failure check. Another big indicator of 0.47 was obtained in through-thickness failure check. All of the rest of the indicators were very small.

The effects of varied mesh densities in span, width and height on Hashin criteria check were investigated. It can be seen from the investigations that most failure indicators were stable in different mesh densities. Some of them converged quickly when mesh densities increased. Two indicators in the transverse matrix and through-thickness directions were found to be especially important by all three investigations considering varied mesh densities in span, width and height of the investigated floor panel.

### 4.4.6.5 Hashin criteria check

Some summarised results from the strength criteria check, considering the mesh densities varied in three dimensions of the floor panel in the previous section are given in Table 4.8. The final FEA model, shown as Fig. 4-7, with mesh densities, 60 divisions in span, 72 divisions in height and 66 divisions in width, is recommended, because it supports the design that passed all checks with a large safety margin. It can be seen from Table 4.8 that calculated Hashin criteria indicators, 0.001 and 0.02, are for fibre failure check in tension and compression, respectively. The Hashin criteria indicators of 0.78 and 0.12 are for transverse
matrix tensile and compressive failure check, respectively. And the Hashin criteria indicators of 0.47 and 0.45 are for through-thickness tensile and compressive failure check, respectively. Although the final strength check was based on the biggest indicator, 0.78 in transverse matrix tensile direction, there is still a safety margin of 22%.

![Figure 4-7: A FEA model of the CFRP panel](image)

Table 4.8: Design check for deflection and strength

<table>
<thead>
<tr>
<th>Design criteria</th>
<th>Modeling results</th>
<th>State</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Deflection criterion</strong></td>
<td></td>
<td></td>
</tr>
<tr>
<td>L/250 = 24mm</td>
<td>Max Del. = 7.90mm</td>
<td>Pass</td>
</tr>
<tr>
<td><strong>Hashin criteria</strong></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Fibre tension</td>
<td>1</td>
<td>0.001</td>
</tr>
<tr>
<td>Fibre compression</td>
<td>1</td>
<td>0.02</td>
</tr>
<tr>
<td>Through-thickness tension</td>
<td>1</td>
<td>0.47</td>
</tr>
<tr>
<td>Through-thickness compression</td>
<td>1</td>
<td>0.45</td>
</tr>
<tr>
<td>Transverse matrix tension</td>
<td>1</td>
<td>0.78</td>
</tr>
<tr>
<td>Transverse matrix compression</td>
<td>1</td>
<td>0.12</td>
</tr>
</tbody>
</table>

4.4.6.6 Stress contour

The stress contour serves as an excellent visible approach for presenting the stress distribution. It can present stress distributions in fibre-direction, transverse matrix direction and through-thickness direction. Stress contour can be used to show the maximum stress values and the locations of stress. The dark red and blue colours in Figures 4-8 through 4-11 indicate the maximum tensile and compressive stresses, respectively.
4.4.6.7 Stress in fibre direction

Figure 4-8 shows stress contour in X (fibre) direction under the previously mentioned factored design load. Higher stresses are located at two crucial areas, the left web bonded area and the middle web at the end of the panel. At the left web, the maximum compressive stress of 124.55MPa and the maximum tensile stress of 31.55MPa are located at the lower left corner shown in Fig. 4-8. The maximum tensile stress is only about 2.5% of the CFPR tensile strength, and the maximum compression stress is about 13% of the CFRP compressive strength. Fig. 4-8 shows that the stresses in fibre direction are not critical stresses because the carbon fibre is very high in strength. These findings agree with those mentioned in Section 4.4.6.1. The Hashin criteria indicators are very small in both tension and compression in fibre direction.

![Figure 4-8: Stress σₓ distribution in fibre direction](image)

4.4.6.8 Stresses in transverse matrix direction

The location of the highest tensile stress in transverse direction, 28.41MPa shown in Fig. 4-9, is different from that in fibre direction, which is at the lower right corner of the end of the panel. The panel is adhesively bonded or mechanically clamped on the steel beam. Under bending, the end of panel is subjected to a big tearing force, which generates the maximum tensile stress at the fixed end. It can be seen from Fig. 4-9 that the maximum compressive stress of 28.4MPa and tensile stress of 22.2MPa are both located at the lower, outer corner of the end section of the panel.
4.4.6.9 Stresses in through-thickness direction

Fig. 4-10 shows the maximum tensile and compressive stress values to be 22.3MPa and 22.99MPa, respectively, in through-thickness direction. These values are closer to those in the transverse direction, however, their locations are different from those in the transverse direction mentioned in the last section. They are located on the left lower corner, bonded area at the end section of panel. It should be noted that the maximum tensile stresses in both transverse matrix and through-thickness directions are closer, and are under the material tensile strength of 32.5MPa, given in Table 4.1. The maximum compressive stress is only about one fifth of the compressive strength of 108.3MPa.
4.4.6.10  Shear stresses in XZ plane

According to the modeling analysis, the maximum shear stress was located in the XZ plane, which is the plane where bonded connection was designed. Fig. 4-11 illustrates the shear stress distribution. Maximum shear stress 13.0MPa is located at the lower left corner of the end of the panel. Comparing this to the material shear strength 71.9MPa, the maximum shear stress is only about 18% of the total shear strength.

Figure 4-11: Shear stress $\sigma_{xz}$ distribution in XZ plane

4.4.7  Discussion of critical design parameters

Table 4.8 provides select, important, computed results for the design check. The maximum deflection was 7.9mm, which is much less than the deflection criterion of 24mm (span/250 recommended by the Eurocodes). The deflection contour was given in Fig. 4-11. All maximum stresses, in different directions, are under the individual material strength and an ample strength safety margin was predicted. Regarding the strength check, attention should be given to the transverse and through-thickness directions because of the weak matrix. The maximum transverse tensile stress, 28.41MPa, is located at the lower right corner at the end of the panel, which can be seen in Fig. 4-9. The predicted maximum tensile stress of 22.3MPa in the through-thickness direction is quite close to that in the transverse direction and is located at the lower left corner of a side web, shown in Fig. 4-10. It should be noted that using the maximum applied stresses and the individual material strengths, the design check passed all Hashin failure criteria, as shown in Table 4.8.
4.4.8 Dimensional variation of floor panels

The design parameters were included in this investigation to consider the varied dimensions for span, width and height of the floor panel. These findings will be useful for the floor panel’s application in buildings with different sizes. The effect of bonding length on the prediction for the deflection and critical stresses in transverse and through-thickness directions was also investigated. Figures 4-12 through 4-15 present the FEA predicted maximum values for deflection, transverse matrix tensile stress, through-thickness tensile stress and XZ plane shear stresses against the varied spans, height, width and bonding length of the floor panel. Because these selected components are important and relatively sensitive in the design check, the deflection criteria and directional strength were individually considered. The investigation of varied dimensions was based on a basic model of the floor panel, with 6m span, 0.5m width and 0.13m height. Only one design parameter varied in each of the investigations presented below.

4.4.8.1 Span variation

This analysis considered the changes of the span, but all other parameters were kept the same as those in the basic floor model. The span varied from 4m to 8m, though there was no change in the width or the height. This investigation aimed to find out the effect of the variation of span on the structural behaviours of the CFRP panel. Fig. 4-12 illustrates the analysis of the panel with varied span. In this figure, the symbol DEF means deflection, DYT is transverse matrix tensile stress, DZT is through-thickness tensile stress and DXZ means the XZ plane shear stress.

It can be seen from Fig. 4-12 that the deflection, transverse matrix tensile stress, through-thickness tensile stress and shear stress were gradually increased when the span increased. The deflection DEF was smoothly increased by span increase. At 8m span, the maximum deflection reached 21.5mm, which is still under the critical value of deflection, 24mm. It can be seen from Fig. 4-12 that both DYT and DZT were affected by varied span significantly. DYT is bigger than DZT in all varied spans from 4m to 8m. It should be noted that the strength check with Hashin criteria failed when the panel had an 8m span, because the maximum tensile stress in the transverse and through-thickness directions at the end section of the panel exceeded the matrix tensile strength of 32.5MPa. If the floor span is expected to
be 8m, the height should be increased accordingly. The XZ plane shear stress \( DXZ \) was slightly increased when span increased.

4.4.8.2 Height variation

The range of height of the floor panel was investigated from 115mm to 150mm. The investigation of height variation aimed to discover how the panel performs with different heights. Simulated responses are presented by the four curves in Fig. 4-13.
It can be seen from Fig. 4-13 that all parameters, DEF, DYT, DZT and DXZ, went down smoothly when height increased as stiffness increased. DYT and DZT reduced quicker than DXY. DYT was bigger than the DZT in all varied heights between 115mm and 150mm. The shear stress DXZ was less affected by the height. Actually, about 5MPa difference can be seen within the range of height from 115mm to 150mm. The value of 120mm was determined to be the minimum height when the span and width were taken as 6m and 0.5m, respectively. When the height of the panel was less than 120mm, the strength check with Hashin criteria failed, because the computed stresses for DYT was over the tensile strength in transverse matrix.

4.4.8.3 Width variation

The variation of width was given from 300mm to 700mm. Fig. 4-14 shows the modeling results of panels with varied width. All parameters, DEF, DYT, DZT and DXZ, increased smoothly as width increased. Variations of DYT and DZT did not increase significantly compared to the case with varied span. The shear stress DXZ changed quite a lot, from 8MPa to 18MPa. The deflection change was between 5mm and 11mm. The deflection at the quarter of the top plate increased as width increased. However, the strength check failed by Hashin criteria when the width was over 700mm. The reason for the failure and the failed location was the same as in the two cases above.

<table>
<thead>
<tr>
<th>Deflection (mm)</th>
<th>Stress (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>35</td>
<td>35</td>
</tr>
<tr>
<td>30</td>
<td>30</td>
</tr>
<tr>
<td>25</td>
<td>25</td>
</tr>
<tr>
<td>20</td>
<td>20</td>
</tr>
<tr>
<td>15</td>
<td>15</td>
</tr>
<tr>
<td>10</td>
<td>10</td>
</tr>
<tr>
<td>5</td>
<td>5</td>
</tr>
</tbody>
</table>

![Figure 4-14: Analysis of panels with varied width](image-url)
4.4.8.4 Bonding length variation

Similar to other investigations on dimensional variation, Fig. 4-15 shows the responses, represented by four curves, to the variation in bonding length from 30mm to 190mm. The stress distribution in the bonding area is very complicated and is also very sensitive to the bonding thickness, bonding length, bonding area and the curing. However, the bonding length would affect the highest stress, because this stress is around the bonding area. Fig. 4-15 presents a significant difference from other figures. It can be seen from Fig. 4-15 that the four curves generally decreased as the bonding length increased. The deflection was not affected significantly by bonding length in this investigation. It seemed interesting that transverse matrix tensile stress DYT and through-thickness tensile stress DZT increased when the bonding length increased from 30mm to 75mm, then DZT slightly decreased after 75mm; DYT decrease after reach to 110mm. In all the responses, DZT dropped quicker than DYT. The predicted shear stress, DXY, decreased significantly when the bonding length increased, because the increased bonding length expanded the shearing areas. The shear stress DXY was very high when bonding length varied from 30mm to 65mm. Beyond that, DXY became stable. It should be noted that the shear stress, DXY, was always below the material shear strength.

![Figure 4-15: Analysis of panels with varied bonding length](image-url)
4.4.9 Design curves

The analysis of the proposed floor panel, with varied span, width and height, is necessary in the design of buildings with varied dimensions. This analysis was based on the basic panel model provided in a previous section. The span, width and height of the panel were changed individually at each investigation. From previous analysis, it was discovered that the major design criteria are the maximum deflection and crucial stresses in transverse and through-thickness directions. Therefore, the analyses in this section are based on the results of deflection and stresses in transverse and through-thickness direction against varied span, width and height, individually. It is clear from Fig. 4-12 and 4-15 that the changes of deflection against span and height have nonlinear features. However, deflection against varied width is approximately linear when the width is less than 70mm and the maximum deflection is located in the middle web of the middle section. Over 70mm of width, the position of the maximum deflection moved from the middle web to the quarter of the top plate at the middle section of panel because of transverse bending.

Minitab, a powerful statistics package, was employed for the data analysis to find out the relationship between the maximum deflection, crucial stresses in transverse/through-thickness directions, and crucial in-plane shear stress with variable span, width, unit height and bonding length. All the formulas generated by Minitab are presented by the power function as shown below.

Deflection - Span curve:  \( \delta_s = 0.0195(L)^{3.3581} \)  \( (4.7) \)
Deflection - Width curve:  \( \delta_w = 0.0415(W)^{0.8456} \)  \( (4.8) \)
Deflection - Height curve:  \( \delta_h = 98152(H)^{-1.936} \)  \( (4.9) \)
Deflection - Bonding length curve:  \( \delta_b = 11.447(B)^{-0.082} \)  \( (4.10) \)

Where, \( \delta_s, \delta_w, \delta_h \) and \( \delta_b \) are the deflection varied with length (L), width (W), height (H) and bonding length (B).

Transverse matrix stress - Span curve:  \( \sigma_{yts} = 0.1422(L)^{2.9509} \)  \( (4.11) \)
Transverse matrix stress - Width curve:  \( \sigma_{ytw} = 3.065(W)^{0.3559} \)  \( (4.12) \)
Transverse matrix stress - Height curve:  \( \sigma_{yth} = 977618(H)^{-2.147} \)  \( (4.13) \)
Transverse matrix stress - Bonding length curve:  \( \sigma_{ytb} = 18.732(B)^{-0.0757} \)  \( (4.14) \)

Where, \( \sigma_{yts}, \sigma_{ytw}, \sigma_{yth} \) and \( \sigma_{ytb} \) are the maximum transverse matrix stress varied with length (L), width (W), height (H) and bonding length (B).
Chapter-4: Proposed FRP Floor Panel System

Through-thickness stress - Span curve: \( \sigma_{zts} = 0.1415(L)^{2.8263} \) (4.15)

Through-thickness stress - Width curve: \( \sigma_{ztw} = 1.6261(W)^{0.4205} \) (4.16)

Through-thickness stress - Height curve: \( \sigma_{zth} = 69190(H)^{-1.649} \) (4.17)

Through-thickness stress - Bonding length curve: \( \sigma_{ztb} = 31.434(B)^{-0.079} \) (4.18)

Where, \( \sigma_{zts} \), \( \sigma_{ztw} \), \( \sigma_{zth} \) and \( \sigma_{ztb} \) are the maximum through-thickness stress varied with length (L), width (W), height (H) and bonding length (B).

In-plane shear stress - Span curve: \( \sigma_{xys} = 0.7772(L)^{1.5775} \) (4.19)

In-plane shear stress - Width curve: \( \sigma_{xyw} = 0.1064(W)^{0.856} \) (4.20)

In-plane shear stress - Height curve: \( \sigma_{xyh} = 529.07(H)^{-0.762} \) (4.21)

In-plane shear stress - Bonding length curve: \( \sigma_{xyb} = 129.66(B)^{-0.487} \) (4.22)

Where, \( \sigma_{xys} \), \( \sigma_{xyw} \), \( \sigma_{xyh} \) and \( \sigma_{xyb} \) are the maximum in-plane shear stress varied with length (L), width (W), height (H) and bonding length (B).

4.5 Buckling and free vibration analysis

4.5.1 Buckling analysis

Because the cross-section of the proposed CFRP panel is a thin-walled beam, it is necessary to investigate the buckling problem under design load as a pressure on the top plate of the panel. A shell model of CFRP panel was created to carry out a linear buckling analysis, consisting of 14,000 thick shell elements (QTS4). It should be noted that the value of design load, 0.0106 N/mm\(^2\), was applied as a pressure in buckling analysis. Table 4.9 shows the eigenvalues of the first eight buckling modes given by the FEA analysis.

<table>
<thead>
<tr>
<th>Buckling mode</th>
<th>1</th>
<th>2</th>
<th>3</th>
<th>4</th>
<th>5</th>
<th>6</th>
<th>7</th>
<th>8</th>
</tr>
</thead>
<tbody>
<tr>
<td>Eigenvalues</td>
<td>8.4</td>
<td>9.4</td>
<td>9.8</td>
<td>10.4</td>
<td>10.8</td>
<td>11.6</td>
<td>11.9</td>
<td>12.9</td>
</tr>
</tbody>
</table>

It can be seen from Table 4.9 that the first buckling load factor is 8.4 times that of the design load. The reason the CFRP panel has high buckling load factors is because the panel subjected to the bending instead of the axial load along the longitudinal direction. Therefore, the buckling problem of the designed CFRP panel can be ignored. Fig. 4-16 shows the first buckling mode, where mainly the local top plate has buckled along with slight buckling on
the web at the end of panel because the end section of panel is subjected to a local compression.

![Figure 4-16: First buckling mode predicted FEA](image)

### 4.5.2 Free vibration analysis

The designed CFRP floor panel as a standard building component must take an account of vibration problems to avoid noise and any possible resonance when the floor panel is used under service conditions. This problem can be solved by using design codes. All design limits specified in British Standards (BS 6472:1992 and BS 6841:1987) and International Standards (ISO 2631-1:1997, ISO 2631 2:2003 and ISO 10137:2006) are recommend by the Eurocodes (NSC, 2007).

Design code (Murray, Allen and Ungar, 2003) suggests the use of a limited ratio of peak acceleration in design check for vibration issues. It is required that the ratio of peak acceleration applied, \( a_p/g \), must be less than the acceleration limit, \( a_o/g \), given by eq. 4.23.

\[
\frac{a_p}{g} = \frac{P_o \exp(-0.35f_n)}{\beta W} \leq \frac{a_o}{g} \tag{4.23}
\]

\( P_o \) is a constant force representing the excitation, \( f_n \) is fundamental natural frequency, \( \beta \) is model damping ratio, \( W \) is effective weight supported by the beam or joint panel, girder panel or combined panel, as applicable, \( a_p/g \) is the ratio of peak acceleration applied, \( a_o/g \) is acceleration limit from Table 4.10.

For the proposed CFRP floor panel, \( P_o \) is 0.29kN as a constant force for large buildings from Table 4.10, \( \beta \) is 0.03 as recommend by Table 4.4, the effective weight (\( W = 96\text{kg} \)) of a whole CFRP panel was calculated using the volume 0.081m\(^3\) of the panel given by AutoCAD multiplied by the density 1502K/m\(^3\), \( f_n \) was taken as 25Hz from the first eigenvalue analysis presented in Tale 4.11. Bringing all these parameters into eq. 4.23 results in eq. 4.24.
It can be seen from eq. 4.24 that the applied peak acceleration ratio is only 0.5% of the acceleration ratio limit. This indicates that the proposed CFRP floor panel is under the vibration criteria; it has a low acceleration ratio from the human activities. Table 4.11 presents the calculated frequencies of the first five free vibration modes. The value of frequency of the first free vibration mode, 25.1, was used in eq. 4.24 to obtain the applied peak acceleration ratio.

### Table 4.10: Recommended values of parameters in Equation 4.23 and acceleration ratio limit

<table>
<thead>
<tr>
<th>Type of building</th>
<th>Constant Force $P_o$</th>
<th>Damping Ratio $\beta$</th>
<th>Acceleration Limit $A_o/g \times 100%$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Offices, Residences, Churches</td>
<td>0.29kN</td>
<td>0.02- 0.05*</td>
<td>0.5%</td>
</tr>
<tr>
<td>Shopping Malls</td>
<td>0.29kN</td>
<td>0.02</td>
<td>1.5%</td>
</tr>
<tr>
<td>Footbridges - Indoor</td>
<td>0.41kN</td>
<td>0.01</td>
<td>1.5%</td>
</tr>
<tr>
<td>Footbridges - Outdoor</td>
<td>0.41kN</td>
<td>0.01</td>
<td>5.0%</td>
</tr>
</tbody>
</table>

*0.02 is for floors with few non-structural components (ceilings, ducts, partitions, etc.), which can be found in open work areas and churches, 0.03 for floors with non-structural components and furnishings, only small, demountable partitions, 0.05 for full height partitions between floors.

### Table 4.11: Frequencies predicted by FEA modeling

<table>
<thead>
<tr>
<th>Free vibration mode</th>
<th>1</th>
<th>2</th>
<th>3</th>
<th>4</th>
<th>5</th>
</tr>
</thead>
<tbody>
<tr>
<td>Frequency (Hz)</td>
<td>25.1</td>
<td>28.4</td>
<td>44.9</td>
<td>54.0</td>
<td>74.3</td>
</tr>
</tbody>
</table>

Fig. 4-17 shows the first free vibration mode of the CFRP floor panel with a frequency of 25.1Hz. It can be seen from Fig. 4-17 that the first model of the panel is a global twisting
vibration mode, however, the human activities-related frequency is much lower than the first vibration frequency according to calculations from eq. 4.24.

4.6 Conclusion

This chapter presented the detailed design of the proposed FRP floor panel. The basic model of the floor panel designed has a 6m span, 0.5m width and 0.13m height. It was designed using Eurocodes and satisfied all the design failure criteria, including critical deflection and material strength in the key directions. According to the Hashin criteria check, the tensional stresses in transverse and through-thickness directions at the end of panel need to be carefully considered. These tensional stresses are concentrated at the corners of the end section, however, the proposed panel has a 35% safety margin. Sections 4.4.8 and 4.4.9 presented and discussed the changes for the stresses and deflection by the variation of span, height, width and bonding length. Figures 4-12 through 4-14 provided three major groups of design curves for the design of the floor panels when different sizes are required. The design curves, given by Equations 4.7 through 4.22, can be further used in the design of CFRP floor panels, while taking into consideration variations for span, height and width. In addition, buckling and free vibration analyses were carried out to be sure the proposed CFRP floor panel has none of those problems when under the service conditions.

It should be noted that this design was supported by FEA modeling work. Considering the cost of pultrusion of the full CFRP floor panel, this design will be validated through experimental work on a scaled CFRP floor panel. Chapter 5 presents detailed validation work together with the consideration of a scaling factor for the application of full CFRP floor panels.
5

EXPERIMENTAL AND MODELING WORK OF SCALED SAMPLES
5.1 Introduction

The designed floor panel given in Chapter 4 is proposed to be manufactured by pultrusion. For the purpose of carrying out experimental validation, test samples are required, but manufacturing a full floor panel with 6m span and a complex cross-section would be expensive. Investigation of scaled specimens aims to validate the design of the full floor panel by an economic approach. Considering high manufacturing cost in the pultrusion of CFRP samples, all specimens were made in the way in which one straight plate and four channel section strips are bonded together. It should be noted that pultrusion and bonding technique would supply different qualities. However, this investigation assumed there is no debonding in scaled test samples when assess their structural behaviour using material strength and deflection based criteria under design load. These criteria will keep designing structures within the limited states. Actually, this assumption makes adhesively bonded samples have similar behaviour to pultruded ones within a limited state. From the view of validation, the debonding load of bonded test samples can be referred as a final failure load to compare with the design load in examining the safety margin. It should also be noted that because scaled samples were used for validation of the design of the CFRP floor panel, a scaling effect was investigated based on small specimens to validate full panels.

Experimental investigations were conducted in conjunction with numerical analyses in this Chapter. There are three basic objectives of the experimental program:

- Experimental test of scaled samples to validate the design of a CFRP floor panel
- Numerical modeling analysis
- Investigation of scaling effects

To accomplish these objectives, a methodology implementing two stages of experimental investigations together with modelling work was given below:

- Design and manufacture of test samples
- Performing experimental tests of scaled samples under statistical loading
- Analytical and numerical work of scaled samples

A general description of experimental program and results from experimental tests, analytical, and numerical work are provided and discussed in the following sections.

5.2 Test sample and mold description
5.2.1 Materials

All test samples of the scaled floor panel were made from the same carbon fibre composite material. This material was supplied by Gurit Ltd, a supplier of composite and adhesive material. Figure 5-1 shows the unidirectional (UD) carbon prepreg, called WE 91 HSC100, which was used for manufacturing test samples. This prepreg was supplied in a thin sheet. The tow sheet was supplied with waxed paper backing to keep the thin layers separate and to aid in unrolling during installation or use. The thickness of the prepreg sheet is 0.1 mm. The properties of the prepreg, WE91 HSC100, are shown in the Table 4.1 in Chapter 4.

Figure 5-1: Carbon prepreg

5.2.2 Geometrical shape and dimensions

The cross section of the designed specimen can be seen in Fig. 5-2. The specimen consists of five elements, including a top plate and 4 channel section strips. The top plate was adhesively bonded on the top of four channel section strips. The middle web consists of two channel section strips bonded together back to back. This specimen was designed using a 1:20 ratio to the original design.

Figure 5-2: A cross-section view of scaled sample

(Unit for all dimensions: mm)
The total width and height of the specimen are 25mm and 7.5mm, respectively. That measurement means the specimen is equivalent to the original size of 500mm width and 150mm height. Because the hydraulic press oven has limited length 220mm for samples, the test sample was designed with 200mm span between two supports. The extra 20mm was spate into 10mm at each end of the sample for installation. Two different CFRP plates with thickness 0.5mm and 0.4mm for the top plate and the channel section, respectively. This scaled specimen is simplified from the designed model with the consideration of changing the complex shape of the cross section to one with a constant thickness. How to produce these elements will be given in the next section.

5.2.3 Production of CFRP elements

As mentioned in Chapter 3, there are a number of different techniques for producing FRP components, such as pultrusion, hand layup, etc. It is difficult to use the pultrusion method for producing the specimen because its thickness is very thin. Also, pultrusion is a very expensive manufacture technique because of the dies required. Therefore, the hot press moulding method was used to produce the thin elements required, including the flat plate and channel section elements.

![Die for producing top plate of specimen](image)

Fig. 5-3: Die for producing top plate of specimen (unit for all dimensions: mm)

It can be seen from Figure 5-2 that there are actually two simple elements needed for making the specimen: the top flat plate and the four channel section strips. To produce the top plate of the specimen, two thick steel plates are required to be created by the moulding approach, shown in Fig. 5-3. The following is the procedure for producing the CFRP plates: first, the
release film A5000 (show as Figure 5-4, left) must be laid on top of one steel plate. The release film is green colour and has 0.1mm thickness. The resistant temperature of the release film is 26°C (Umeco, n.d). The designed thickness of the plate is 0.5mm. Therefore, five WE 91 HSC100 carbon sheets were applied. Then, another release film was placed on top of the prepregs and covered by another steel plate. Finally, all of these were placed into the oven for curing, with a temperature of 120°C and 2 bar pressure for one hour. After curing and cooling down the steel plate, the moulding with the CFRP materials was moved out, the release agents peeled, and the CFRP plate cut with the geometric dimensions required by the design.

The channel section strip is a bit of difficult in producing because of the thin materials and curved corners. The designed mould was shown in Fig. 5-5. The mould consists of top die and bottom die, and both are steel elements. Detailed dimensions of dies can be seen from Figure 5-5. The procedure for producing the channel section strips is similar to that for making top plates. The release film with 0.05mm thickness, A2000 (Fig 5-4 right), was required to be laid on top of the lower die. The resistant temperature of the release film is 150°C (Umeco, n.d). The designed thickness of the channel section strip is 0.4 mm. Therefore, four WE 91 HSC100 carbon prepregs were required. Then, one must be laid on another release film on top of the prepregs and be covered with the upper die. This prepared mould was put into the oven with 120°C and 2 bar pressure for one hour. The dies together with films and CFRP sheet can be seen from Figure 5-6. After curing and cool down, samples were removed, and release films were peeled. Finally, the required CFRP plate was cut in accordance with the designed dimensions.

![Figure 5-4: Release films A5000 (left) for top plate and the A2000 (right) for channel section strip](image-url)
5.2.4 Adhesives

Bonding the top plate with the four channel section strips was completed using Araldite adhesive. The Araldite adhesive is an epoxy resin which comes with two tubes. One is epoxy, and the other is the hardener. Application of the Araldite requires these two tubes mix at the same time and completely. The Araldite epoxy adhesive has a good performance on metals,
glass, porcelain, china, and fibre composite materials. Heat is not necessary, although warming the adhesive will reduce the curing time and improve the strength of the bond. The properties of the Araldite are shown in table 5.1

Table 5.1: Mechanics properties of Araldite 2015 (Adhere n.d)

<table>
<thead>
<tr>
<th>Adhesive (Araldite 2015)</th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Yong’s modulus</td>
<td>E</td>
<td>1850 MPa</td>
</tr>
<tr>
<td>Shear modulus</td>
<td>G</td>
<td>650 MPa</td>
</tr>
<tr>
<td>Poison ratio</td>
<td>ν</td>
<td>0.35</td>
</tr>
<tr>
<td>Tensile Strength</td>
<td>Σ</td>
<td>30 MPa</td>
</tr>
</tbody>
</table>

5.3 Facilities

Experimental investigations of the scaled samples were performed at the Mechanical Engineering Laboratories, for the sample making and testing. The following facilities were used in experimental work.

5.3.1 Oven

A hydraulic press, shown in Figure 5-7, was used for making all scaled specimens. The hydraulic press is 2.5 meters in height and 2 meters in width. Specimens, up to 500 mm wide and 400 mm long, can be placed into the oven. The oven allows a max temperature of 500°C and 150 bar pressure.

![Figure 5-7: Bytec Hydraulic Press](image-url)
5.3.2 Test machine

The Zwick/Roell Z030 universe machine with force range of 10mN to 30 KN, as shown in Figure 5-8, was used in the tests.

![Figure 5-8: Zwick/Roell Z030 test machine](image)

5.4 Experimental set-up

All experimental tests were carried out by three-point bending. The experimental set-up can be seen in Figure 5-9.

![Figure 5-9: Experimental set-up](image)

5.5 Description of specimens

Figure 5-10 shows one of the specimens. Due to the error of hot press die and hand bonding, the specimen cannot be accurately produced as what was designed. Therefore, the thickness of the sample can only be produced as an averaged value to meet the thickness required by design. Although the thickness of the adhesive layer cannot be controlled exactly, the total height of the specimen was measured at the value designed.
5.6 Test procedure

In total, eight samples were used in this investigation. At each test, the loading speed 1mm/min was applied. The computer recorded the load deflection data. The record step was set as one second, and the test stopped when the large deformation reached 10mm.

5.7 Test results and discussion

Figure 5-11 shows test results from eight samples presented by load-deflection curves. It can be seen from Figure 5-11 that the initial failure load ranged from 330N to 490N for most specimens, except one specimen, #3, reached to 490N. Corresponding deflection to the failure load ranged from 3.7mm to 5.4mm. Load-deflection curves have significant drops at failure loads, due to the large area of debonding. It can be seen from Figure 5-11 that load-deflection curves are almost linear before failure loads. The residual stiffness is very low after failure. This indicated that the CFRP panel lost loading capacity, once failed by debonding. According to the lab test report, most samples debonded first at the interface between top plate and side web. This debonding happened because the side web was very deformed due to stability lost when loads were increased. It should be noted that load-deflection curves are not always smooth because of micro-matrix cracks or fault from the adhesives.
Chapter 5: Experimental and Modeling work of Scaled Samples

Figure 5-11: Load-deflection curves

Table 5.2: Specimens’ test results

<table>
<thead>
<tr>
<th>Specimen</th>
<th>#1</th>
<th>#2</th>
<th>#3</th>
<th>#4</th>
<th>#5</th>
<th>#6</th>
<th>#7</th>
<th>#8</th>
<th>Mean</th>
</tr>
</thead>
<tbody>
<tr>
<td>Failure Load (N)</td>
<td>330</td>
<td>426</td>
<td>490</td>
<td>452</td>
<td>416</td>
<td>402</td>
<td>389</td>
<td>334</td>
<td>407</td>
</tr>
<tr>
<td>Deflection (mm)</td>
<td>4.5</td>
<td>5.4</td>
<td>5.4</td>
<td>5.3</td>
<td>5.0</td>
<td>3.7</td>
<td>4.4</td>
<td>3.9</td>
<td>4.7</td>
</tr>
</tbody>
</table>

Figure 5-12: Histogram for 8 samples
The failure loads are relevant to the bonding quality of the scaled test samples. It was hard to control the thickness of the adhesives between elements of samples. The actual thickness of the adhesives determined the bonding quality and affected the failure loads. Another reason for adverse effects is the manufacturing error inherent in making samples. As mentioned in Section 4.5, the thickness of the sample is only 0.4 and 0.5 mm for the web and top plate. The mould’s error could be 0.01 mm, but the error from the mould plus the error from pressures applied from the oven, along with the thickness of the components, could hardly be controlled exactly at 0.4 and 0.5 mm. However, an averaged thickness seems a good value in terms of reported results from tests.

Table 5.2 shows selected results, including maximum deflection and failure loads from the eight tested specimens. Averaged deflection value is 4.7 mm. It should be noted that the values for the deflection and failure load from specimens #1, #6, and #8 are lower than the average. The reason for this is the individual poor bonding quality. Specimen #3 has the highest failure load and over-average deflection. From Figure 5-12, it can be seen that these eight samples presented a standard distribution as the highest frequency is in the middle, the figure is symmetric, and shape is normal. Therefore, from the feature of Figure 5-12, the normal distribution method can be used for these eight samples’ statistical analysis. Figure 5-11 also shows the mean curve by the red line, which was created by the mean of deflection and failure load, based on results from the eight tested samples given in Table 5.2.

Table 5.3: Statistical results

<table>
<thead>
<tr>
<th>Statistical Items</th>
<th>Mean</th>
<th>Standard Deviation (σ)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Failure Load (N)</td>
<td>407</td>
<td>54.7</td>
</tr>
<tr>
<td>Design value</td>
<td></td>
<td><strong>Failure Probability against design value</strong></td>
</tr>
<tr>
<td></td>
<td>53.50</td>
<td>Mean - Design value=350 $\geq 4\sigma=219$, 0%</td>
</tr>
</tbody>
</table>
Table 5.3 presents statistical results, including the mean and standard deviation, for the deflection and failure load, according stochastic theory.

Figure 5-13: Normal distribution

Figure 5-13 shows the normal distribution of the test results. The graph is generated by eq. 5.1. The parameter \( \mu \) is the mean, \( \sigma \) is known as the standard deviation and \( x \) is variable.

\[
f(x) = \frac{1}{\sigma \sqrt{2\pi}} e^{-(x-\mu)^2/2\sigma^2} \quad (5.1)
\]

The probability density of the mean has the highest value. However, the design load is 53.50N, which is much smaller than the mean 407N.

Figure 5-14: Cumulative distribution
Figure 5-14 is the cumulative distribution curve, which is an easy way to find out the possibility of the failure load. This curve is plotted using eq. 5.2, where ERF is called the error function, also called the Gauss error function (Patel and Read, 1996).

\[
\frac{1}{2} \left[ 1 + \text{erf} \left( \frac{x - \mu}{\sqrt{2\sigma^2}} \right) \right] \tag{5.2}
\]

According to the theory of the normal distribution (Patel and Read, 1996), if the value of probability density is smaller than \(4\sigma\), the probability is less than 0.0001%. In this investigation, the design value \(F_d\) (53.5N) is smaller than the mean \(\mu\) (407N) minus four times standard deviation \(\sigma\) (54.7N) as shown in eq. 5.3.

\[F_d < \mu - 4\sigma \tag{5.3}\]

Therefore, the failure probability against design value is 0%. This statistical result proved that the loading capacity of the designed panel is high enough with 0% failure probability.

### 5.8 Modeling of scaled floor panels

The FEA model for the scaled test panel is a half of the geometrical body, considering the symmetry in longitudinal direction. An initial FEA model consists of 10,000 thick shell elements in total.

![Figure 5-15: A half FEA model with a line load](image)

The boundary conditions for this model are simply support at the end and fixed restraint in X direction at the middle of the panel to simulate the symmetry on the longitudinal direction. A point load was applied on the top of the middle section of the panel. This point load was actually applied as a distributed line load across the width of panel, which was simulating the
loading condition in the tests. The value of this load was worked out by the un-factored design load divided by the width of the panel as \(37.5 \text{N}/25 \text{mm}=1.5 \text{N/mm}\). The final line distributed load on the panel was taken half as 0.75N/mm and used in the half modelling.

The deformed model, shown in Figure 5-16, is mainly a panel bent in the direction of the length. It can be seen from the Figure 5-16 that the side web at the end of the panel bent outward. This bending at the end section can cause high stresses locally around the corners. These high stresses at the corners would be critical stresses which may fail the panel. Figure 5-16 also shows that the top plate of the model deflected between two webs. This minor bending could increase the stresses at the corners between the top plate and the side web at the end section of the panel.

**Figure 5-16: A deformed mesh**

### 5.8.1 Effects of varied mesh densities on structural behaviour

Before performing a strength check, a convergent problem similar to that of full panel modelling given in Chapter 4 was studied in this section. Tables 5.4 and 5.5 show the investigation of maximum stress varied mesh densities in span and height respectively. Effects of the varied mesh densities in the direction of the width were ignored because they are less effective in terms of the investigation of full panel given in Chapter 4. In-plane stress components, \(\sigma_{xt}, \sigma_{xc}, \sigma_{yt}, \sigma_{yc},\) and \(\sigma_{xy}\), were investigated in this section. Stress in the thickness direction was ignored in the shell modelling. Table 5.4 shows the results against varied mesh densities from 100 to 160 divisions along the span. By considering stress concentration around the corners, high density meshes were given in the area which is closer to both ends. It can be seen from the Table 5.4 that the maximum stress is generally convergent after mesh
density 140. Therefore, mesh density with 140 divisions along the span was chosen in the final FEA model for the strength check.

Table 5.4: Effects of varied mesh densities in span on the maximum stresses

<table>
<thead>
<tr>
<th>Stress component</th>
<th>100</th>
<th>120</th>
<th>140</th>
<th>160</th>
</tr>
</thead>
<tbody>
<tr>
<td>$\sigma_{xt}$</td>
<td>85.16</td>
<td>85.2</td>
<td>85.2</td>
<td>85.2</td>
</tr>
<tr>
<td>$\sigma_{xc}$</td>
<td>-127.67</td>
<td>-127.81</td>
<td>-127.84</td>
<td>-127.85</td>
</tr>
<tr>
<td>$\sigma_{yt}$</td>
<td>22.06</td>
<td>22.06</td>
<td>22.06</td>
<td>22.06</td>
</tr>
<tr>
<td>$\sigma_{yc}$</td>
<td>-28.26</td>
<td>-28.29</td>
<td>-28.3</td>
<td>-28.3</td>
</tr>
<tr>
<td>$\sigma_{xy}$</td>
<td>-16.9</td>
<td>-16.9</td>
<td>-16.9</td>
<td>-16.9</td>
</tr>
</tbody>
</table>

The convergent study regarding varied mesh densities in height is given in table 5.5. Mesh densities varied from 80 to 120 divisions in height. The stress components, $\sigma_{xc}$, $\sigma_{yc}$, and $\sigma_{xy}$, appeared to converge at the range of mesh densities 80 to 120. The high density area was given at the areas which are closer to the bottom because the highest values of $\sigma_{yc}$ and $\sigma_{yc}$ were found around the corners at the bottom. According to the results shown in table 5.5, the mesh density with 120 divisions in height was applied in the final FEA modelling analysis for strength check.

Table 5.5: Effects of varied mesh densities in height on the maximum stresses

<table>
<thead>
<tr>
<th>Type of stress</th>
<th>80</th>
<th>90</th>
<th>100</th>
<th>100</th>
<th>120</th>
</tr>
</thead>
<tbody>
<tr>
<td>$\sigma_{xt}$</td>
<td>85.22</td>
<td>85.22</td>
<td>85.22</td>
<td>85.22</td>
<td>85.22</td>
</tr>
<tr>
<td>$\sigma_{xc}$</td>
<td>-227.4</td>
<td>-228.87</td>
<td>-229.97</td>
<td>-230.82</td>
<td>-231.51</td>
</tr>
<tr>
<td>$\sigma_{yt}$</td>
<td>22.06</td>
<td>22.06</td>
<td>22.06</td>
<td>22.06</td>
<td>22.06</td>
</tr>
<tr>
<td>$\sigma_{yc}$</td>
<td>-65.27</td>
<td>-65.50</td>
<td>-65.69</td>
<td>-65.83</td>
<td>-65.93</td>
</tr>
<tr>
<td>$\sigma_{xy}$</td>
<td>-32.855</td>
<td>-32.895</td>
<td>-32.925</td>
<td>-32.945</td>
<td>-32.965</td>
</tr>
</tbody>
</table>

**5.8.2 Effects of varied mesh densities in span and height on Hashin criteria indicators**

All data of maximum stresses were collected from the FEA modelling with varied mesh densities in span and height. When brought into the Hashin criteria equations stated in Chapter 4, the failure indicators were calculated and this is shown in the Tables 5.6 to 5.7. It can be seen from the Table 5.6 that the maximum failure indicator is the matrix tensile failure
indicator 0.46. The corresponding matrix compressive failure indicator is only 0.15. This indicates that the investigated test panel has enough safety margins under design load.

Table 5.6: Hashin criteria check with varied mesh densities in span

<table>
<thead>
<tr>
<th>Mesh Density in span</th>
<th>100</th>
<th>120</th>
<th>140</th>
<th>160</th>
</tr>
</thead>
<tbody>
<tr>
<td>Stresses in the middle of the top plate between two webs for fibre tensile failure check</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>$\sigma_{xt}$</td>
<td>85.16</td>
<td>85.2</td>
<td>85.2</td>
<td>85.2</td>
</tr>
<tr>
<td>$\sigma_{xy}$</td>
<td>0.001</td>
<td>0.001</td>
<td>0.001</td>
<td>0.001</td>
</tr>
<tr>
<td>Stresses at the at right-low corner of the end section for fibre compressive failure check</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>$\sigma_{xc}$</td>
<td>-127.67</td>
<td>-127.81</td>
<td>-127.84</td>
<td>-127.85</td>
</tr>
<tr>
<td>Stresses in the middle of the top plate between two webs for transverse matrix tensile failure check</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>$\sigma_{yt}$</td>
<td>22.04</td>
<td>22.05</td>
<td>22.06</td>
<td>22.06</td>
</tr>
<tr>
<td>$\sigma_{xy}$</td>
<td>-0.001</td>
<td>-0.001</td>
<td>-0.001</td>
<td>-0.001</td>
</tr>
<tr>
<td>Stresses at the right-low corner of the end section for transverse matrix compressive failure check</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>$\sigma_{yc}$</td>
<td>-28.26</td>
<td>-28.29</td>
<td>-28.3</td>
<td>-28.3</td>
</tr>
<tr>
<td>$\sigma_{xy}$</td>
<td>-3.64</td>
<td>-3.7</td>
<td>-3.7</td>
<td>-3.7</td>
</tr>
<tr>
<td>Hashin Criteria Check (failure indicator: 1)</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Fibre tensile failure indicator</td>
<td>0.004</td>
<td>0.004</td>
<td>0.004</td>
<td>0.004</td>
</tr>
<tr>
<td>Fibre compressive failure indicator</td>
<td>0.02</td>
<td>0.02</td>
<td>0.02</td>
<td>0.02</td>
</tr>
<tr>
<td>Transverse matrix tensile failure indicator</td>
<td>0.46</td>
<td>0.46</td>
<td>0.46</td>
<td>0.46</td>
</tr>
<tr>
<td>Transverse matrix compressive failure indicator</td>
<td>0.15</td>
<td>0.15</td>
<td>0.15</td>
<td>0.15</td>
</tr>
</tbody>
</table>

Table 5.7 shows the effect of varied mesh densities in height on Hashin criteria indicators. It can be seen from Table 5.7 that the Hashin failure indicators remained stable or converged very well in the arranged of mesh densities from 80 to 120 divisions. Two crucial failure indicators 0.46 and 0.51 are in the matrix tensile direction and matrix compressive direction respectively.

Table 5.7: Hashin criteria check with varied mesh densities in height

<table>
<thead>
<tr>
<th>Mesh Density in height</th>
<th>80</th>
<th>90</th>
<th>100</th>
<th>110</th>
<th>120</th>
</tr>
</thead>
<tbody>
<tr>
<td>Stresses in the middle of the top plate between two webs for fibre tensile failure check</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>$\sigma_{xt}$</td>
<td>85.22</td>
<td>85.22</td>
<td>85.22</td>
<td>85.22</td>
<td>85.22</td>
</tr>
<tr>
<td>$\sigma_{xy}$</td>
<td>0.001</td>
<td>0.001</td>
<td>0.001</td>
<td>0.001</td>
<td>0.001</td>
</tr>
</tbody>
</table>
Stresses at the at right-low corner of the end section for fibre compressive failure check

| $\sigma_{xc}$ | -227.4 | 228.87 | 229.97 | 230.82 | 231.51 |

Stresses in the middle of the top plate between two webs for transverse matrix tensile failure check

| $\sigma_{yt}$ | 22.06 | 22.06 | 22.06 | 22.06 | 22.06 |
| $\sigma_{xy}$ | -0.009 | -0.009 | -0.009 | -0.009 | -0.009 |

Stresses at the right-low corner of the end section for transverse matrix compressive failure check

| $\sigma_{yc}$ | -65.27 | -65.50 | -65.69 | -65.83 | -65.93 |
| $\sigma_{xy}$ | -13.6 | -13.75 | -13.88 | -14 | -14.11 |

Hashin Criteria Check (failure indicator: 1)

| Fibre tensile failure indicator | 0.004 | 0.004 | 0.004 | 0.004 | 0.004 |
| Fibre compressive failure indicator | 0.05 | 0.05 | 0.05 | 0.06 | 0.06 |
| Transverse matrix tensile failure indicator | 0.46 | 0.46 | 0.46 | 0.46 | 0.46 |
| Transverse matrix compressive failure indicator | 0.50 | 0.51 | 0.51 | 0.51 | 0.51 |

Results from the analysis of Hashin criteria indicators given in the Tables 5.4 to 5.7 were used for making a final FEA model with the mesh density 140 and 120 divisions in span and height respectively. The mesh density in width was taken as 120 divisions. The final FEA model is shown in Figure 5-17.

Table 5.8 presents the states of strength check using the Hashin failure criteria. It can be seen from Table 5.8 that the scaled test panel passed all the Hashin failure criteria checks when using the total design load of 53.5N. The maximum Hashin criteria indicators, 0.46 and 0.51, are for transverse matrix tension and transverse matrix compression check respectively. These can produce a big safety margin about 49%.
It can be seen from Table 5.8 that the transverse matrix tensile and compressive stress play an important role in critical strength checks. These can be used to work out an initial failure load in the transverse direction which is presented by Equations 4.3 and 4.4. Within the elastic stage of stress analysis, all stresses increase linearly. Therefore, the relationship between the initial failure load and design load can be expressed by corresponding stress ratio \( \lambda \) given in equation 5.4.

\[
\frac{\text{Failure load}}{\text{Design load}} = \frac{\text{Stress under failure load}}{\text{Stress under design load}} = \lambda \quad (5.4)
\]

Therefore, stress under failure load = \( \lambda \times \) (stress under design load) \quad (5.5)

Substituting Equation 5.5 into Equations 4.3 and 4.4 satisfies the Hashin criteria in the transverse direction which then results in Equations 5.6 and 5.7 respectively.

\[
\alpha^2 \left( \frac{\sigma_{22d} + \sigma_{33d}}{Y_T^2} \right)^2 + \frac{\sigma_{23d}^2 - \sigma_{22d} \sigma_{33d}}{S_{23}^2} + \frac{\sigma_{12d}^2 + \sigma_{13d}^2}{S_{12}^2} = 1 \quad (5.6)
\]

\[
\alpha^2 \left[ \left( \frac{Y}{2S_{23d}} \right)^2 - 1 \right] \left( \frac{\sigma_{22d} + \sigma_{33d}}{4S_{23}^2} \right)^2 + \frac{\sigma_{23d}^2 - \sigma_{22d} \sigma_{33d}}{4S_{23}^2} + \frac{\sigma_{12d}^2 + \sigma_{13d}^2}{S_{12}^2} = 1 \quad (5.7)
\]

Where, \( \sigma_{12d} \), \( \sigma_{13d} \), \( \sigma_{22d} \), \( \sigma_{23d} \) and \( \sigma_{33d} \) are the stresses under design load. Through-thickness stress can be ignored in shell modelling analysis. Therefore \( \sigma_{13d} \), \( \sigma_{23d} \), \( \sigma_{23d} \) and \( \sigma_{33d} \) were given as zero in the Equations 5.6 and 5.7.

Using results given in the Tables 5.6 and 5.7, it can be known that \( \sigma_{12d} \) is very small. Thus Equation 5.6 can be written as

\[
\alpha = \frac{Y_T}{\sigma_{22d}} \quad (5.8)
\]

Combining eqs. 5.4 and 5.8 results

\[
\text{Failure load (Matrix tensile)} = \frac{Y_T}{\sigma_{22d}} \times \text{Design Load} = \frac{32.5}{22.6} \times 53.5 = 76.9N
\]
Using stresses given in Tables 5.6 and 5.7 into Equation 5.7, the calculated rate of $\frac{\sigma_{22d}^2}{S_{12}^2}$ is also small, about 2%. Therefore, this rate can be ignored.

Thus
\[
\left( \frac{Y_c}{2S_{23}} \right)^2 = \left( \frac{108.3}{2 \times 71.9} \right)^2 = 0.567 .
\]

Substituting these values into Equation 5.7 results in Equation 5.9 as shown below

\[
\lambda^2 \left[ \frac{\sigma_{22d}^2 - 0.433\sigma_{22d}}{4S_{23}^2} \right] = 1 \quad (5.9)
\]

Bring the value of the maximum $\sigma_{22d}=108.3$MPa given in the table 4.1 results in $\lambda=1.33$.

Thus, a failure load based on the matrix compression can be worked out as below.

\[
\text{Failure load (Matrix compression)} = \lambda \times \text{Design Load} = 1.33 \times 53.5 = 71.22\text{N}
\]

Comparing the results 76.9 and 71.22N, the smaller value should be used as a predicted initial failure load. This estimated initial failure load, 71.22N, was used in modelling analysis, and corresponding stresses were brought back into Hashin criteria in Equation 4.4. The calculated indicator 1.0 in the transverse matrix compressive direction given in the Table 5.8 proved this estimated initial failure load. This estimated initial failure load caused a problem because the transverse matrix stress produced was in excess of the matrix compressive strength at the end of scaled panel. It should be noted that this predicted initial failure load is about 18% of the tested debonding load 407N. Therefore, a large safety margin has been seen again in this investigation.

<table>
<thead>
<tr>
<th>Table 5.8: Design check for deflection and strength</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Design criteria</strong></td>
</tr>
<tr>
<td><strong>Deflection criterion</strong></td>
</tr>
<tr>
<td>L/250 = 0.8mm</td>
</tr>
<tr>
<td><strong>Hashin criteria</strong></td>
</tr>
<tr>
<td>Failure indicator</td>
</tr>
<tr>
<td>Fibre tension</td>
</tr>
<tr>
<td>Fibre compression</td>
</tr>
<tr>
<td>Transverse matrix tension</td>
</tr>
</tbody>
</table>
### Transverse matrix compression

<p>| | | | |</p>
<table>
<thead>
<tr>
<th></th>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>1</td>
<td>0.51</td>
<td>Pass</td>
</tr>
</tbody>
</table>

**Predicted initial failure load 71.22N**

<p>| | | | |</p>
<table>
<thead>
<tr>
<th></th>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Fibre tension</td>
<td>1</td>
<td>0.01</td>
<td>Pass</td>
</tr>
<tr>
<td>Fibre compression</td>
<td>1</td>
<td>0.08</td>
<td>Pass</td>
</tr>
<tr>
<td>Transverse matrix tension</td>
<td>1</td>
<td>0.96</td>
<td>Pass</td>
</tr>
<tr>
<td>Transverse matrix compression</td>
<td>1</td>
<td>1</td>
<td>Failure</td>
</tr>
</tbody>
</table>

#### 5.8.3 Stress contour

The presentation of the stress contour is similar to that given in the Section 4.4.6.6, which investigated stress distribution in fibre-direction and transverse matrix direction. Stress contour will show the maximum stress values and their locations.

#### 5.8.3.1 Stress contour in X-direction

Figure 5-18 shows stress contour in the X (fibre) direction. The maximum compressive stress, 231.5 MPa, is located at the lower right corner of the panel. It is only about 23% of compressive strength (984.2MPa). The maximum tensile stress (85.22MPa) is located on the top of the plate and in the middle of the component. The maximum tensile stress is about 6% of the CFPR tensile strength. Obviously, these values indicated that the stresses in fibre direction are not crucial because the carbon fibre has very high strength.

![Stress distribution in fibre direction](image)

(a) Tensile stress (b) compressive stress

Figure 5-18: Stress distribution in fibre direction

#### 5.8.3.2 Stress contour in transverse direction
The maximum stresses in the transverse direction were located at the end section of the scaled panel. The maximum tensile stress, 22.06MPa, is located in the middle of the top plate and between two webs, and the maximum tensile stress, 65.93 MPa, is located at the low-right corner of the web, show in Figure 5-19 (a) and (b) respectively. The maximum tensile stress is only about 67% of the tensile strength of the CFRP used in test samples, and the maximum compressive stress is about 61% of the compressive strength. Because the tensile stress and compressive stress in transverse direction are considerable values comparing to the material strength, which are critical stresses in strength check.

(b) Tensile stress                                                 (b) compressive stress

Figure 5-19: Stress distribution in transverse direction

5.8.3.3 Shear stress contour in YZ Plane

From the modelling analysis, the maximum shear stresses are located in the YZ plane.

Figure 5-20: Shear stress distribution in YZ plane

Figure 5-20 shows the shear stress distribution. The highest shear stresses, -32.965MPa, are located at the low-left corner at the end of panel. Compared to the shear strength of 71.9MPa,
the maximum shear stress is about 46% of shear strength. Therefore, the shear stress is not a critical stress for the specimen.

5.8.4 Debonding of scaled test samples

Because this scaled sample was adhesively bonded, debonding modes was briefly investigated by 15 samples. According to the sample tests results, there were three failure modes, as shown in Figure 5-21. It can be seen from the stress contour that the maximum compressive stress in the transverse direction is a critical stress (65.93MPa), located on the right bottom corner. It is nearly 61% of material tensile strength 108.3MPa. Therefore, the right corner would more likely be damaged first. Then, the damaged corner would cause the web to become unstable. Any small increased load, extra bending movement due to inaccurate geometry, dislocation in bonding procedure, etc., would let the web buckle earlier because the buckling capacity was reduced by above-mentioned reasons. The buckled web was largely deformed and finally broken in transverse direction. This is the first failure mode, shown in Figure 5-21a, that was observed from a number of samples with good manufacturing quality. It is interesting to point out that the crack on the web did not go through the whole length of the specimen. Crack length was about a quarter length of the sample. There were 11 samples out of 15 that were broken on the web. The percentage of this type of failure mode is 73%.

The bulked web quite possibly let the upper right corner bend down significantly which caused debonding at the right corner between the right web and the top plat. Thus, the whole component suddenly lost loading capacity. This is the second failure mode observed from those samples with less bonding quality. There were 3 specimens out of 15, which appeared debonding failure. The debonding started at the outer edge of the bonding area and propagated inward as seen in Figure 5-21b. The percentage of debonding failure is 20%. Debonding went nearly through the whole length of the specimen. Low bonding quality could be caused by several factors, such as short curing time, non-uniformly distributed adhesives, etc.

There was only one sample out of fifteen which broke at the top plate. The position of the crack on the top plate is close to the upper right corner. No matter what the reason for this
failure mode is, it should be not a main failure mode (see Figure 5-21c). Experimental investigation also indicated that some samples were slightly crushed on the loading line. That crushing means that the compressive stress through the thickness was greater than compressive strength. However, the crushing would not affect the actual behaviour of the panel. The proposed floor panel would not be designed to directly undertake the line distributed load. Therefore, the study of the failure mechanism of scaled floor panels should be focused on the web breaking and debonding. As mentioned in previous sections, design load is much less than the debonding failure load, this brief investigation just draws an attention on the estimated loading capacity of bonded CFRP samples.

![Failure modes](image)

Figure: 5-21: Failure modes (a) Web broken (b) Debonding (c) Top plate broken

5.9 Scale effect

The scaled samples have the benefit for saving time and financial source. However, understanding the relationship between scaled model and the full model is necessary. Many investigations of ‘composite size effect’ have been based on statistical strength theory or statistical weakest link theory for conventional brittle fracture study. Weibull (1939) made
great advances in the subject, giving his name to the most widely used distribution used in Weakest-Link theory and showing that the theory could be applied to many brittle materials. Rosen (1964), Zweben and Rosen (1970), Harlow and Phoenix (1978), Smith (1980), Batdorf (1990), and Daniels (1945) stated that Weakest-Link theory accurately describes the failure of brittle materials and fibre composite.

5.9.1 Weakest-link theory
The probability of failure of each link subjected to a stress increase from 0 to \( \sigma \) is described by the distribution function \( F(\sigma) \). The probability of survival of that link is then given by eq. 5.10.

\[
S(\sigma) = 1 - F(\sigma) \quad (5.10)
\]

Hence the probability of failure of a chain of \( n \) elements can be given by eq. 5.11.

\[
F_n(\sigma) = 1 - [1 - F(\sigma)]^n \quad (5.11)
\]

The function \( F(\sigma) \) can be expressed generally by eq. 5.12.

\[
F(\sigma) = 1 - \exp[1 - \varphi(\sigma)] \quad (5.12)
\]

Equation (5.12) forms the basis of statistical weakest link theory. A specific form of \( \varphi(\sigma) \) was put forward by Weibull (1939), and this function has become known as the ‘Weibull distribution’ and is given by eq. 5.13.

\[
\varphi(\sigma) = \left( \frac{\sigma - \sigma_u}{\sigma_0} \right) \quad (5.13)
\]

Where \( \sigma_u \) is a characteristic strength, \( \sigma_0 \) is called the shape parameter. This form is termed the three parameter distribution. The lower bound of strength, \( \sigma_u \), is assumed to be zero (Stevens & Clausen, 1969). This leads to a probability of failure of \( n \) elements in series of

\[
F_n(\sigma) = 1 - \exp \left[ -n \left( \frac{\sigma}{\sigma_0} \right)^{\delta} \right] \quad (5.14)
\]

Where \( \delta \) is a scale parameter. Considering a volume of material comprising of small elemental volumes, \( \delta V \), instead of a chain of `links' gives

\[
F_v(\sigma) = 1 - \exp \left[ - \int \left( \frac{\sigma}{\sigma_0} \right)^{\delta} dV \right] \quad (5.15)
\]
Where $\sigma$ is tested tensile stress, uniformly distributed through the material volume. Integrating eq. 5.15 gives

$$F_v(\sigma) = 1 - \exp\left[-V\left(\frac{\sigma}{\sigma_0}\right)^m\right] \quad (5.16)$$

If the strength distribution of a material is described by Weibull theory, then it is possible to correlate the strengths of specimens or components of differing size. An assumption is made that the values of the shape, scale parameters $m$, and shape parameter $\sigma_0$, are material constants and independent of the size of the specimen and its stress field. Using Equation 5.16 for the same probability of failure of two specimens with identical stress distributions, a relationship between stress ratio and value ratio can be expressed by eq. 5.17.

$$\frac{\sigma_2}{\sigma_1} = \left(\frac{V_1}{V_2}\right)^{\frac{1}{m}} \quad (5.17)$$

This equation directly links strength to volume and hence quantifies the size effect. A logarithmic plot of stress versus volume gives a straight line relationship of slope $-1/m$, as shown in Fig. 5-22, which shows that strength is reduced as the volume increases.

![Logarithmic plot of a strength size effect.](image)

The volumes for the scale model and full panel required by eq. 5.17 were calculated by AutoCAD. The ratio between $\sigma_1$ (full panel strength) and $\sigma_2$ (scale model strength) can be known by the scale parameters $m$ as two values, $V_1$ and $V_2$. Previous researchers Sutherland, L.S, Shenoi, R.A, and Lewis (1999) studied a large variation of $m$ from 10.3 to 38.4 and suggested that an averaged value of $m$ could be taken as 20 in the Weibull analysis.
5.9.2 Calculation of scaling effects

As mentioned above, eq. 5.17 can be used to reduce the strength by the scale effect. Therefore, the final material strength of the full panel can be calculated in the following. Scaled sample: $A=28.82 \text{mm}^2$, $V=A \times L = 28.82 \times 200 = 5764 \text{mm}^3$. Full panel: $A=13026 \text{mm}^2$, $V=A \times L = 13026 \times 6000 = 78156000 \text{mm}^3$. Bring these parameters into eq. 5.17 results

$$\frac{\sigma_2}{\sigma_1} = \left(\frac{V_1}{V_2}\right)^{\frac{1}{m}} = \left(\frac{78156000}{5764}\right)^{\frac{1}{20}} = 1.6$$

This ratio, 1.6, between $\sigma_1$ and $\sigma_2$ means the material strength of the full panel should be reduced by 37.5%, assuming scaled specimen has original material strength. The reduced material strength due to the scale effect was used for the final Hashin criteria check in the transverse matrix tensile direction, which is crucial according previous analysis, and calculated failure indicators are below 1. This is shown in the Table 5.9.

Table 5.9: Hashin criteria check using reduced strength by the scaled effect

<table>
<thead>
<tr>
<th>Hashin criteria</th>
<th>Failure indicator</th>
<th>Result</th>
<th>State</th>
</tr>
</thead>
<tbody>
<tr>
<td>Transverse matrix tension</td>
<td>1</td>
<td>0.90</td>
<td>Pass</td>
</tr>
<tr>
<td>Transverse matrix compression</td>
<td>1</td>
<td>0.86</td>
<td>Pass</td>
</tr>
</tbody>
</table>

The scaling effect of the studied CFRP materials for flooring in buildings has been investigated in this section. The scale effect ratio 1.6 was worked out in terms of the weakest-link theory. Accounting this scale effect, the final strength check by Hashin failure criteria was carried out, and the crucial indicator of transverse matrix tension showed that the designed CFRP floor panel still passed strength check.

5.10 Conclusion

Chapter 5 presented an investigation of scaled floor panels. It validated the design of the full floor panels and verified the corresponding FEA models. The work in this chapter includes design, manufacture, testing, and FEA modelling work of scaled floor panels. All tested results have been used in comparison with the FEA prediction. Although scaled test samples
were bonded panels, the tested results under design load passed all the design criteria with plenty of safety margin. Because design load is much less than the debonding load, there will be no debonding in service condition. Therefore, the test of bonded panel under design load can validate the designed CFRP floor panel proposed to be manufactured by pultrusion. Comparing to the loading capacity predicted by FEA, the design load of the full floor panel is just about a quarter of the predicted loading capacity. This was verified by the tested failure loads of scaled floor panels. Finally, the scaling effect was also considered, which verified that a full CFRP floor panel is still safe when using a reduction factor of material strength between scaled and full panel.
SHEAR EFFECTS OF CFRP FLOOR PANELS
6.1 Introduction

Either a full CFRP floor panel or a scaled test panel is a beam with an open cross section and double cells consisting of a number of thin-walled plates. As mentioned in chapters 3 and 4, the deflection check plays an important role in designing such CFRP floor panels. The thin-walled beam usually has a significant shear effect on the deflection. This section will focus on the investigation into theoretical calculations of deflection with shear effects. Analytical solutions will be checked by experiment and FEA modelling work of a scaled panel. Then a proposed formulation of deflection with shear effects will be applied to the prediction of deflection of full floor panels and proved by FEA modelling.

6.2 Analytical calculation of deflection

6.2.1 Deflection of a floor beam with an open cross-section due to bending

According to Megson’s previous work (Megson, 2005), the deflection due to the bending of a beam with a symmetric cross-section can be calculated by eq. 6.1 given below:

\[
\frac{d^4 v}{dx^4} = -\frac{w}{EI} \quad (6.1)
\]

The deflection at the middle of the beam, with uniformly distributed load (UDL) and simple supports, can be calculated by eq. 6.2:

\[
v_{\text{mid-span}} = -\frac{5wL^4}{384EI} \quad (6.2)
\]

The cross-section of the investigated CFRP floor panel is not exactly symmetric because of two side slits for overlapping. This asymmetrical feature can be seen in figure 4-1. Therefore, an exact equation to consider any effects due to this asymmetrical feature on bending behaviour is required.

Figure 6-1: A beam with simple supports and uniformly distributed load
Figure 6-1 shows a beam with simple supports, subjected to an un-factored design load of 3.75N/mm. This uniformly distributed load was calculated by an un-factored design load of 0.0075N/mm², given in chapter 4, multiplied by the width (500mm) of the panel. Therefore, the bending moment at any point along the X-axis can be calculated by eq. 6.3:

$$ M_z = \frac{wx(l-x)}{2}, \quad M_y = 0 \quad (6.3) $$

Firstly, assume the beam is symmetric, and then the bending moment and flexural rigidity can be presented by the follow equation:
\[ \frac{1}{R} = \frac{M}{EI} \quad (6.4) \]

It can be seen from figure 6-2 that the curved radius is perpendicular to the neutral axis. Assume that at some section of a beam, the deflection is normal to the neutral axis. Therefore, an absolute deflection is \( \zeta \). Then, as shown in figure 6-3, the central G is displaced to G'. The components of \( \zeta \), u and v, are given by eq. 6.5 as shown below:

\[ u = \zeta \sin \theta, \quad v = \zeta \cos \theta \quad (6.5) \]

The centre of curvature of the beam lies in a longitudinal plane perpendicular to the neutral axis of the beam and passes through the centric of any section.

Figure 6-4: Bending of a beam with an asymmetrical cross-section (Megson, 2005)

Hence, for a radius of curvature R, by direct comparison with eq. (6.4) that:

\[ \frac{1}{R} = \frac{d^2 \zeta}{dx^2} \quad (6.6) \]

Substituting eq. 6.6 into eq. 6.5 results in eq. 6.7 as shown below:

\[ \frac{\sin \theta}{R} = \frac{d^2 u}{dx^2}, \quad \frac{\cos \theta}{R} = \frac{d^2 v}{dx^2} \quad (6.7) \]

From the direct stress distribution formula:
\[ M_z = -\int_A \sigma_x y \, dA \quad , \quad M_y = -\int_A \sigma_x y \, dA \quad (6.8) \]

As stress is strain multiplied by Young’s modulus then:

\[ \sigma_x = -\frac{Ey}{R} \quad (6.9) \]

From figure 6-4, eq. 6.9 can be rewritten as:

\[ \sigma_x = -\frac{E}{R} (z \sin \theta + y \cos \theta) \quad (6.10) \]

Substituting \( \sigma_x \) from eq. 6.10 into eq. 6.8 obtains:

\[
egin{align*}
M_z &= \frac{E \sin \theta}{R} \int_A zy \, dA + \frac{E \cos \theta}{R} \int_A y^2 \, dA \bigg{) } \\
M_y &= \frac{E \sin \theta}{R} \int_A z^2 \, dA + \frac{E \cos \theta}{R} \int_A zy \, dA \\
\end{align*}
\]

(6.11)

In eq. 6.11:

\[ \int_A zy \, dA = I_{zy} \quad \int_A y^2 \, dA = I_z \quad \int_A z^2 \, dA = I_y \]

Eq. 6.11 may, therefore, be rewritten as:

\[
\begin{align*}
\frac{E \sin \theta}{R} &= \frac{M_y I_z - M_z I_{zy}}{I_z I_y - I_{zy}^2} \quad (6.12) \\
\frac{E \cos \theta}{R} &= \frac{M_z I_y - M_y I_{zy}}{I_z I_y - I_{zy}^2} \quad (6.13)
\end{align*}
\]

Therefore, from eq. 6.4, we can have:

\[
\begin{align*}
\frac{d^2 u}{dx^2} &= \frac{M_y I_z - M_z I_{zy}}{E(I_z I_y - I_{zy}^2)} \quad (6.14) \\
\frac{d^2 v}{dx^2} &= \frac{M_z I_y - M_y I_{zy}}{E(I_z I_y - I_{zy}^2)} \quad (6.15)
\end{align*}
\]

Substitute eq. 6.3 into eq. 6.11, eq. 6.12 can be reduced as q. 6.16 below:
\[
\frac{d^2 u}{dx^2} = \frac{-wI_{zy}}{2E(I_x I_y - I_{zy}^2)}(lx - x^2) \quad (6.16)
\]

Integrating with respect to x obtains:

\[
\frac{du}{dx} = \frac{-wI_{zy}}{2E(I_x I_y - I_{zy}^2)}\left(\frac{x^2}{2} - \frac{x^3}{3} + C_1\right)
\]

Consider the symmetry in longitudinal, at the mid-span section \(x=l/2\) the slope gradient \(du/dx=0\). Hence:

\[
0 = \frac{l^3}{8} - \frac{l^3}{24} + C_1
\]

Whence:

\[
C_1 = -\frac{l^3}{12}
\]

Therefore:

\[
\frac{du}{dx} = \frac{-wI_{zy}}{2E(I_x I_y - I_{zy}^2)}\left(\frac{x^2l}{2} - \frac{x^3}{3} - \frac{l^3}{12}\right) \quad (6.17)
\]

Integrating eq. 6.17 with respect to x results:

\[
u = \frac{-wI_{zy}}{2E(I_x I_y - I_{zy}^2)}\left(\frac{x^3l}{6} - \frac{x^4}{12} - \frac{x l^3}{12} + C_2\right)
\]

When \(x=0, u=0\), then \(C_2 = 0\) can be achieved from the above equation, so we have:

\[
u = \frac{-wI_{zy}}{2E(I_x I_y - I_{zy}^2)}\left(\frac{x^3l}{6} - \frac{x^4}{12} - \frac{x l^3}{12}\right) \quad (6.18)
\]

The displacement \(u\) at the middle of the beam, \(x=l/2\), can be expressed as below:

\[
u = \frac{5wl^4 I_{zy}}{384E(I_x I_y - I_{zy}^2)} \quad (6.19)
\]

Using a similar process to achieve \(u\), the deflection \(v\) in the vertical direction can be obtained from eq. 6.15 as:
Chapter-6: Shear Effects of CFRP Floor Panels

\[ v = \frac{wI_y}{2E(I_z I_y - I_{zy}^2)} \left( \frac{x^3 l}{6} - \frac{x^4}{12} - \frac{x l^3}{12} \right) \]  \hspace{1cm} (6.20)

At the middle of the beam \( x = l/2 \):

\[ v_{l/2} = \frac{-5wl^4 I_y}{384E(I_z I_y - I_{zy}^2)} \]  \hspace{1cm} (6.21)

Where \( w \) is distributed load, \( I_z \) and \( I_y \) are the second moment of area about the Z and Y axes, \( I_{zy} \) is second moment of area about the Z-Y plane, \( l \) is the span of the beam. In eq. 6.21, \( I_{zy} \) presents the effect of an asymmetrical cross-section on the deflection of beam. In the case of symmetrical cross-section, \( I_{zy} = 0 \), thus eq. 6.21 is the calculation of deflection at the middle of beam with symmetrical cross-section as same as eq. 6.2.

In order to find out the asymmetrical effect, the result from eq. 6.21 will be used to compare the result from eq. 6.2. Substituting the \( w = 3.75N/mm \), \( A = 13026mm^2 \), \( I_z = 31786226mm^4 \), \( I_y = 391171840mm^4 \), \( I_{zy} = 653491mm^4 \), \( L = 6000mm \), \( E = 130330Pa \) and \( G = 3590Pa \) into eq. 6.21 and eq. 6.2 respectively conducts results as below:

Asymmetrical cross section:

\[ v_{l/2} = \frac{-5 \times 3.75 \times 6000^4 \times 391171840}{384 \times 130330 \times (31786226 \times 391171840 - 653491^2)} = -15.2759mm \]

Symmetrical cross section:

\[ v_{l/2} = \frac{-5 \times 3.75 \times 6000^4}{384 \times 31786226 \times 130330} = -15.2754mm \]

It can be seen from the above results that the effect of the asymmetrical cross section of the proposed floor CFRP panel on the deflection can be ignored. The formulas for calculating bending deflection of a beam with a symmetrical cross-section can be accepted in the application of the proposed CFRP floor panel to make the calculation simple.
6.2.2 Shear effects on the deflection of a CFRP floor panel

In general, the deflection of the beam due to shear can be calculated from the follow equation (Megson, 2005):

\[ v_s = \frac{\beta}{G} \int \left( \frac{S_y}{A} \right) dx \] (6.22)

Where, \( L \) is span, \( A \) is the total area of cross-section, and \( G \) is material shear modulus, \( \beta \) is defined as the form factor (Megson, 2005) and \( S_y \) is shear force at the section investigated. It should be noted that eq. 6.22 is a general formula derived from the case of a beam with a solid cross-section (Megson, 2005). Thus, applying eq. 6.22 in the case of a beam with an open cross-section and multi cells consisting of thin-walled plates results in an incorrect answer in comparison to a tested result. Therefore, it was suggested that the shear related deflection should be corrected considering the loading related distortion of such a cross-section using experimental work studied in this investigation. A similar result was reported by Schniepp (2002) regarding the shear effect of a thin-walled CFRP beam used in bridges. Thus, the deflection at the middle of the investigated beam due to shear was proposed to be expressed by eq. 6.23, under a UDL load:

\[ v_s = \frac{\beta \alpha w l^2}{8AG} \] (6.23)

In this investigation, an uniformly unfactored distributed load \( w=3.75N/mm \), \( G \) is material shear modulus 3590MPa given in table 4.1, \( A \) was calculated as the area 13026 mm\(^2\) by Autocad, span \( L \) is taken as 6000 mm. The form factor \( \beta \) in eq. 6.23 will be theoretically derived to partly consider the shear effects on deflection in the particular case of the panel with an open cross-section and multi cells consisting of thin-walled plates, and validated by experimental work on a scaled test panel with a similar cross-section in the following section. It should be noticed that \( \alpha \) in eq. 6.23 is proposed as a deflection correction factor to account for the loading related distortional effects on the deflection of the panel with an open cross-section considering distortion. This deflection correction factor was proposed to vary with different loading cases. Both point loading and uniformly distributed loading cases were considered in this investigation of a beam with simple supports. The following sections will give details of the conduction of the form factor \( \beta \) and the load-deflection factor \( \alpha \).
6.2.3 Analysis of scaled test panels

6.2.3.1 Shear distribution

As with the proposed full CFRP floor panel, the scaled test beam is a one-way spanning slab because fibres are placed along the span of beam. It has a similar cross-section (shown in figure 6-5) to the full CFRP floor panel. The dimension of the cross-section of the scaled test beam can be seen in figure 5-6. Firstly, longitudinal bending stress can be simply calculated by eq. 6.24:

\[
\sigma_z = -\frac{M}{I_z} y \quad (6.24)
\]

Where, \( \sigma_z \) is stress in fibre direction, \( I_z \) is second moment of area about z axis, \( y \) is the distance from the neutral axis to the stress point studied. It can be seen from eq. 6.24 that the maximum bending stress is located at the middle section of the slab because of the maximum bending moment applied at the middle section, where, \( M = 2625 \text{Nmm}, I_z = 201.36 \text{mm}^4 \), \( y = 2.4 \text{mm} \). The bending stress distribution under design load is shown in figure 6-5:

![Figure 6-5: Bending stress distribution](image)

Secondly, shear stress distribution at the cross-section shown in figure 5-21 can be calculated by eq. 6.25:

\[
\tau = \frac{FA'}{b_0 I_z} \quad (6.25)
\]

Where, \( F \) is the maximum shear force and was taken as 26.5N in the investigated scaled panel, \( A' \) is a variable area above \( \bar{y} \), which is the distance of the centroid of the variable area from the neutral axis of the cross section, \( b \) is the variable width of the layer on which shear stress distribution is sought, and the value of \( I_z \) was calculated by Autocad as 201.36mm\(^4\).
Because the width of the cross-section is not uniform from the top to the bottom, the cross-section needs to be separated into four sections as A-B, C-D, E-G and H-I, which are shown in figure 6-6. The shear stress is zero on top and on the bottom, the maximum shear stress will occur on the neutral axis as shown in figure 6-7.

Shear stress at point A in figure 6-6, $\tau_A = 0$

At point B:

\[
A' = Bt_1, \quad b = B = 25\text{mm}
\]

At point C:

\[
A' = Bt_1, \quad b = 8\text{mm}
\]

Figure 6-6: Dimension of specimen’s cross-section

At point C:

\[
A' = Bt_1, \quad b = 8\text{mm}
\]
\[ \bar{y} = h_1 + t_2 + \frac{t_1}{2} = 2.15 \text{ mm} \]

\[ \tau_c = \frac{FBt_1 \times \bar{y}}{bI_z} = 0.44 N / mm^2 \]

At point D:

\[ A' = Bt_1 + bt_2, \ b = 8 \text{mm} \]

\[ \bar{y} = \frac{Bt_1 \times (h_1 + t_2 + t_1/2) + t_2b \times (h_1 + t_2/2)}{A'} = 2.06 \text{mm} \]

\[ \tau_d = \frac{FA' \bar{y}}{bI_z} = 0.5273 N / mm^2 \]

At point E:

\[ A' = Bt_1 + 8t_2, \ b = 1.6 \text{mm} \]

\[ \bar{y} = \frac{Bt_1 \times (h_1 + t_2 + t_1/2) + t_2b \times (h_1 + t_2/2)}{A'} = 2.06 \text{mm} \]

\[ \tau_e = \frac{FA' \bar{y}}{bI_z} = 2.64 N / mm^2 \]

At point F:

\[ A' = Bt_1 + 8t_2 + bh_1 = 18.1 \text{mm}^2, \ b = 1.6 \text{mm} \]

\[ \bar{y} = \frac{Bt_1 \times (h_1 + t_2 + t_1/2) + t_2 \times 8 \times (h_1 + t_2/2) + h_1b \times (h_1/2)}{A'} = 1.88 \text{mm} \]

\[ \tau_f = \frac{FA' \bar{y}}{bI_z} = 2.77 N / mm^2 \]

At Point I, \[ \tau_i = 0 \]

At point H:

\[ A' = bt_3 = 3.2 \text{mm}^2, \ b = 8 \text{mm} \]

\[ \bar{y} = h_2 + \frac{t_2}{2} = 4.9 \text{mm} \]

\[ \tau_h = \frac{FA' \bar{y}}{bI_z} = 0.26 N / mm^2 \]

At point G:
Chapter-6: Shear Effects of CFRP Floor Panels

\[ A' = 8t_3 = 3.2 \text{mm}^2, \ b = 1.6 \text{mm} \]

\[ \bar{y} = h_2 + \frac{t_1}{2} = 4.9 \text{mm} \]

\[ \tau_G = \frac{FA' \bar{y}}{bI_z} = 1.28 \text{N/mm}^2 \]

![Figure 6-7: Shear stress distribution]

### 6.2.3.2 Form factor

The form factor \( \beta \) can be calculated by eq. 6.26 (Megson, 2005).

\[
\beta = \frac{A}{I_z} \int_{y_1}^{y_2} \frac{(A' \bar{y})}{b} \ d\bar{y} \quad (6.26)
\]

Using eq. 6.26, \( \beta \) can be calculated by adding all contributions from each segment shown in figure 6-6 as stated below. **A** is the total area of the cross-section of the specimen (28.82 mm\(^2\)).

**Form factor** \( \beta_{A-B} \)

\[
\beta_{A-B} = \frac{A}{I_z} \int_{y_1}^{y_2} \frac{(A' \bar{y})}{b} \ d\bar{y} \quad A = 28.82 \text{mm}^2 \quad I = 201 \text{mm}^4
\]

\[
A' = 25 \times (2.4 - y) \quad Z(y) = \frac{2.4 - y}{2} \quad \bar{y} = 2.4 - Z(y)
\]

\[
\beta_{A-B} = \frac{28.82}{201^2 \times 25} \int_{y_1}^{y_2} (25 \times (2.4 - y)(y + \frac{2.4 - 7}{2}))^2 \ d\bar{y} = 0.0036
\]
Form factor $\beta_{C-D}$

$$\beta_{C-D} = \frac{A}{I_z} \int_{0.5}^{1.5} \frac{(A')^2}{b} dy \quad A' = 25 \times 0.5 + 8 \times (1.9 - y)$$

$$Z(y) = \frac{25 \times 0.5 \times 0.25 + 8 \times (1.9 - y)(2.4 - y) \div 2}{25 \times 0.5 + 8 \times (1.9 - y)} \quad \bar{y} = 2.4 - Z(y)$$

$$\beta_{C-D} = \frac{28.82}{201^2 \times 6.8} \int_{0.5}^{1.5} (25 + 8(1.9 - y)(24 - \frac{25 \times 0.5 \times 0.25 + 8 \times (1.9 - y)(2.4 - y) \div 2}{25 \times 0.5 + 8 \times (1.9 - y)})^2 dy = 0.031$$

Form factor $\beta_{E-G}$

$$\beta_{E-G} = \frac{A}{I_z} \int_{-4.7}^{1.5} \frac{(A')^2}{b} dy \quad A' = 25 \times 0.5 + 8 \times 0.4 + (1.5 - y) \times 1.6$$

$$Z(y) = \frac{25 \times 0.5 \times (0.5 \div 2) + 8 \times 0.4 \times (0.5 + 0.4 \div 2) + 1.6 \times (h_k - y)(0.9 + 0.5(1.5 - y))}{25 \times 0.5 + 8 \times 0.4 + (1.5 - y) \times 1.6}$$

$$\bar{y} = 2.4 - z(y)$$

$$\beta_{E-G} = \frac{28.82}{201^2 \times 1.6} \int_{-4.7}^{1.5} (25 + 8 \times 0.4 + 1.6(1.5 - y)(24 - \frac{25 \times 0.5 \times (0.5 \div 2) + 8 \times 0.4 \times (0.5 + 0.4 \div 2) + 1.6 \times (h_k - y)(0.9 + 0.5(1.5 - y))}{25 \times 0.5 + 8 \times 0.4 + (1.5 - y) \times 1.6})^2 dy = 2.479$$

Form factor $\beta_{H-I}$

$$\beta_{H-I} = \frac{A}{I_z} \int_{-5.1}^{-4.7} \frac{(A')^2}{b} dy \quad A' = 8 \times (5.1 - y)$$

$$Z(y) = (5.1 - y) / 2 \quad \bar{y} = 5.1 - Z(y)$$

$$\beta_{A-B} = \frac{28.82}{201^2 \times 8} \int_{-5.1}^{-4.7} (8 \times (5.1 - y)(y + \frac{5.1 - 7}{2}))^2 dy = 0.0030$$

Therefore, the form factor of this open cross-section of the scaled test panel can be summated as:

$$\beta = \beta_{A-B} + \beta_{C-D} + \beta_{E-G} + \beta_{H-I} = 2.51$$
6.2.3.3 Conduction of load-deflection correction factor

The test sample has a symmetric cross-section as shown in figure 6-4. Therefore, its bending deflection under a point load and a simple support can be calculated by eq. 6.27.

\[ v_{\text{mid-span}} = -\frac{PL^3}{48EI_z} \] (6.27)

Meanwhile, the deflection due to shear cannot be ignored because this beam has an open cross-section and multi cells consisting of thin-walled plates. The total deflection in the middle of the beam can be expressed by a combination of bending deflection and the deflection due to shear for the case with simple supports and a point load shown in figure 5-9. The second item in eq. 6.28 was actually conducted using eq. 6.22 for the case of a simply supported beam under a point load, and considering an open cross-section and multi cells consisting of thin-walled plates.

\[ v_{l/2} = -\frac{PL^3}{48EI_z} - \frac{\beta \alpha PL}{4AG} \] (6.28)

In eq. 6.28, \( I_z = 201,360 \, \text{mm}^4 \) and \( A = 28.82 \, \text{mm}^2 \), calculated by AutoCad. The values for point load \( P \) and deflection were taken as means given in table 5.2. Material Young’s modulus and shear modulus were taken as 130330MPa and 3590MPa from table 4.1. In order to conduct the deflection correction factor in the point loading case, tested mean deflection 4.7mm, failure load 404N and the form factor (\( \beta = 2.51 \)) were used in eq. 6.28. Thus the shear deflection corrector \( \alpha \) was worked out as 4.28 for the point loading case. This conducted load-deflection correction factor is a physically determined correction factor because conduction used tested data in eq. 6.28. From eq. 6.28, the deflection \( V_{l/2} = 4.7 \text{mm} \) and includes a bending deflection of 2.6mm, so the value of shear deflection is 2.1mm, which is about 45% of total deflection at the middle of beam. This indicates that the shear effect on deflection is significant because of an open cross-section with multi cells consisting of thin-walled plates. It should be noticed that previous research by Schniepp (2002) used PL/kAG as shear stiffness to replace the second item in eq. 6.28. In Schniepp’s investigation, the parameter \( k \) was determined by experimental work and varied with different loading cases. Actually, \( k \) is equivalent to \( 4/\beta \alpha \) in this investigation.
Using a similar approach presented above, load deflection correction factors in the case of a beam under a UDL load and simple supports can be worked out. Unfortunately, because of the equipment restraints in the laboratory, the required tests of the scaled samples under UDL and simple supports were not completed during the lab work period during this investigation. As a complement, it was suggested the load-deflection correction factor could be worked out using UDL related eq. 6.29. The required values for deflection and loading can be obtained from FEA modelling.

\[
v_{u/2L} = -\frac{5wL^4}{384EI} - \frac{\beta\alpha_1L^2}{8AG} \quad (6.29)
\]

In eq. 6.29, the form factor \( \beta \) is taken as the same value 2.51 and all other material and beam section properties were kept as the same values as that in the point loading case. In a corresponding FEA modelling analysis, a UDL of \(-0.188\text{N/mm}\) was used, the predicted deflection was \(-0.19\text{mm}\). Thus, bringing \(-0.19\text{mm}\) together with all other parameters into eq. 6.29 results in \(\alpha=1.8\) in the case of the beam under UDL and simple supports. Table 6.1 shows all conducted load-deflection correction factors together with the form factor. Thus, these conducted factors based on a scaled panel will be used in the detailed calculation of deflection of a full CFRP panel in the following section.

It should be noted that in table 6.1 the theoretical deflections are exactly the same with a tested mean or a modelling prediction because they were used in the conduction of correction factors. However, FEA modelling prediction of the deflection of a full CFRP panel using these deflection correction factors conducted from a scaled panel will be a validation given in the following section. It can be seen from table 6.1 that the theoretical shear deflection is about 45% and 21% of total deflection in the point load and UDL cases, respectively. This demonstrates the shear effect of a scaled panel with an open cross-section is significant.

<table>
<thead>
<tr>
<th>Loading case</th>
<th>Scaled sample under un-factored design load and simple supports load (point load 37.5N, UDL 0.188N/mm), form factor (\beta=2.51)</th>
<th>Theoretical deflection (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Test load-deflection correction factor</td>
<td>Tested deflection mean (mm)</td>
<td>Bending</td>
</tr>
</tbody>
</table>

Table 6.1 Form factor and load-deflection correction factors conducted from a scaled panel
6.2.4 Theoretical calculation of deflections of full CFRP panels

Theoretical calculation of the deflections of full CFRP panels under un-factored design load and simple supports was carried out using the proposed formulas given in last section. Investigation includes FEA modelling work only for validation. The proposed formulas in the last section can be used for calculating the maximum deflection at the middle of a simple supported full CFRP panel with thin-walled and open cross-section. Two different loading cases were considered. Eq. 6.28 is for the point load case and eq. 6.29 is for the UDL case. In these equations, the form factor $\beta$ is taken as 2.5, the load-deflection correction factor $\alpha$ is taken as 4.3 for the point load case and 1.8 for the UDL case in accordance with the work given in the last section.

In the point load case, $P=0.0075\times500\times6000=22500N$, $A=13026mm^2$, $I_z=31786226mm^4$, $L=6000mm$, $E=130330Pa$, $G=3590Pa$, $\beta = 2.5$ and $\alpha = 4.3$. Bringing these parameters into eq. 6.28, the deflection of a full panel with a pointed load and simple supports can be calculated as:

$$-\frac{22500\times6000^3}{48\times31786226\times130330} - \frac{2.5\times4.3\times22500\times6000}{4\times13026\times3590} = -24.441 - 7.217 = -32.23mm$$

The theoretically calculated deflection -32.23mm agrees with the FEA modelling prediction - 30.78 shown in table 6.2. The difference is only 5%. The theoretical shear deflection is about 22% of total deflection in this case.

In the UDL case, $w=0.0075\times500=3.75N/mm$, $A=13026mm^2$, $I_z=31786226mm^4$, $L=6000mm$, $E=130330Pa$, $G=3590Pa$, $\beta = 2.5$ and $\alpha = 1.8$. Bringing these parameters into eq. 6.29, the deflection of a full panel under UDL can be expressed as:
This theoretically calculated deflection of \(-16.9\,\text{mm}\) basically agrees with the FEA modelling prediction of \(-17.9\,\text{mm}\) shown in table 6.2. The difference between theory and modelling is about 6%. The shear deflection in this UDL case is 10% of total deflection.

It can be seen from table 6.2 that the theoretical deflection agrees well with the FEA modelling prediction in both the point load and UDL cases. Therefore, the conducted form factor 2.5 and load-deflection correction factor, 4.3 (pointed load) and 1.8 (UDL), are basically suitable for the calculation of deflection of the full CFRP panel. It also can be seen from tables 6.1 and 6.2; the shear effect on the deflection of a full panel is reduced by about 50% compared to that in the scaled panel.

### Table 6.2: Deflections of a full CFRP panel given by theory and FEA modeling

<table>
<thead>
<tr>
<th>Loading case</th>
<th>Full panel under un-factored design load and simple supports load (point load 22500N, UDL 3.75N/mm), form factor (\beta=2.51)</th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Load-deflection correction factor (\alpha)</td>
<td>FEA modelling deflection (mm)</td>
<td>Theoretical deflection (mm)</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Bending def.</td>
<td>Shear def.</td>
</tr>
<tr>
<td>Point load</td>
<td>4.28</td>
<td>-30.78</td>
<td>-24.44</td>
</tr>
<tr>
<td>UDL</td>
<td>1.8</td>
<td>-17.94</td>
<td>-15.28</td>
</tr>
</tbody>
</table>

### 6.3 Conclusion

The shear effect on the deflection of a full CFRP floor panel was identified by FEA modelling analysis in this investigation. This certainly proved that the form factor and load-deflection correction factors conducted from scaled panel are basically suitable in the calculation of the deflection of a full CFRP panel. Therefore, eqs. 6.28 and 6.29 are the final formulas for calculating the deflection of a simply supported CFRP full panel under a point load or a UDL, in which the form factor \(\beta\) should be taken as 2.5, the deflection correction
factors $\alpha$ are suggested to be taken as 4.3 and 1.8 in the point load and UDL cases respectively to correct the shear effects on deflection. The shear effects on the deflection of a full panel were represented by 22% and 10% of total deflection in the point load and UDL cases respectively. Obviously, this shear effect cannot be ignored, and the shear effect on deflection reduces as the span of the panel increases because the increased span of the panel will increase the bending effect.
7

THERMAL BEHAVIOUR OF FRP COMPONENTS AND FIRE SAFETY ENGINEERING DESIGN
Chapter-7: Thermal Behaviour of FRP Components and Fire safety engineering Design

7.1 Introduction

In this Chapter, the thermal behaviour of FRP materials will be discussed together with the behaviour of traditional building materials, such as brick, concrete, and steel. The application of FRP as a building material can benefit the environmental impact because FRP material has good thermal insulation properties. This property means that the application of FRP materials in building will save energy consumption (heat generation) and reduce carbon dioxide emissions. In order to analyse the thermal behaviour of the FRP materials, FEA modeling was employed to simulate the thermal insulation and heating energy loss caused by floor panels which consist of different building materials. Modeling results will be presented and discussed in the following sections.

7.2 Energy efficiency

In general, good insulation property has low thermal conductivity, high R-value, and low U-Value properties. Their definitions are given below.

7.2.1 Thermal conductivity (K), thermal resistivity(r), thermal resistance (R-Value) and thermal transmittance (U-Value)

Thermal Conductivity is usually called the k-value, and its unit is W/mK (Kelvin-meter per watt). Thermal conductivity is a measure of the rate at which heat is conducted through a particular material under specified conditions.

Thermal Resistivity is expressed by an R-Value with a unit of mK/W (Watt per meter-Kelvin). This is merely the reciprocal of thermal conductivity and can be calculated by:

\[ r = \frac{1}{k} \quad (7.1) \]

Where k represents the conductivity of the material (W/mK).

Similar to the k-value, r should be calculated based on a one metre thick piece of material. Thermal Resistance is usually presented by the R-value with a unit of m^2K/W. The R-value takes into account the thickness of the material, thus allowing for more accurate comparisons between materials with similar functions. Steel, wood, and concrete may all be used for the frame of the building; however, each has a radically different thickness and R-value. To
allow for a more accurate comparison in the insulation properties of each material, an R-value should be calculated. This unit is a measure of the opposition to heat transfer which is offered by a particular component in a building element. R-values are created by dividing the thickness of the material (metres) by the k-value for a particular material.

\[ R = \frac{d}{k} \quad \text{(or } R = d r) \quad (7.2) \]

Where \( d \) is the thickness of the material (m).

Thermal Resistance provides a specific result for a material of known thickness and can, therefore, allow for almost any material on site insulation properties to be examined. Thermal Transmittance is measured by the U-value, using the unit of W/m\(^2\)K. This unit is a measure of the overall rate of heat transfer by all mechanisms under standard conditions through a particular section of constructions. This measure takes into account the thickness of each material involved and is calculated from the R-values of each material, as well as constants accounting for surface transmittance (Rsi and Rso, inner and outer surfaces, respectively). Each of these has a standard value assigned which, in reality, may vary slightly but for the purposes of this work will be ignored.

\[ U = \frac{1}{Rsi} + R1 + R2 + \ldots + Ra + Rso \quad (7.3) \]

Where Rsi is the R value for the inner surface material, Rso is the R value for the outer surface material. R1 to Ra are the R value between the inner surface and the outer surface.

From the definitions of thermal conductivity, thermal resistivity, thermal resistance, and thermal transmittance, it can be concluded that a good insulation material must have low thermal conductivity (CLEAR Comfortable Low Energy Architecture, n.d).

### 7.2.2 Thermal conductivity for construction materials

Table 6.1 shows the thermal properties for traditional and modern building materials. It can be seen from this table that polyethylene foam has the best thermal resistance at 0.043 W/m°C conductivity, and it has the highest specific heat capacity and the lowest density. Brick has a thermal resistance of 0.84 W/m°C. However, stone and concrete have closer thermal conductivities at 1.4 and 1.3 W/m°C, respectively. The conductivity of CFRP is nearly three times less than concrete and stone at 0.43 W/m°C. Compared to the traditional
construction materials, such as brick, concrete, and stone, FRP material clearly has lower values for thermal conductivity, density, and specific heat capacity. Considering the practical applications of CFRP panels for floors and walls in buildings, the thermal conductive property of CFRP panels in through thickness direction should be used for accounting the heat lost. Mutnuri (2006) has pointed out that CFRP materials have better thermal conductive behaviour in through thickness direction than other directions. This thermal conductive property of CFRP panels in through thickness direction was given in the table 7.1 and used in this investigation.

Table 7.1: Thermal properties of construction materials

<table>
<thead>
<tr>
<th>Material</th>
<th>Density (kg/m³)</th>
<th>Thermal Conductivity (W/m°C)</th>
<th>Specific Heat Capacity (J/kg°C)</th>
<th>Thermal mass (MJ °C m³)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Brickwork (outer leaf)</td>
<td>1700</td>
<td>0.84</td>
<td>800</td>
<td>1.36</td>
</tr>
<tr>
<td>Cast concrete (dense)</td>
<td>2100</td>
<td>1.40</td>
<td>840</td>
<td>1.76</td>
</tr>
<tr>
<td>Stone (Artificial)</td>
<td>1750</td>
<td>1.3</td>
<td>1000</td>
<td>1.75</td>
</tr>
<tr>
<td>CFRP *</td>
<td>1650</td>
<td>0.43</td>
<td>950</td>
<td>1.57</td>
</tr>
<tr>
<td>Polyethylene foam</td>
<td>30</td>
<td>0.037</td>
<td>2220</td>
<td>0.07</td>
</tr>
</tbody>
</table>


7.2.3 Low CO₂ emission

There have been conflicting studies relating the comparisons between producing steel, concrete, and FRP products. Overall, it is felt that concrete and steel have similar environmental effects with the difference being that steel is more readily recycled at the end of life (Zhang, Amauchi, & Takahashi, 2011). Compared to concrete and steel production, manufacturing of FRP materials produces more CO₂ emission about 30 times more than that from manufacturing concretes (Greenspec, n.d) (Zhang, Amauchi, & Takahashi, 2011). These figures can be seen in Table 7.2. The same table also shows that the total CO₂ emission when manufacturing a FRP slab weighing 2994kg is about 2.5 times higher than that from manufacturing concrete slab. This emission would be a disadvantage when using FRP
materials in civil engineering. However, FRP still has attractive merits because of its significant low thermal conductivities which save heating energy and reduce CO₂ emissions from the heating system. Detailed thermal analysis and CO₂ emissions accounted from the heating system will be discussed in later sections.

Table 7.2 CO₂ emissions in producing the different building materials

<table>
<thead>
<tr>
<th>Materials (kg)</th>
<th>CO₂ emission (kg)</th>
<th>Concrete</th>
<th>0.76</th>
<th>CFRP</th>
<th>22.4</th>
</tr>
</thead>
<tbody>
<tr>
<td>Per Slab</td>
<td></td>
<td></td>
<td>Per Slab</td>
<td></td>
<td></td>
</tr>
<tr>
<td>(6m×0.5m²)</td>
<td></td>
<td></td>
<td>(6m×0.5m²)</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Total CO₂ emission (kg)</td>
<td>1197</td>
<td></td>
<td>2994</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Source from: materials (Greenspec, n.d) (Zhang, Amauchi & Takahashi, 2011)

7.2.4 Low embody energy and low thermal mass

According to the theory of passive solar design (Crobbie, 1997), high thermal mass can capture plenty of free solar energy (via large glazed walls or conservatories); it can store this heat inside of the walls and floors of heavyweight structures. During the night, instead of having to put the heating system on its highest setting, people can enjoy the residual heat stored inside of the structures (Brinkley, 2006). However, there is problem. A large amount of glazing is needed to draw enough heat during the day, and at night, this glazing will leak much of this stored heat back outside. In fact, even double glazed units leak six times more heat at night than walls or roofs. Most of the passive solar gains will be given back through the glazing at night. Another problem is in mostly overcast climates, like the U.K., only a small proportion of winter space heating can come from solar radiation. It is often estimated to be between 20% and 35% of the total space heating load. It is difficult to increase this proportion because if you insulate the house massively, the overall heat load can be reduced, but in so doing, the useful contribution from solar gain is also reduced. Because a massively insulated house shortens the period for required space heating to just the very coldest months of the year, this is precisely the time when passive solar radiation has the least energy to offer. From Table 7.1, the traditional building materials, concrete, stone, and brick, have similar thermal mass around 1.4 to 1.7 MJ/ °C m³. However, the insulation materials, polyethylene foam and rock wool, have very low thermal mass, rounded 0.03 to 0.07. These numbers mean that the traditional materials will absorb more than 20 times the heat of the
insulation materials. Therefore, the lower thermal mass and low thermal conductivity materials of polyethylene foam and rock wool, applied in the building, will benefit to energy and CO2 emission.

7.2.5 Energy loss

Thermal behaviour changes with different materials which will bring different levels of energy loss when these materials make up building components. This section will investigate this basic thermal feature of FRP, together with traditional building materials, through a simple panel model. When the heat goes through the panel, there are two kinds of energy loss. Firstly, heat goes through the panel and is lost on another side of the environment. Secondly, heat is absorbed by the panel. Figure 7-1 shows these two kinds of heat loss. The amount of heat lost in the panel depends on the building materials. Good thermal resistant material is better for stopping the heat that goes through the panel, but materials with lower thermal mass cause less heat to be absorbed by the panel. Therefore, the best option is to choose the construction material with the best thermal resistance and low thermal mass.

![Figure 7-1: Energy loss in brick block](image)

7.2.5.1 Analysis of heat energy absorbed by FRP panels

7.2.5.1.1 FEA models

There are five different panels selected in this investigation using FEA modeling simulation. These five panels have the same thickness of 0.1 meter. Each panel has two different temperatures on the inner surface and outer surface, 25°C and 0°C respectively, as shown in figure 7-1, which also present the outdoor and indoor temperatures. The investigated area of the panel is 1m x 1m. FEA modeling aims to discover the temperature change due to heat absorption of panels made from different materials. The investigation used Equation 1 for calculating the amount of energy absorbed by the panel.
\[ Q = cm\Delta T \quad (6.4) \]

Where \( c \) is Specific Heat, \( m \) is mass, and \( \Delta T \) is change of temperature.

Figure 7-2 shows the FEA model of a brick block. It can be seen from Table 7.1 that the thermal conductivity of a brick is 0.62 W/m°C. Specific heat (Thermal mass) of the brick is 1360000 J/m³°C. Equation 2, used to calculate the specific heat, is shown below.

\[ C = E \rho \quad (6.5) \]

Where \( C \) is specific heat, \( E \) is specific heat capacity, and \( \rho \) is density.

The convective heat transfer coefficient of air was used as 30W/m²°C, and it ranges from 10-100W/m²°C.

7.2.5.1.2 Analysis and discussion for absorption of heat energy by FRP panels

Figure 7-3 shows the contour of the temperature distribution after two hours. The inner side and outer side temperatures are 25°C and 0°C at the beginning. After two hours, according to the entity PHI (Potential) analysis, the inside panel’s temperature dropped down to 20.2°C, and the temperature of the other side of the panel raised to 2.5 °C by heat absorption.
Figure 7-3: Temperature distribution at panel cross section after 2 hours

![Temperature Distribution](image1)

Figure 7-4: Inside temperature against time of brick block

![Temperature Vs Time](image2)

Figure 7-4 shows the temperature variation on the inner surface of the panel throughout the two-hour process with 30 second intervals. It can be seen that there is a slight vibration in the
temperature during the first 1000 seconds. The temperature decreases first and then bounces back up; it goes down again, and this is followed by a gradual fall in the temperature-time curve. This range occurs because the panel entraps some of the energy within itself and blocks heat flowing outward. When heat starts flowing from the inner side of the panel, the entrapped energy slightly increased the temperature at the inner side the panel. Soon, this entrapped energy found its way out, and the temperature started to fall down gradually. From Figure 7-4, the inner panel temperature continued to drop during the two hours. The temperature dropped down to 5°C after two hours’ time. Thus, the heat absorbed by the panel, calculated by Eq. 7.4, is 6.5MJ/ m³.

Figure 7-5 shows the temperature changes of six different panels after two hours. It can be seen from Figure 7-5 that concrete and stone dropped dramatically because they both have higher thermal conductivity. CFRP material has better thermal insulation than the brick block, so the CFRP panel only had 1.8°C decreased change after two hours.

![Graph showing temperature changes of different panels after 2 hours](image_url)

**Figure 7-5: The reduction of temperature at inner panel after 2 hours**

The amount of energy absorption depends on two factors of the panel: thermal conductivity and thermal mass. Figure 7-5 shows five different panel’s thermal behaviours during the same time period (0 to 2 hours).

In the first 20 minutes, these five panels have similar thermal behaviours; however, after 20 minutes, stone, concrete and brick dropped dramatically. The inner temperature of the concrete block dropped from 25 to 18°C in a two hour period which was a 7°C decrease.
CFRP panel only had about a 1.5°C temperature drop, which is four times less than that of the concrete and stone block. On the other hand, the temperature of the CFRP panel with polyethylene foam has a tiny drop (0.5°C), which is 14 times less than concrete block’s.

Figure 7-6 shows an extended period from two hours to five hours in the analysis of thermal behaviour of five panels as discussed above. After five hours, all temperatures of the five panels became stable, but at different levels. In this phase, the panels stopped absorbing any heat energy. The CFRP panel with polyethylene foam has very good thermal resistance properties. It took about 1.5 hours to finish the process with heat absorbing and only a 0.5 °C temperature drop down. It saves lots of energy in the absorption stage. The other three panels have similar changes. The saturated stage is around 200 minutes.

![Figure 7-6: Construction material thermal behaviours in 5 hours’ time](image)

Table 7-3 shows the amount of energy absorbed by these five panels. Concrete block absorbed the most heat energy (13.8 MJ); stone block and brick block absorbed 13.0 MJ and 7.3 MJ, respectively. CFRP panel absorbed 4.6 MJ, which is 150% less than the traditional brick block. It is around three times less than concrete and stone block. However, it is not true that the whole panel consists of the CFRP materials. As Chapter 4 mentions, the weight of CFRP slab is 20 times lighter than that of concrete slab. Therefore, the best design is to use polyethylene attached to the FRP material to have better insulation and fire resistance properties. The result given in Table 7-3 shows that the energy absorption is very small: 0.04
MJ energy loss by absorption in this advanced panel. This property reduces energy loss 100 times when compared to traditional floors.

Table: 7.3 Energy losses by absorption from different panels

<table>
<thead>
<tr>
<th></th>
<th>Brick</th>
<th>Concrete</th>
<th>Stone</th>
<th>CFRP</th>
<th>CFRP + Polyethylene foam</th>
</tr>
</thead>
<tbody>
<tr>
<td>∆T(°C)</td>
<td>5.4</td>
<td>7.8</td>
<td>7.4</td>
<td>2.9</td>
<td>0.5</td>
</tr>
<tr>
<td>Specific Heat (J/kg°C)</td>
<td>800</td>
<td>840</td>
<td>1000</td>
<td>950</td>
<td>2200</td>
</tr>
<tr>
<td>Density (kg/m³)</td>
<td>1700</td>
<td>2100</td>
<td>1750</td>
<td>1650</td>
<td>40</td>
</tr>
<tr>
<td>Energy absorption (MJ/m³)</td>
<td>7.3</td>
<td>13.8</td>
<td>13.0</td>
<td>4.6</td>
<td>0.04</td>
</tr>
</tbody>
</table>

7.2.5.2 Analysis of energy loss by heat transmittance

The heat is not only lost through absorption into the panel, but it is also lost by the transmittance through the panel. Equation 7.6 gives the calculation of heat loss by transmittance through a panel with the same difference in temperature, area, and thickness. It presents low thermal conductivity and less heat lost by transmittance.

\[ Q = \frac{KA∆T}{L} \]  

(7.6)

Where:  
Q = heat transferred per unit time (W)  
K = thermal Conductivity (W/m°C)  
A = heat transfer area (m²)  
L = thickness of the panel (m)

And ∆T = temperature difference across the barrier (°C)

Figure 6-7 presents heat loss due to heat transmittance through different panels per day. The largest heat loss is found in concrete block at 39.5 MJ. The CFRP panel saves 70% of the heat energy lost by heat transmittance when compared to a concrete block. It can be seen
from Figure 7-7 that the thermal property of CFRP panel improves a lot by the addition of polyethylene foam which has eight times lower thermal conductivity than the pure CFRP panel.

![Energy loss by heat transfer through different panels](image)

**Figure 7-7: Energy loss by heat transfer through different panels**

### 7.2.5.3 Energy saves

The energy loss includes two phases: absorption by the panel and heat transmittance.

\[
Q_{\text{Total}} = Q_{\text{Thermal Mass}} + Q_{\text{Heat Transmittance}} \quad (7.7)
\]

Table 7.4 shows the total amount of energy lost by five panels during 24 hours. CFRP panel is three times less than the concrete and stone block; also, it is two times less than the brick block. The CFRP panel with Polyethylene foam can save much energy, as it had the smallest rates of energy absorption and heat transmittance loss.

**Table 7.4: Total Energy loss by different panels per day**

<table>
<thead>
<tr>
<th></th>
<th>Brick</th>
<th>Concrete</th>
<th>Stone</th>
<th>CFRP</th>
<th>CFRP with Polyethylene foam</th>
</tr>
</thead>
<tbody>
<tr>
<td>Energy used MJ/Day</td>
<td>25.0</td>
<td>43.3</td>
<td>40.4</td>
<td>13.7</td>
<td>1.1</td>
</tr>
</tbody>
</table>

### 7.2.6 Conclusion

CFRP panels, when compared to traditional floors, have significant advantages for energy saving through both energy absorption and heat transmittance. CFRP is a good insulation
material for use in buildings when compared to the traditional building materials, such as concrete, brick, and stone. From the analysis and results shown above, it can be seen that the CFRP material with polyethylene foam can improve thermal properties dramatically. It can save 20 to 40 times the amount of energy per day as compared to the traditional building materials.

7.3 Fire safety engineering and design

7.3.1 Introduction

By referring to Monuritz and Gibson (2006) good heating performance and dullness in burning are the traits of composites. Composites’ rate of thermal conduction is far lower than that of metals, which is a great strength to prevent the fire from spreading from one room to another. For fires, flame, heat, and poisonous smokes, composites serve as a good barrier. Thus, composites are good candidates for sheltering heat in re-entry spacecraft and rockets. High fire risk applications are also being promoted for protection from fire, for example, the offshore oil platforms (Mouritz & Gibson, 2006). However, the damage can be permanent once the CFRP panel system is destroyed by fire. For the whole design of the panel system, fire protection plays an indispensable role. It should be noted that when the temperature reaches 120°C, the performance of FRP may not be as good as in normal conditions. In order to resolve this issue, this part will focus on the general fire performance, design of fire codes, and design of fire protection.

7.3.2 Building fire codes

Building codes are regulations and disciplines that limit the design and building of the construction environment. Fire code is a unique kind of construction code, setting a minimum standard of fire security.

Generally speaking, two kinds of codes exist (Cote, 1997): prescriptive and performance-based. Prescriptive codes clarify the accurate instructions of accomplishing fire security for the construction category and utilization. The content covers the employment of materials and products, fabrication means, and the whole construction design. Performance-based building codes are a standard specifying the exact fire security goals and the regulations for
deciding if the performances are satisfying (Cote, 1997). The detailed manners of achieving the goals are not stated clearly.

Prescriptive codes have a long history and are normally supported by schools with an ensured business interest in the building industry. They are often easier to follow with their low requirements on evaluation and analysis as well as for their acceptance of a finite number of choices, but two weak points about this format exist:

- Only general projects, like construction category and usage, can adopt the fire security method. According to these fields, variations of the performance lead to differential rates of real fire safety.
- There is no encouragement in creation. To find a regulation for materials and integration, which are not clearly stated in the code, is almost impossible.

Performance-based codes were set up to address these weak points. They made sure of the fire safety by defining the security targets instead of the accurate steps. By authorized models or standardized exams, new materials accept certification or a rating. Standard test processes are developed and published by organizations like the ISO, ASTM, UL, and DIN. The following fire reaction characteristics were checked during the tests:

- Ignitability (ASTM E2102-04a / ISO 5657:1997)
- Smoke production (ASTM E662-03e1 / ISO 5659-1:1996)

Other tests, such as the ASTM E 119-00a, ISO 834-1. 1999, and EN 1365, are used to decide system-based fire resistance properties. Several fire response and resistance properties can be measured by test procedures like the Single Burning Item (EN 13238) and the Room Corner Test (ISO 13784) simultaneously. The result of these tests can serve as a reference by the construction code. For instance, all doors that compose part of a fire unit getting an F-90, or 90 minute endurance rating, under ASTM E-119, are required by a stylish performance-based construction code.
7.3.3 Fire codes in the European Union

For European Union members, there has been a unified construction code under research by the European Committee for Standardization (CEN) since the 1970’s. Nowadays, the existing code, which contacts with fire security in the design and building of constructions, is:

Euro Code 1: Measures on Structures: Part 1.2: Steps on Structures Exposed to Fire (European Committee for Standardization, 1991). First published in 1990, the code was recently modified in 2002. Two types of design fires are thought through within the code: prescriptive and parametric. The prescriptive design fire is employed in the normative part of the code and relates to the curves of time and temperature supplied by the ISO 834 standard (ISO, 1975). A performance-based design approach is innovated by the parametric part of the code. Instead of adopting model curves of time and temperature, naturalistic fire situations can employ a selection of easy or advanced fire standards.

The necessary functions of construction parts are indicated by the performance. The identification “R” indicates delay of structural duration, “E” indicates remain of the combination of the parts, and “I” indicates remain of heating circulation. Coherently, a number, in multiples of 30, follows which indicates the minimum durability that these properties are maintained when attributed to fire situation. For instance, walls with both burden-carrying and form component of a fire structure can achieve a rating of REI30.

The purpose of the code is to limit fire spread and to conserve structural resistance, mainly via the employment of passive measures in fire security. Demands for active ways and, specifically, fire sprinklers are postponed to national or local standards (Gulvanessian & Menzies, 2000). In resolving the use of FRP materials within the EU, P. Briggs (Briggs, 2003) put forward a deeper review of the performance of fire codes.

The Norme de Protection Incendie, published by the Association of Cantonal Fire Insurance Establishments 10 (VFK/AEAI, 1993), gives a definition for a lot of the specific fire security demands of SIA 183. The fire resistance demands are prescribed for burden-carrying parts within this code. The summary of these requirements, classified by the height of the construction are:
• Single-story constructions: no demands
• Two-story constructions: 30 or 60 minutes, influenced by the building size, usage, etc.
• Three-story constructions: 60 minutes
• More than three stories: 90 minutes

Requirements for endurance time are cut to 30 minutes in all situations for constructions set up with fire sprinklers since it is assumed that the sprinklers will rapidly repress most fires.

7.3.4 Solution methods

Generally, a compound of passive and active safety methods should be involved in proper fire safety. The usage of the passive measures is to prevent the ignition of fires and to change the influence of fires via mechanisms which ask for no manual steps or automotive reaction. A complete physical reaction, by men or automotive procedure, is part of the active measures. Both kinds of methods are carried out via strategies illustrated in Fig. 7-8. For instance, the employment of circuit crackers or wires in an electrical compartment is an active method, which automatically explores a short circuit and prevents electricity flow, and, therefore, drops in the tactic “Manage Heat-Energy Sources”. Passive measures include the adoption of non-ignitable fuse insulation materials and a metal vessel, which tries to manage fuel. By adopting both methods, their joint effects largely reduce the chance of short circuits, leading to fire risks of a construction. The following sections will state the active and passive measures which are suitable for FRP construction parts.

![Figure 7-8: Methods for achieving fire safety objectives (Cote, 1997)](image-url)
7.3.5 Fire behaviours of FRP materials

During the FRP material burning, there is a process involving four stages, as illustrated in Fig. 7-9 (Hilado, 1990):

1. **Heating:** Energy is transited to the rigid polymer to change it from the surrounding temperature to the Td temperature at which it starts to degrade in a chemical way.

2. **Decomposition:** More fuel is required in this phase to crack the covalent combination of the organic composites and to cut it to its degraded parts: rigid remains (char, ash), partly decayed polymer, entrained substances (smoke), inflammable gasses, and flammable gasses (which are produced from the free radicals).

3. **Ignition:** No time dimension exists in this phase; it is the moment when all facades of the fire tetrahedron encounter and ignition starts.

4. **Combustion:** In this ultimate step, the energy needed for more of the rigid polymer to decay is provided by the exothermic response between the ignitable gases and the oxidizing agent. When the fire generates more heat, more decay happens, innovating more energy which enables further burning, and, therefore, the procedure becomes self-propagating. There are two kinds of combustion: flaming as “fire”, and non-flaming as the ignition of a cigarette. The ratio of the amount of fuel manufactured by burning to the amount of fuel needed for decay greatly impacts the burning rate and flame spread.

Figure 7-9: Mechanisms involved in the thermal decomposition of polymer composites, showing feedback loops involving heat flux (Mouritz & Gibson, 2006)
7.3.6 Post fire properties

The results of temperature and resistance of a fire on the remaining mechanical traits of epoxy composites has been researched by Pering and partners (1989), Mouritz (2006) and Seggewiss (2003). Due to the high ignitability of the epoxy matrix, when the temperature is increased or fire is heated in time, the post-fire characteristics can disappear abruptly, which is a confusing issue. Fig. 7-10 illustrates the great decrease to the after-fire tensile strength of an epoxy or carbon platelet, having come into exposure with medium to high temperature smoke fires, 540 to 980°C, in a short period. Pering et al. assumed the reason for this decrease was the speeded heating decomposition of the epoxy matrix. Likewise, Mouritz (2002) found great decrease in the after-fire characteristics of epoxy and carbon composites, and the main cause of this abrupt decrease in performance is the rapid degradation.

![Figure 7-10: Effects of temperature and time of a gas fire on the post-fire tensile strength of a 16-ply (1.9 mm thick) carbon/epoxy laminate (Pering, Farrell & Springer, 1989)](image)

As illustrated in Fig. 7-11 of the Elastic Modulus curve, when the FRP materials are heated beyond their normal temperatures (some 120°C), their mechanical characteristics will change suddenly. However, different variations will happen to the steel and concrete. The advantage of steel is a decrease of liner. At 350°C, a cross spot exists between CFRP and steel because the strength of both decreased to 60%. However, the strength ratio of CFRP from this view is lower than steel. Good fire resistance is embraced by concrete. From 0°C to 45°C, no change will occur to the strength of concrete, unless the temperature gets to 450°C. At that temperature and above, the strength begins to decrease, and the change is linear.
7.3.7 Fire resistance

The strength the FRP material will decrease when the temperature reaches 120°C. It is important to protect the CFRP panel against the fire. A fire’s temperature can go up to 2000°C. Therefore, construction materials such as brick, CFRP, steel, and wood cannot withstand that temperature. All of these materials under fire conditions will be failed. Therefore, the fire protection measures need to be applied to all kind of building materials, not only for CFRP. There are two kinds of insulation materials mentioned in previous sections: polystyrene foam and Rockwool. It can be seen from Fig. 7-12 that polystyrene foam melts at 500°C. However, Rockwool melts at 1100°C which is more than twice the fire resistance of polystyrene foam. Therefore, Rockwool has lower thermal conductivity and higher fire resistance than the polystyrene foam. The Rockwool was considered as an insulation material in this project.
7.3.8 Fire safety engineering design of proposed FRP floor System

According to Katoh, Ishida, and Ogasawara (2009), the maximum operation temperature for CFRP material should be around 120°C in order to keep the structure safe. There are several fire protections integrated into the panel system. First of all, if the building catches fire, the raised floor panels, suspended ceilings, and insulation stop the heat from being directly transferred to the CFRP panel. Another fire protection is the sprinkler system shown in Fig. 7-13. Sprinkler activation will do less damage than a fire department hose stream, which provides approximately 900 litres per min. In addition, a sprinkler will usually activate between one and four minutes. The heat has a hard time going through the material. In this case, the heat goes through the raised floor panel or ceiling first and then goes through the insulation materials. Finally, the heat reaches the panel system. In this process, when the heat reaches the CFRP panel, it reduces at a limited level. It would not burn through the CFRP panels because the raised floor panel and ceiling are designed with very good insulation materials. This can be seen in Figure 7-13.

Figure 7-13: Fire protection for CFRP panel system

7.3.9 Modeling of fire behaviour

In order to simplify the model, there are two assumptions made in this investigation. The first one is to assume that the raised floor and suspended ceiling have no fire resistant capabilities. The other one is to assume that the Rockwool in 120 minutes time and under 1000°C does not melt (Rockwool, n.d). The model has 80mm thickness with 400 thermal plane field elements. The total area of the modeling panel is 1 m². The property of the material is shown in Table 7.1. The modeling will discover the optimized thickness which can resist the 1000°C fire
temperature for 120 minutes. Figure 7-14 shows the FEA modeling results. Temperature at the bottom surface was given at 1000°C as a fire temperature. The temperature at the top surface was initially given a temperature of 25°C. After 120 minutes, the contour shows that the top surface temperature changed from 25°C to 103.5°C. This modeling shows that the Rockwool has very good fire resistant properties.

![Figure 7-14: PHI contour of Rockwool insulation, with 80mm thickness under 1000°C, after 120mm burning analysis](image)

Table 7.5: Temperature against Rockwool thickness

<table>
<thead>
<tr>
<th>Thickness</th>
<th>50mm</th>
<th>60mm</th>
<th>70mm</th>
<th>80mm</th>
<th>90mm</th>
<th>100mm</th>
</tr>
</thead>
<tbody>
<tr>
<td>Temperature °C (after 120 minutes)</td>
<td>144</td>
<td>127</td>
<td>114</td>
<td>103</td>
<td>95</td>
<td>89</td>
</tr>
</tbody>
</table>

Considering that the maximum operation temperature for CFRP materials is around 120°C, the thickness must be designed to control the temperature to under 120°C at the inner surface of the Rockwool board connected with CFRP panels to make sure that the panel can still handle loading capacity. Table 7.5 shows different temperatures using different thicknesses of Rockwool after a two hour fire simulation. The minimum thickness is 70mm, and the 80mm thickness is proposed to be adequate for fire protection in this project because this thickness can produce close to 20% safety margins.
7.4 Conclusion

The CFRP material has a low thermal conductivity which is a benefit for energy saving and the environment. However, the maximum operation temperature for CFRP material is around 120°C, so the fire protection layers, such as raised floor, suspended ceiling, and insulation boards, must be used to satisfy the criteria of fire safety engineering design. The Rockwool has better insulation and fire resistant properties than the other insulation materials. A minimum thickness of 80mm is proposed for this insulation board according to the simulation.
INSTALLATION AND ASSEMBLY
8.1 Introduction

This chapter presents the procedure for installation and assembly of the investigated FRP composite floor system. Also, the detail of the architectural function and mechanical service are described.

8.2 Industrial fabrication

All components are required to be premade, including CFRP slabs and the additional elements needed for locating composite slabs on determined positions. The CFRP slab is made by economical pultrusion which produces high quality products. Regarding the equipment required by pultrusion, the steel die needs to be manufactured for shaping the CFRP slabs. This die can be designed based upon the one used for scaled floor panels given in Chapter 5. It is proposed that the position locaters be made from plastics, and they will be economic products purchased from market.

8.3 On-Site installation and assembly

This section aims to provide a simple procedure. It shows how to install and operate a CFRP floor panel system in the field of construction. Because all floor slabs are CFRP panels, they can be handled throughout the installation process by one or two persons. The following proposed construction sequence for the CFRP floor panel system assumes that the foundation system and the concrete or still frames already exist and are in place at the time of erection.

Step One

First, a number of the position locaters are laid facing up at the positions planned on beams from the design. The number of locaters must be consistent with the number of floor panels required in terms of the designed size of the building. All locaters will be adhesively bonded onto the beams. Araldite rapid set series is of one adhesive product which is a popular engineering epoxy adhesive. The position locaters, shown in Fig. 8-1, will help with the installation and assembly of the CFRP panels.
Chapter-8: Installation and Assembly

Figure 8-1: Position locaters on a steel I beam

Step Two

After bonding the position locaters, the CFRP slabs can be laid on top of the beams, as shown in Fig. 8-2. First, the epoxy adhesive will be pasted in the area between two locaters on the top of the beam. Then, each slab will be laid at the position determined by the two locaters. Attention should be paid to the side pins for connection when placing on the next CFRP panel, and be sure the two panels are placed properly at up and down connectional pins. Heat curing can be applied to make the adhesive set more quickly. This step will complete the installation of the required CFRP floor slabs as an assembly of a primary floor structure.

Figure 8-2: CFRP slabs installation

Step Three

After installation and assembly of CFRP panels, the raised floor system can be installed. First, the plastic supports and aluminium frame will be placed above the CFRP floor panels.
Then, the raised floor panels with carpets or other type of floor finish will be dropped into the aluminium frame. This can be seen in Fig. 8-3.

Figure 8-3: raised floor installation

*Step Four*

The suspended ceiling will be installed at the same time as the raised floor system. The suspended cable will be installed first and followed by the aluminium frame. Installation of ceiling panels will begin after the installation of mechanical services, such as ventilation, sprinklers, and lighting systems. Figure 8-4 shows a proposed suspending ceiling and underneath are the CFRP floor panels.

Figure: 8-4: Suspended ceiling installation
**Step Five**

Finally, the mechanical services, including sprinkler system, ventilation systems, lighting systems, etc., will be installed. These are schematically shown in figure 8-5.

![Figure 8-5: Final assembled CFRP floor system](image)

### 8.4 Architectural function

#### 8.4.1 Raised floor

The raised floor system consists of a gridded metal framework, or substructure, of adjustable-height supports (called "pedestals") that provide support for removable floor panels, which are usually 60×60 cm in size. The height of the legs/pedestals is dictated by the volume of cables and the services provided beneath but are typically arranged for a clearance of at least 40 mm or 60mm (BS EN 12825). The panels are normally made from steel-clad particleboard or a steel panel, but some tiles have hollow cores. Panels may be covered with a variety of flooring finishes to suit the application, such as carpet tiles, high-pressure laminates, marble, stone, and antistatic finishes, for use in computer rooms and laboratories. The main function for the raised floor is its practicality, as the underlying systems, can be accessed and repaired without affecting the building works, providing the advantages in terms of management times and cost. Another important function of the raised floor system is protecting the CFRP floor panels from impact by the heavy drops.

#### 8.4.2 Insulation and fire resistance

According to Corus (Corus, 2006), there are five main methods and equipment choices recommended for building fire protection: sprinklers, section factor and protection thickness
incensement, site applied protection materials, off-site fire protection, and structural material fire resistance. In this project, sprinkler systems were selected for application. Structural material fire resistance cannot be applied because significantly improving the resin’s fire property, in CFRP, is very expensive. Instead, fire resistant coating can be considered as an option in practical application. The best coating material is Rockwool because the features of Rockwool include high fire resistance and a lightweight property, allowing it to be used underneath and above CFRP panels to improve the fire resistance of the entire CFRP floor system. Rockwool panels are also excellent materials in preventing acoustic effects, and recommended to be used in construction. (ROCKWOOL, n.d).

8.4.3 Suspended Ceiling

A typical suspended ceiling consists of a grid-work of metal channels in the shape of an upside-down "T", as shown in Figure 8-5, suspended on wires from the overhead structure. These channels snap together in a regularly spaced pattern – typically a 600×600 mm grid in Europe (BS EN 13964) (BS 8290-2:1991). This is the modular size of the grid, the tiles are actually 595mm x 595mm or 595mm x 1195mm. Each cell will be filled with lightweight "tiles" (Orme, et al. 2001) or "panels" which simply drop into the grid. Tiles can be selected from a variety of materials, including wood, metal, plastic, or mineral fibres, and can come in almost any color. Light fixtures, HVAC air grilles, and other fixtures are available, which can fit the same space as a tile for easy installation. Most tile materials are easily cut to allow fixtures in other shapes, such as incandescent lights, speakers, and fire sprinkler heads.

8.5 Mechanical services

Mechanical services include ventilation sprinklers and lighting systems. They will become part of the suspended ceiling system.

8.5.1 Ventilating service

Before installing the ceiling panels, the ventilation system needs to be installed. Ventilation is the process of bringing outdoor air into a building, circulating it, and later purging it into the environment (Bearg, 2001). The purpose of ventilation is to provide acceptable indoor air
quality by diluting and removing contaminants from the indoor air (Bearg, 2001) (Orme, et al. 2001) by natural or mechanical means (Orme, et al. 2001). Most commercial buildings, such as offices, schools, and hospitals, use mechanical ventilation, which is more controllable and responsive than natural ventilation in providing adequate indoor quality. This includes heating, ventilation, and air conditioning (HVAC) systems. These are schematically shown in Figure 8-6.

8.5.2 Sprinkler system

Fire sprinkler systems vary with different buildings or properties. Many different types of fire sprinklers for commercial buildings have been developed over the years. These sprinklers include wet, dry, deluge, pre-action, and foam (Fisher, n.d). The most commonly used system in commercial buildings is a wet pipe system, which is composed of steel pipes that are always filled with water. The sprinkler heads fit into the suspended ceiling panels.
8.5.3 Lighting system

Another benefit of the suspended ceiling is that the large overhead space can be used for the installation of the lighting system. According to HSE, every workplace needs to have suitable and sufficient lighting (HSE, 1998). Figure 8-7 shows four kinds of typical office lighting. All of these are aluminium frames installed with lamps. The lighting can also fit into the suspended ceiling panels in the CFRP floor system.

Figure 8-7: Typical office lighting systems
CONCLUSION AND FUTURE WORK
9.1 Overview

Modern buildings can benefit from better energy efficiency, lower maintenance, higher quality industrial manufacturing techniques, and quicker construction through the use of fibre-reinforced polymers. Thanks to their high specific strength, low thermal conductivity, good environmental resistance, and their ability to be formed into complex shapes, FRP materials are well-suited to fulfilling many building requirements. By integrating several layers into single function-integrated components and industrially fabricating those components, the amount of on-site labour can be greatly reduced. As such, CFRP materials have a strong potential for fuelling the next great advance in the conception of buildings.

These materials, however, are also relatively expensive, combustible, have low operating temperatures, and are generally less stiff than traditional building materials. In order to overcome these weaknesses in the development of optimised applications, their advantageous characteristics (high strength-to-weight ratio, good environmental resistance, low thermal conductivity, facilitates part-count reduction) must be fully exploited. This philosophy of material-adapted usage constitutes the logical foundation of the project.

9.2 Proposed CFRP floor panel system

The ultimate objective of the project was to develop concepts for a CFRP floor panel system in buildings for which CFRP materials are used in a material-adapted manner. Through a review of significant building projects involving the use of CFRP materials throughout history, there is a lack of data about CFRP components applied to buildings. Most of the FRP building has poor function integration. Also, there is a lack of thermal behaviour research into quality thermal behaviour material. Another problem is the need for a solution regarding poor fire safety issues. Therefore, a new building system was conceived with a strong focus on resolving this issue. A CFRP panel can be protected by the Rockwool insulation board, and water sprinklers. In these systems, CFRP components can draw heat away from load-bearing CFRP components and thus prolong their endurance in a fire.

Therefore, the investigation into an advanced CFRP building floor panel system is presented in this thesis. The design of the proposed CFRP floor panel was successfully completed, using Eurocodes. FEA analysis was used and had an important role in supporting the
conceptual design. Scaled floor CFRP panel specimens were manufactured and tested. Experimental work successfully validated the initial design and proved corresponding FEA models. The proposed CFRP floor panel was verified, with a large safety margin. Design load is only about one fourth of that predicted, and tested highly in failure loads from both FEA and scaled floor panel samples. A scaling effect was given consideration in the use of the full CFRP floor panel. With the scaling effect, the material strength was reduced; however, the designed CFRP panel still fully passed the Hashin criteria check. The investigated CFRP floor panel is recommended as a standard construction component in modern buildings, considering the long term benefits in energy saving and CO$_2$ emissions reduction. The proposed CFRP panel system can be applied in large buildings, such as offices, hospitals, schools, and also in small buildings, such as residential houses. In the practical application of investigating CFRP floor panels, for buildings with different sizes, deflection checking is one of the main design tasks.

Besides using the FRP floor panels as primary floor structures, a raised floor system and a suspended ceiling system were the principle designs. The raised floor system was designed to aid the installation of cables and insulation materials. Also, it would be a good protection against the impact of heavy items accidentally dropped. The suspended ceiling system was designed to aid the installation of insulation materials and ventilation etc.

Finally, this thesis gives guidance on how to install and assemble a whole FRP floor panel system in buildings, including step by step instructions for installing position locators, individual FRP panels, assembling floor panel systems, raised floor systems, suspended ceiling systems, insulation coating materials and equipment required by fire safety engineering design. Therefore, the investigation of FRP floor panel systems, within the scope of this PhD project, has been completed. The proposed FRP floor panel is recommended as a standard FRP slab in the practical application of constructing new buildings, and reconstructing existing buildings.

9.3 Experimental investigation

The work in this chapter includes the design, manufacture and testing of scaled floor panels. All tested results have been compared with the FEA prediction. Meanwhile, statistical methods were used to find out the mean and standard deviation of the maximum deflection
and failure load. Compared to the loading capacity predicted by FEA, the design load of the full floor panel is much smaller, a quarter of predicted loading capacity.

**Shear effect**

As the proposed FRP floor panel is a pultruded beam with an open cross section, a closed form, to calculate the deflection due to bending and shear, was produced. The shear effect of the thin walled open cross on deflection, was theoretically derived. A geometrically based form factor and a load-deflection correction factor involved in the calculation of shear related deflection was proposed and validated by experimental data on scaled floor panel specimens. This shear effect on the deflection of full CFRP floor panel was identified by FEA modeling in this investigation. Theoretical deflections of proposed panels with simple supports, using conducted form factor 2.5 for the open cross-section consisting of multi cells and thin-walled plates, and the load-deflection correction factor 1.8 for the UDL, and 4.3 for point load case, agree with FEA modeling predictions well. This proved that the form factor and the load-deflection correction factors derived from scaled samples are suitable in the calculation of the deflection of the proposed full panel.

**Scale effect**

A scale effect is investigated from a small test panel to a full panel. Experimental tests and modeling work in this investigation show that the weakest-link theory worked very well in the investigation of the scaling effect in composites. The conducted scale effect was amended by a reduction factor of 0.625 (37.5% reduction) for the material strength of the full CFRP panel. This is the scale effect used for the full CFRP panel in the Hashin criteria check, and the final strength check still passed.

**9.4 Design curves**

A group of design curves were produced by this investigation for the quick working out of maximum deflections and critical stresses in transverse and through thickness directions, against the span, width and height of CFRP floor panels. The proposed design curves can be used for designing buildings with different sizes, and have been briefly published in a journal paper by Y Gao, J Chen, Z Zhang and D Fox, “An advanced FRP floor panel system in buildings”, Composites Structures, 96, 683–690, 2013.
Buckling and free vibration analyses were carried out by FEA modeling. The first buckling load factor is 8.4 times that of the design load, which means there are no buckling failures within the values of the design load. The first predicted frequency for vibration is 25.1Hz, thus the corresponding peak acceleration ratio is 0.002%, which is much lower than the acceleration limit for buildings required by BS 6472.

This project also carried out an investigation into the thermal behaviour of general FRP panels in building construction. It has been proved by FEA simulations that FRP panels can be good insulation materials. In general, buildings with FRP components can save heating energy by up to 50%, and reduce CO₂ emissions by 40%, when compared with traditional construction materials.

Fire safety engineering design is also addressed in this investigation, according to Eurocodes. Practically, although CFRP is a good insulation material, the CFPR panels must be used together with the proposed coating materials both underneath and above as they have much higher fire resistance, to ensure that the application meets the fire safety engineering standard. FEA modeling shows that the minimum thickness of the Rockwool is 70mm, and 80mm thickness is proposed to be adequate for the fire protection of the proposed CFRP floor panel because 80mm thickness can give about a 20% safety margin. Also, other pieces of equipment, e.g. water sprinklers, are suggested to be part of the fire safety engineering design.

9.5 Future work

Further investigation will include the following work:

1. Experimental tests of full CFRP floor panels are necessary to verify the feasibility of the proposed system.
2. Practical installation of the proposed CFRP panels in buildings need a further design detail.
3. Further investigation into the mechanical connection between CFRP panels and beams, such as mechanical clamp joint.
4. Investigation into the shear effects of a CFRP floor panel with fixed supports.
5. Impact tests with the drop weight on CFRP floors need to carry out to investigate.
6. Acoustic investigation for the buildings with CFRP panels, rock-wool and other construction materials.
7. Fire tests on buildings with CFRP floors in terms of the Eurocode.
8. Thermal behaviour tests on buildings with CFRP components to get the accurate heat energy saving and work out the CO₂ reduction amount.
9. Investigation into earthquake responses of buildings with CFRP components is necessary. The investigation should include the numerical and experimental analysis.
References:


BS 6472 (1992). Guide to evaluation of human exposure to vibration in buildings (1 Hz to 80 Hz)


BS EN 12825: *Raised Floor Access*

BS EN 13964: *Suspended ceilings - Code of practice for installation and maintenance*


Brooker, O. (2006). *Concrete buildings schemes design manual*


EC2 (1992). *Eurocode 2: Design of Concrete Structure*


products for bridge applications. edited by J. P. Busel and J. D. Lockwood., Market Development Alliance, Harrison, New York, USA.


Appendix:

Provided for non-commercial research and education use. Not for reproduction, distribution or commercial use.

This article appeared in a journal published by Elsevier. The attached copy is furnished to the author for internal non-commercial research and education use, including for instruction at the author's institution and sharing with colleagues.

Other uses, including reproduction and distribution, or selling or licensing copies, or posting to personal, institutional or third party websites are prohibited.

In most cases authors are permitted to post their version of the article (e.g. in Word or Tex form) to their personal website or institutional repository. Authors requiring further information regarding Elsevier’s archiving and manuscript policies are encouraged to visit:

http://www.elsevier.com/copyright
An advanced FRP floor panel system in buildings

Y. Gao, J. Chen, Z. Zhang, D. Fox

School of Civil Engineering and Surveying, University of Portsmouth, Portsmouth PO1 3HE, United Kingdom
School of Engineering, University of Portsmouth, Portsmouth PO1 3HE, United Kingdom

Abstract

This paper presents the design, modelling analysis and experimental verification of a carbon fibre reinforced polymer (CFRP) floor panel system in buildings. The CFRP panel was designed as a pultruded beam with an open cross-section. The design was carried out using the Eurocodes and supported by the finite element analysis (FEA). Modelling results indicated that the proposed floor panel passed all design checks recommended by Eurocode, and the safety check on both deflection and material strength are important for producing CFRP panel products to meet the dimensional requirements in design of buildings with different design specification. Experimental results of scaled CFRP panel samples were also presented in this paper, which successfully validated the design and modelling analysis. This proposed CFRP floor panel as a primary component in the floor system aims to replace traditional concrete floor slab in buildings. This new CFRP floor panel can be easily installed in buildings because of its lightweight feature, and easily integrated with the suspending ceiling ventilation and lighting system because of designed shape. This investigation provided a potential building component in the low carbon construction industry for saving energy and reducing CO₂ emission.

Crown Copyright © 2012 Published by Elsevier Ltd. All rights reserved.

1. Introduction

CFRP material has been used in many industries to build up primary structures because of its features such as high strength, lightweight and durability. CFRP also has another outstanding feature of low thermal conductivity. This brings an application of CFRP in civil engineering for making significant contribution in a low carbon construction industry. In the last decade, apart from the application of CFRP as a repairing material for strengthening damaged structures, a number of CFRP bridges were built up in the world; CFRP residual houses have been building in Europe; CFRP pier has also been building recently in USA, etc.

For speeding up the development of a low carbon construction industry, structural FRP components such as pultruded CFRP beams and composite sandwich decks have been applying in bridge engineering [1]. In building construction, the feature of low thermal conductivity enables FRP lead-carrying components act as insulating materials, in addition to their structural function. A full glass fibre reinforced polymer (GFRP) house was manufactured by Starlink in the UK, 2011 [11]. Comparing to traditional houses, GFRP house could be economic, thermally efficient and sustainable by accounting the long-term benefits. The existing or current GFRP houses were designed with short span, single story or two stories for residents. However, the CFRP houses are restrained in application because of the fire resistance of GFRP material. In principle, it is feasible to construct large buildings with CFRP materials, because the CFRP has much higher strength and better thermal conductivity than GFRP. The potential applications of CFRP constructional components can be used for schools, offices, hospital, etc. An advanced CFRP floor panel system for large buildings was investigated in this paper. A design approach and techniques for producing a new CFRP floor panel to replace traditional heavy and thick concrete slab were investigated. The economic benefits of buildings with CFRP components can be obtained from energy saving, and saving from foundation, installation, transportation, etc. The environmental benefits will be gained significantly through CO₂ emission reduction from manufacturing CFRP materials and heating buildings with CFRP components. Therefore, the CFRP floor panel should be one of the novel building materials bring long-term benefits to construction industry.

The benefits from the application of CFRP floor system include the following aspects: (1) The self weight of a CFRP deck is only about 8–10% of a reinforced concrete slab. Therefore, replacing a concrete slab by a CFRP floor panel reduces the dead load significantly. Table 1 shows the properties of the carbon fibres and epoxy polymers [10,16]. These basic material data were supplied by Gurit [10]. A light dead load can save materials in structures, and the foundations of new buildings should be also saved. (2) Rapid installation can save the labour thus construction would be economic. Pultruded FRP panels offer the advantage over cast-in-situ concrete
Appendix

Table 1
Material properties.

<table>
<thead>
<tr>
<th>Property</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>CFRP (W/EMI HSC 100)</td>
<td></td>
</tr>
<tr>
<td>Elastic modulus</td>
<td></td>
</tr>
<tr>
<td>$E_1$</td>
<td>130.33 GPa</td>
</tr>
<tr>
<td>$E_2$</td>
<td>7.22 GPa</td>
</tr>
<tr>
<td>$E_3$</td>
<td>7.22 GPa</td>
</tr>
<tr>
<td>Shear modulus</td>
<td></td>
</tr>
<tr>
<td>$G_{12}$</td>
<td>4.23 GPa</td>
</tr>
<tr>
<td>$G_{13}$</td>
<td>4.23 GPa</td>
</tr>
<tr>
<td>$G_{23}$</td>
<td>3.59 GPa</td>
</tr>
<tr>
<td>Poisson ratio (XY)</td>
<td>ν_{12}</td>
</tr>
<tr>
<td>Poisson ratio (XZ)</td>
<td>ν_{13}</td>
</tr>
<tr>
<td>Poisson ratio (YZ)</td>
<td>ν_{23}</td>
</tr>
<tr>
<td>Mass density</td>
<td>ρ</td>
</tr>
<tr>
<td>Longitudinal tensile strength</td>
<td>$X_L$</td>
</tr>
<tr>
<td>Longitudinal compressive strength</td>
<td>$X_C$</td>
</tr>
<tr>
<td>Transverse tensile strength</td>
<td>$Y_T$</td>
</tr>
<tr>
<td>Transverse compressive strength</td>
<td>$Y_C$</td>
</tr>
<tr>
<td>In-plane shear strength</td>
<td>$S_{12}$</td>
</tr>
<tr>
<td>Adhesive (Araldite 2015)</td>
<td></td>
</tr>
<tr>
<td>Young's modulus</td>
<td>$E$</td>
</tr>
<tr>
<td>Shear modulus</td>
<td>$G$</td>
</tr>
<tr>
<td>Poisson ratio</td>
<td>ν</td>
</tr>
<tr>
<td>Tensile strength</td>
<td>$σ$</td>
</tr>
</tbody>
</table>

Slabs, e.g., the quality of products can be properly monitored in the concreted factory environment. (3) High strengths and stiffness of carbon materials bring CFRP panel a high safety factor. (4) Significant energy saving can be presented by the following points. Large buildings with CFRP floor panels are less heating energy demanding in the winter, and corresponding CO2 emission will be reduced [1]. Ready Homes Enterprises, USA (2002) summarised that the FRP comes house could save 50-70% heating energy compared to traditional house, and turning down the emission of CO2 by 50% at least [15]. The quantities of raw materials and energy required to construct the foundation of buildings with CFRP floor system would be a small fraction of that required in conventional buildings. (5) Building construction is a complicated but often wasteful process, waste and off-cuts from materials used on site are seldom recycled [11]. However, pre-pultruded CFRP floor components can certainly cut the waste down. Previous investigator Keoleian et al. [12] indicated that considering FRP bridge decks over a 50-year service life, the FRP deck could reduce the cost of the concrete deck by 7%, consume less total primary energy by 40%, and reduce the emission of carbon dioxide by 30%. Considering the total benefits from all aspects e.g. heating energy savings [13] and corresponding CO2 emission reduction, less foundation, quick installation and economic transportation, and free maintenance cost [11], etc., the problem of initially higher material costs (estimated double cost than conventional ones [1–3]) of the CFRP components should then be economically justified.

2. Design of CFRP floor panel

The investigated CFRP floor panel was proposed to be made by carbon fibres and epoxy polymers, and to be produced using pultrusion technology. The CFRP floor panel as a proposed standard building component will be pre-manufactured in factories and be installed on site. The proposed CFRP floor panel shown in Fig. 1 was designed as a pre-pultruded one-way spanning beam with an open cross-section, consisting of a top plate and three webs. The fibres are orientated along the length of panel for taking the action against the bending or deflection. The CFRP floor panel was proposed to be supported by steel or concrete beams, and be connected on the beams by either adhesive joints or mechanical joints with assistance from the longitudinal stoppers for locating the panel. This can be seen from Fig. 2 which shows schematically the adhesive connection between a CFRP panel and a supporting steel beam. The overall size of the investigated basic floor panel is 6 m in length, 0.5 m in width, and 0.13 m in height. The designed CFRP panel has two-cell rectangular open cellular-sections, which give two advantages: (i) Integration with heating, cooling, fire sprinkler, ventilation and lighting systems by a suspended ceiling system. (ii) Low production costs by pultrusion. Fig. 1 also shows the cross-section view of CFRP floor panel with varied thickness. The feature of cross-section is varied inner arc. A small inner arc was given at each corner and large inner arc was given at the middle of each segment of cross-section. This design aims to reduce the stress concentration at corners. Two short flanges at each side of panel were designed for overlapping panels through a simple adhesive connection to achieve structural integrity.

However, the FRP composite material has two major disadvantages when they are used in civil engineering structures. Firstly, FRP material is brittle and its impact resistance is low. Secondly, the fire resistance of FRP is relatively low. However, these problems can be solved by installation of secondary structures or systems, the raised floor and suspending ceiling systems. The raised floor system was designed to sit on the top of the CFRP panels shown in Fig. 3. It consists of gridded aluminium frames that provide a support for removable square, which could be 50 x 50 cm² timber or polymer plates. The height of the legs/pedestals can be adjusted in term of the size of cables and other services required underneath, but typically arranged for a clearance of height at least 15 cm. The raised floor system can provide some benefits such as reducing vibration movement and impact effect. The room between the CFRP floor panels and raised systems can be filled with anti-fire foams for improving fire performance.

The proposed CFRP panel with an open cross-section enables the installation of suspend ceiling system shown in Fig. 3, which can integrate with lighting, sprinkler and ventilation systems together. Ceiling tiles is proposed to be made by mineral fibres or fire-rated wood panels. This aims to meet an acceptable level of fire engineering standards/ratings. These tiles can also provide additional resistance to satisfy the “time rating” required for various fire engineering codes, and to improve the fire performance of buildings with CFRP floor panels.

3. Structural analysis

The design of FRP floor panel was supported by the finite element analysis. The self weight of the floor panel, raised floor, services (heating, ventilation and cable trunk) and suspended ceiling were considered together as a dead load. The volume 0.06593 m³ of the designed CFRP panel was worked out by designing tool AUTOCAD. The CFRP density 1502 kg/m³ was taken from the data base of composites supplied by Gurit [10]. Thus surface load from the panel self-weight was calculated as 0.312 kN/m². Apart from the panel self-weight, the total dead load accounted the weight

Fig. 1. A CFRP floor panel and a cross-section view.
Appendix

Fig. 2. A CFRP floor panel connected by adhesive bonding joints on beams.

Fig. 3. A CFRP floor panel systems with raised floors and suspending ceilings.

Fig. 4. A half FEA model.

Table 2

<table>
<thead>
<tr>
<th>Design criteria</th>
<th>Modelling results</th>
<th>State</th>
</tr>
</thead>
<tbody>
<tr>
<td>Maximum deflection criteria</td>
<td>Max def. = 7.60 mm</td>
<td>Pass</td>
</tr>
<tr>
<td>Indicator</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Height failure criteria</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Longitudinal tension ( fibre tension)</td>
<td>0.010</td>
<td>Pass</td>
</tr>
<tr>
<td>Longitudinal compression ( fibre compression)</td>
<td>0.011</td>
<td>Pass</td>
</tr>
<tr>
<td>Transverse tension (matrix tension)</td>
<td>0.065</td>
<td>Pass</td>
</tr>
<tr>
<td>Transverse compression (matrix compression)</td>
<td>0.080</td>
<td>Pass</td>
</tr>
<tr>
<td>Through thickness tension</td>
<td>0.044</td>
<td>Pass</td>
</tr>
<tr>
<td>Through thickness compression</td>
<td>0.11</td>
<td>Pass</td>
</tr>
</tbody>
</table>

CRFP floor panel. Consider the symmetry of designed CRFP panel, a half model relating to the span was generated and shown in Fig. 4, which consists of 90,000 solid elements in total, the solid element H3XM was used in the mesh of the panel, solid element P76 was used in sharp geometrical area for simulating adhesives. Two different types of material models, isotropic and anisotropic model listed in Table 1 were used in the structural analysis for modelling the adhesives and unidirectional carbon fibre composites in panel respectively. It should be noted that shear modulus and Poisson ratio of unidirectional carbon fibre composites between transverse and through thickness direction given in Table 1 were calculated using the shear energy balanced formulation, which was referred.
Appendix

to that used in one of authors' previous work [17]. A uniformly distributed load obtained from the section of design of floor panel was applied on the top surface of the panel. A fixed condition was applied over the supporting area at the end of panel. A symmetric condition was applied at the middle section of the investigated panel.

Table 2 gives selected computed important results for design check. The maximum deflection was 10.45 mm which is much less than the deflection criteria 24 mm (span/250 recommended by the Eurocode). The deflection contour is shown in Fig. 5. All maximum stresses in different directions are below the individual material strength. A plenty of strength safety margins was predicted. Attention on strength check should be given in the transverse and through thickness directions because of the weak matrix. The maximum transverse tensile stress 24.86 MPa is located at the lower corner of the middle web at the end of the panel, which can be seen from Fig. 6. Predicted maximum tensile stress 24.6 MPa in through thickness direction is quite close to that in transverse direction, and is located at the lower corner of a side web shown in Fig. 7. Hashin failure criteria given by Eqs. (1)-(6) were used for the strength check [18].

Longitudinal tension:

$$\left( \frac{\sigma_{x}}{X_T} \right)^2 + \frac{\sigma_{y}}{Y_T} = 1$$ (1)

Longitudinal compression:

$$\left( \frac{\sigma_{z}}{X_C} \right)^2 = 1$$ (2)

Transverse tension:

$$\left( \frac{\sigma_{x} + \sigma_{y}}{Y_T} \right)^2 + \frac{\sigma_{z}}{Y_T} + \frac{\sigma_{xy}}{S_T} = 1$$ (3)

Transverse compression:
Through thickness tension:

\[
\frac{\sigma_{11}}{\sigma_t} = 1
\]

Through thickness compression:

\[
\frac{\sigma_{11}}{\sigma_t} = 1
\]

where \(\sigma_i (i, j = 1-3)\) denote the stress components in longitudinal direction 1, transverse direction 2 and through thickness direction 3. This material coordinate system was used for each single plate in the proposed floor panel. The tensile and compressive strengths for composites in this application are denoted by subscripts T and C, respectively. \(X_0, Y_0, Z_0\) denote the tensile strengths in three respective material directions. Similarly, \(X_0, Y_0, Z_0\) denote the compressive strengths in three respective material directions. Further, \(S_{12}, S_{13}, \text{ and } S_2\) denote shear strengths in the respective principal material directions. All values of strengths used in this investigation are given in Table 1. It should be noted that strength value in through thickness direction was taken as same as that in transverse direction because of unidirectional composite materials. All shear strengths were simply taken as \(S_{12}\) given in Table 1. Using maximum applied stresses and individual material strengths, design check passed all Hashin failure criteria as shown in Table 2.

4. Dimensional variation in floor panels

The design parameters were included in this investigation to consider the varied dimensions for span, width and height of the proposed floor panel in the buildings with different sizes. The effect of bond length on the prediction for the deflection and critical stresses in transverse and through thickness direction was also investigated. Fig. 8 presents the predicted maximum values for deflection, transverse matrix tensile stress, through thickness tensile stress and in-plane shear stresses against varied span of floor panel because these values are important in the design check considering the deflection criteria and individually directional strength. In Figs. 8-11, the symbol DEF is deflection, DYT is the transverse matrix tensile stress, DTT is the through thickness tensile stress and DXY is the in-plane shear stress. Investigation of varied dimensions was based on a basic model of floor panel with 6 m span, 0.5 m width and 0.13 m height. There was only one design parameter varied in each investigation. In Fig. 8, span varied from 4 m to 8 m, there was no change for width and height. It can be seen from Fig. 8 that the deflection, transverse matrix tensile stress, through thickness tensile stress and shear stress gradually increase when span increases. It should be noted that strength check with Hashin criteria is failed when panel is 6 m span because the maximum tensile stresses in transverse and through thickness direction at the end section of the panel exceed the matrix tensile strength 32.5 MPa given in Table 1. If floor span is expected to be 8 m, the height should be accordingly increased. Variations for DYT and DTT presented by two curves in Fig. 8 are almost same. In Fig. 9, the effect of variation of the height on the behavior of panel is presented. The range of variable height was given from 115 mm to 150 mm. All simulated responses presented by four curves in Fig. 9 decreases when the height increases. The value of 115 mm is the minimum height when span and width are taken as 6 m and 0.5 m respectively. When the height of panel is less than 115 mm, strength check with Hashin criteria is failed because computed stresses for DYT and DTT are over the transverse matrix tensile strength at the same location discussed in above case in which the span varied only. Fig. 10 shows the response of panel with varied width from 300 mm to 700 mm. Response of panel presented by four curves smoothly increase as width increases. However, strength check is failed by Hashin criteria when the width is over 700 mm. Thereason for the failure and failed location are same with that mentioned in above two cases. Finally, the
effect of bonding length between supporting beam and floor: panel was investigated for the case in which adhesive joint is selected for connection. Similar to other investigations on dimensional variation Fig. 11, shows the responses presented by four curves to the variation of bonding length from 30 mm to 150 mm. It can be seen from Fig. 11 that the effect of bonding length on deflection is not significant, four curves generally decrease when bonding length increases. It seemed to be interesting that matrix tensile stress
5. Validation

The basic model of building panels proposed in the section of design of CFRP floor panel was verified by corresponding scaled test samples. Fig. 12 shows the scaled test panel designed proportionally based on the proposed floor panel using a determined scaling factor in all dimensions. Considering the cost in pultruding CFRP panels, scaled test samples were manufactured by bending four open channels on a plate. Some unidirectional carbon fibre composites with that proposed for pultruded floor panels were used in all pieces in scaled samples. Fig. 13 shows the cross-section view of scaled test sample. Verification was carried out by bending test and corresponding shell modelling simulation. Same load and boundary conditions used in the full model of floor panel were applied on the scaled test model. Similar FEA techniques used in the full model were employed for modelling the scaled floor panel. Test results together with modelling simulation are shown in Fig. 14. It can be seen from Fig. 14 that a very good agreement on structural stiffness between test solution and modelling prediction was achieved. Total design load of this scaled panel was calculated as 53 N using a factored design load 0.0106 N/mm² given in the section of structural analysis, which is about 1% of minimum tested final failure (debonding) load 400 N shown in Fig. 14. This indicates that the designed CFRP floor panel has a plenty of safety margin. The failure mode obtained from tested scaled samples is mainly debonding at interface between top plate and side web shown in Fig. 15. This should relate to the bonding quality in manufacturing scaled samples. This also implies that pultruded panels will have higher loading capacity comparing to bonded ones because the transverse matrix strength of pultruded CFRP panel is higher than the adhesive strength of adhesions used for bonding scaled samples. Table 3 presents the states of stress check using Hashin failure criteria. It can be seen from Table 3 that the scaled test panel passed all Hashin failure criteria when using the total design load 53 N. Because the scaled floor panel consists of thin plates, shell elements were employed in the FEA modeling for stress analysis. Modeling predicted the crucial transverse tensile strength 0.2 = 20.35 MPa. Through thickness stress 0.13, both shear stress 0.13 and 0.13 were ignored in this thin shell modeling analysis. Using these stress values together with transverse tensile strength, Eq. (3) can be simplified, and a safe Hashin failure indicator can be calculated by Eq. (7) under the design load 53 N.

\[
\frac{(0.7)^2}{0.43} < 1
\]

Assuming the crucial stress 0.2 reaches the transverse strength 0.2 = 32.5 MPa under an expected initial failure load, within the scope of small deformation, the initial failure load can be estimated by the following equation:

\[
\frac{0.2}{0.2} \text{ Design load} = \frac{0.2}{\text{Initial failure load}}
\]
Appendix

Table 3 Design check for strength of scaled test samples.

<table>
<thead>
<tr>
<th>Hashin failure criteria</th>
<th>Indicator</th>
<th>Result</th>
<th>State</th>
</tr>
</thead>
<tbody>
<tr>
<td>Applied design load 53 N</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Longitudinal tension (fiber tension)</td>
<td>1</td>
<td>0.003</td>
<td>Pass</td>
</tr>
<tr>
<td>Transverse matrix tension</td>
<td>1</td>
<td>0.43</td>
<td>Pass</td>
</tr>
<tr>
<td>Transverse matrix compression</td>
<td>1</td>
<td>0.12</td>
<td>Pass</td>
</tr>
<tr>
<td>Pretensioned failure load 867 N</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Longitudinal tension (fiber tension)</td>
<td>1</td>
<td>0.008</td>
<td>Pass</td>
</tr>
<tr>
<td>Transverse matrix tension</td>
<td>1</td>
<td>0.04</td>
<td>Pass</td>
</tr>
<tr>
<td>Transverse matrix compression</td>
<td>1</td>
<td>0.01</td>
<td>Failure</td>
</tr>
</tbody>
</table>

Bring the value of $\sigma_{zz}$ (21.35 MPa) under the design load 53 N together with $Y$ (32.5 MPa) into Eq. (8) gives an estimated initial failure load 80.7 N for the scaled floor panel. Using the estimated initial failure load, FEA modelling predicted a problem that the transverse matrix stress is in excess of the matrix tensile strength at the end of scaled panel. Detailed Hashin failure check under this estimated initial failure load is given in Table 3. Calculated failure indicator based on Eq. (3) is bigger than 1, which proved the initial failure load 80.7 N. It should be noted that this predicted initial failure load is just about 20% of the final debonding load. Detailed simulation of debonding will be published in different papers in the future.

6. Conclusion and future work

A carbon fibre-based FRP floor panel system was investigated to contribute the development of a low carbon construction industry. Designed floor panel with basic dimensions, 6 m span, 0.15 m height and 0.5 m width, passed all design safety check and was verified by scaled test samples. Proposed pultruded floor panel has relatively weak strength in the transverse matrix and through thickness direction. The weak location is the lower corner of the side web at the end of the panel. However, the actual loading capacity of the proposed floor panel is about eight times of design load. Both span and height of the designed panel played more important roles compared to the width, and they dominated the behaviour of the proposed floor panel. Actually, the height can be down to 0.115 m, but the panel must be restrained within 6 m span. However, suitable dimensions for span and height can be adjusted to meet varied design specification in terms of design curves investigated. Three groups of design curves proposed in this paper can be used for any further designs of floor panels with different dimensions. The failure mode of scaled test panel is debonding at upper corner between the top plate and side web at the end section of panel. This is mostly affected by bonding quality and adhesive strength of adhesives used. Therefore, FRP floor panel is suggested to be manufactured by pultrusion to ensure a better quality of panel products, and should be a pre-pultruded FRP floor panel as one of proposed standard building components in the future similar to the pre-cast concrete system.

Future work includes detailed investigation of failure modes of proposed pultruded FRP panels. Predicting loading capacities of FRP floor panel with varied dimensions; further investigation of connections between floor panel and supporting beams; adjustable modification of FRP floor panel for pultrusion in manufacture; and detailed investigation of fire-resistant performance of FRP floor panel through experimental test and modelling simulation.

References