CONCEPTUAL AND STRUCTURAL DESIGN OF BUILDINGS MADE OF LIGHTWEIGHT AND INFRA-LIGHTWEIGHT CONCRETE

vorgelegt von
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Especially, I would like to give my thanks to my parents and my family whose patient love enabled me to complete this work.

Mohamed El Zareef
ABSTRACT

Some times the need to reduce the weight of a structural element is not less important than increasing its strength, especially in heavy structures such as tall buildings and bridges where the structure’s weight is one of the main problems that faces the designers. In spite of the increasing use and demand of Lightweight Concrete (LWC), the conceptual and structural design aspects for buildings made from LWC and Infra-Lightweight Concrete (ILWC) have not been adequately explained.

Issues such as element dimensions, connections, and reinforcement types and details as well as short- and long-term deformations and dynamic behaviour for LWC structures should be covered in up-to-date codes. Therefore, this study deals with conceptual and structural design of buildings made from LWC and ILWC and generally consists of two main objectives:

- Development and production of new mixtures for LWC and ILWC with minimized dry density and very good mechanical and physical properties.
- The ability to apply and involve these new materials in the construction field through intensive series of experimental tests on different structural elements and connections under static and dynamic loads.

In order to achieve the first objective in the study, two targets were defined, the first: fair-faced ILWC for walls with minimum dry density ($\rho_{\text{min}} < 800 \text{ kg/m}^3$), minimum thermal conductivity enough to eliminate the heat insulation materials, and maximum strength enough to resist the vertical bearing stress from floors. The second target: fair-faced LWC for construction of floor slabs and beams with minimum dry density, minimum thermal conductivity and maximum strength enough to resist flexural and shear stresses comparable to normal concrete (NC).

Once the ILWC and LWC materials were developed and their mechanical and physical properties were determined, a series of large-scale experiments was conducted.

For ILWC, a real application i.e. a one-family house in Berlin, was built in 2006. Because of its favourable physical properties and its good durability, ILWC reinforced with GFR was used for the first time as monolithic cast-in-site concrete to construct the outer walls of this house without any additional insulation [Schlaich M., et. al., 2008]. Infra-lightweight concrete is an engineered high-tech material whose potential and various other design aspects are not yet fully exploited. The study shows the limits of ILWC, but also its great potential for fair-faced concrete buildings.

For LWC, eight beams constructed from the newly developed LWC mixture with concrete strength class of LC 30/33 and reinforced with glass-fibre bars and steel bars, in addition to two control beams constructed from normal concrete C 30/37 and reinforced with steel bars, were tested experimentally for flexural strength capacity, shear strength capacity, ductile behaviour and bond behaviour in tension and compression zones of the beams.

From the economic point of view, using LWC in construction of the floor slabs in tall buildings will reduce the total costs of tall buildings through the reduction of the amount of steel reinforcement, the reduction of foundation volume, and the reduction of vertical members’ cross-sections that saves the used horizontal area.

Because they are the most affected components of tall buildings during earthquake excitations, an experimental study was done to investigate the behaviour of interior and exterior joints between LWC beams and NC columns under seismic loads. The development of highly damage-tolerant beam-column connections would allow structural engineers to design joints for moderate shear distortions which exhibit little damage, reduce rotation demands in beam plastic hinges, and eliminate the need for post-earthquake joint repairs. One option for achieving this goal is to use LWC beams which were reinforced with glass-fibre reinforcement bars with superior deformation capacity in beam-column connections.
KURZFASSUNG


Für Probleme wie Dimensionierung, konstruktive Durchbildung, Bewehrungswahl und Detailausführung, Kriechen und Schwinden sowie das dynamisches Verhalten der Leichtbetonbauwerke ist es zwingend notwendig, dass diese in neuesten Normen aufgenommen werden. Die vorliegende Arbeit beschäftigt sich mit den Entwerfen und Konstruieren von Bauwerken aus Leicht- und Infraleichtbeton und umfasst zwei Schwerpunkte:

- Entwicklung und Herstellung neuer Rezepturen von LB und ILB mit minimierter Trockenrohdichte und sehr guten mechanischen und physikalischen Eigenschaften.
- Eignung dieser Materialien in Bauteilen und Verbindungen anhand von intensiven Testreihen und Experimenten mit statischer und dynamischer Belastung.

Um dem ersten Schwerpunkt zu definieren wurden zwei Ziele festgelegt; zum einen: ein Sichtbeton aus Infraleichtbeton für Wände mit einer Mindesttrockendichte von $\rho_{\text{min}} < 800 \text{ kg/m}^3$, eine Mindestwärmeleitfähigkeit um die Wärmédämmung einsparen zu können und hohe Druckfestigkeiten zum Abtragen der Deckenlasten. Zum anderen: ein Sichtbeton aus LB für Decken, Platten und Balken mit einer Mindesttrockendichte, einer Mindestwärmeleitfähigkeit und einer Druckfestigkeit vergleichbar mit Normalbeton (NB) um Biege- und Querkräfte aufnehmen zu können.


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<tr>
<td>$A_s$</td>
<td>Cross section area of tensile reinforcement bars</td>
</tr>
<tr>
<td>$A_s$'</td>
<td>Cross section area of compression reinforcement bars</td>
</tr>
<tr>
<td>$a_{dry}$</td>
<td>Dry weight of aggregates and additional materials</td>
</tr>
<tr>
<td>$C$</td>
<td>Cement content</td>
</tr>
<tr>
<td>$C_c$</td>
<td>Compression force on concrete</td>
</tr>
<tr>
<td>$C_s$</td>
<td>Compression force on reinforcement bars</td>
</tr>
<tr>
<td>$C_c'$</td>
<td>$C_c + C_s$ for column</td>
</tr>
<tr>
<td>$C_b'$</td>
<td>$C_c + C_s$ for beam</td>
</tr>
<tr>
<td>$^\circ C$</td>
<td>Temperature in grad Celsius</td>
</tr>
<tr>
<td>$d$</td>
<td>Effective depth</td>
</tr>
<tr>
<td>$d_b$</td>
<td>Bar diameter</td>
</tr>
<tr>
<td>$E$</td>
<td>Modulus of elasticity</td>
</tr>
<tr>
<td>$E_{cm}$</td>
<td>Modulus of elasticity for normal concrete</td>
</tr>
<tr>
<td>$E_{lcm}$</td>
<td>Modulus of elasticity for lightweight concrete</td>
</tr>
<tr>
<td>$E_{tot}$</td>
<td>Total energy</td>
</tr>
<tr>
<td>$E_{ela}$</td>
<td>Elastic energy</td>
</tr>
<tr>
<td>$f_{c,\text{max}}$</td>
<td>Maximum compression strength</td>
</tr>
<tr>
<td>$f_{ck, \text{cyl}}$</td>
<td>Characteristic cylinder compression strength</td>
</tr>
<tr>
<td>$f_{ck}$</td>
<td>Characteristic compression strength for lightweight concrete</td>
</tr>
<tr>
<td>$f_{ck, \text{cube}}$</td>
<td>Characteristic cube compression strength for lightweight concrete</td>
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<tr>
<td>$f_{ci}$</td>
<td>Compression strength for each sample</td>
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<tr>
<td>$f_{cm}$</td>
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<td>$f_{ct, \text{fl}}$</td>
<td>Flexural tensile strength for lightweight concrete</td>
</tr>
<tr>
<td>$f_{ct, \text{sp}}$</td>
<td>Splitting tensile strength for lightweight concrete</td>
</tr>
<tr>
<td>$f_{ctm}$</td>
<td>Central tensile strength for lightweight concrete</td>
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<tr>
<td>$f_{ct,\text{eff}}$</td>
<td>Effective concrete tensile strength</td>
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<td>$f_t$</td>
<td>Tensile strength</td>
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<td>$f_y$</td>
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<td>$f_{bd}$</td>
<td>Design bond stress</td>
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<td>$f_{ck,0.05}$</td>
<td>Characteristic value of normal concrete tensile strength with 5 % Quantile</td>
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<tr>
<td>$f_{ck,0.05}$</td>
<td>Characteristic value of lightweight concrete tensile strength with 5 % Quantile</td>
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<tr>
<td>$J$</td>
<td>Overall performance factor</td>
</tr>
<tr>
<td>$J.$</td>
<td>Joint</td>
</tr>
<tr>
<td>$M_u$</td>
<td>Ultimate bending moment</td>
</tr>
<tr>
<td>$M_c$</td>
<td>Column bending moment</td>
</tr>
<tr>
<td>$M_b$</td>
<td>Beam bending moment</td>
</tr>
</tbody>
</table>
\( M_{0.001} \) Bending moment at maximum fibre concrete strain of 0.001
\( m \) Mass
\( n \) Number of samples
\( P \) Applied load
\( P_o \) Column design axial load
\( S \) Wall thickness
\( S_{r,max} \) Maximum distance between cracks
\( T_c \) Column reinforcement bars tensile force
\( T_b \) Beam reinforcement bars tensile force
\( T_h \) Horizontal tensile force
\( T_v \) Vertical tensile force
\( Typ. \) Type
\( U \) Heat transfer coefficient
\( V \) Volume
\( V_b \) Beam shear force
\( V_c \) Column shear force
\( w/c \) Water cement ratio
\( w_k \) Crack width
\( x \) Depth of compression zone
\( Z \) Distance between compression and tensile reinforcement bars

**Greek Letters**

\( \eta_E \) Modulus of elasticity reduction factor
\( \eta_1 \) Tensile strength reduction factor
\( \gamma_c \) Concrete strength safety factor
\( \rho \) Density
\( \rho_{min} \) Minimum density
\( \rho_{dry} \) Dry density
\( \rho_{eff} \) Effective reinforcement ratio
\( \lambda_{min} \) Minimum thermal conductivity
\( \lambda_{dry,10} \) Dry thermal conductivity at 10 °C middle temperature
\( \alpha \) Reduction factor for durability and for design compression strength
\( \alpha_e \) Ratio between the E-modulus of reinforcement and the E-modulus of concrete
\( 1/\alpha_i \) Inner heat transfer resistance
\( 1/\alpha_a \) Outer heat transfer resistance
\( \varphi(t,t_o) \) Creep factor at time \((t-t_o)\)
\( \varphi_{NC} \) Creep factor for normal concrete
\( \varphi_{LWC} \) Creep factor for lightweight concrete
\( \varepsilon_u \) Strain at ultimate limit state
\( \varepsilon_c \) Maximum concrete compression strain
\( \varepsilon_{cm} \) Mean strain of concrete between cracks
\( \varepsilon_{sm} \) Mean strain of reinforcement bars
\( \varepsilon_{cr}(t) \) Time dependent deformation (creep) experienced in period \((t-t_0)\)
\( \varepsilon_{el}(t_o) \) Instantaneous elastic deformation at time \((t_o)\)
\( \sigma_s \) Reinforcement tensile stress
\( \Delta_1 \) Elastic deformation at time of loading \((t_o)\)
\( \Delta_2 \) Creep deformation
\( \Delta_3 \) Elastic recovery deformation
\( \Delta_4 \) Deformation of relaxation
\( \Delta_R \) Residual deformation
\( \delta \) Standard deviation
\( \delta_h \) Horizontal displacement
\( \theta \) Angle of compression trajectories
\( \mu \) Deformability factor
\( \mu_M \) Overall Deformability factor
\( \mu_A \) Displacement ductility
\( \mu_\phi \) Curvature ductility
\( \mu_\theta \) Rotation ductility
\( \mu_{en} \) Ductility index
\( \Delta \) Story horizontal displacement
\( \Delta_u \) Displacement at ultimate limit state
\( \Delta_y \) Displacement at yield
\( \Delta_l \) Deformation at un-cracked state
\( \phi_u \) Curvature at ultimate limit state
\( \phi_y \) Curvature at yield
\( \phi_{0.001} \) Curvature at maximum fibre concrete strain of 0.001
\( \phi_{ex} \) Experimental curvature
\( \theta_u \) Plastic hinge rotation at ultimate limit state
\( \theta_y \) Plastic hinge rotation at yield

**Others**

ACI American Concrete Institute
ASCE American Society of Civil Engineers
CCC Joint has three compression forces – Strut-and-tie model method
ESCSI Expanded Shale Clay and Slate Institute
EX Experimental
FE Finite Element
FEM Finite Element Method
FRPs Fibre Reinforced Polymers
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<td>Glass-Fibre Reinforcement</td>
</tr>
<tr>
<td>GF</td>
<td>Glass-Fibre</td>
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<td>ILWC</td>
<td>Infra-Lightweight Concrete</td>
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<td>LWA</td>
<td>Lightweight Aggregate</td>
</tr>
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<td>LWC</td>
<td>Lightweight Concrete</td>
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<td>Lightweight Aggregate Concrete</td>
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<td>LC</td>
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<td>NC</td>
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<td>NDC</td>
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<td>PP</td>
<td>Polypropylene fibres</td>
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<td>RFT</td>
<td>Reinforcement</td>
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<td>Relative Humidity</td>
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<td>SG</td>
<td>Strain Gauge</td>
</tr>
</tbody>
</table>
CHAPTER 1

INTRODUCTION

Some times the need to reduce the weight of a structural element has not less importance than increasing its strength, especially in heavy structures such as tall buildings and bridges where the own weight of the structure is one of the main problems that faces the designers. Another important demand in concrete structures is to get monolithic fair-faced concrete, which does not only possess high visual qualities. Monolithic concrete structures are also particularly durable, and the fact that no plastering or cladding is required leads to cost savings and makes buildings more sustainable and easier to recycle. However, due to the high thermal conductivity of normal concrete, fair-faced concrete without insulation causes prohibitive air conditioning costs in cold and warm countries like Germany and Egypt respectively.

Today, Lightweight Aggregates (LWA) are available in a wide range of densities, strengths, and sizes. This makes it possible to design Lightweight Concrete (LWC) with a very wide spectrum, a concrete of very low density for insulation and, at the same time, a high strength lightweight concrete for structural purposes. These types of LWC will be presented in Chapter 2. The basic advantage of LWC is its low density, which reduces the dead load and provides insulating properties. Along with this, it is easy to handle, and heavy duty tools are not required.

In spite of the increasing use and demand of LWC, there is still a lack of adequate explanations to identify the conceptual and structural design aspects for buildings made from LWC and Infra-Lightweight Concrete (ILWC). Issues such as element dimensions, connections, and reinforcement types and details as well as short- and long-term deformations and dynamic behaviour for LWC structures are necessary to be covered in recent codes. Therefore, this study treats conceptual and structural design of buildings made from LWC and ILWC and generally consists of two main objectives:

- Development and production of new mixtures for LWC and ILWC with minimized dry density and very good mechanical and physical properties.
- The ability to apply and involve these new materials in the construction field through intensive series of experimental tests on different structural elements and connections under static and dynamic loads.

In order to achieve the first objective in the study, two targets were defined, the first: fair-faced ILWC for walls with minimum dry density ($\rho_{\text{min}} < 800 \text{ kg/m}^3$), minimum thermal conductivity ($\lambda_{\text{min}}$) enough to eliminate the heat insulation materials and maximum strength ($f_{c,\text{max}}$) enough to resist the vertical bearing stress from floors. The second target: fair-faced LWC for construction of floor slabs and beams with minimum dry density ($\rho_{\text{min}} = 1000 - 2000 \text{ kg/m}^3$), minimum thermal conductivity ($\lambda_{\text{min}}$) and maximum strength ($f_{c,\text{max}}$) enough to resist flexural and shear stresses comparable to normal concrete (NC).

In other words, these new materials will have two important potential benefits. The first: its good thermal properties that allows saving energy (heating energy in cold country like Germany and cooling energy in warm country like Egypt). The second benefit is its low weight with good mechanical properties, which leads to reduce the global costs of heavy
structures such as tall buildings and bridges when this LWC is applied in construction of floor slabs of tall buildings or the concrete deck of bridges.

Once the ILWC and LWC materials were developed and their properties were determined, a series of large-scale experiments was realised.

For ILWC a real application, a one-family house in Berlin was built in 2006 [Schlaich M., et. al., 2007]. Because of its favourable physical properties and its good durability, ILWC reinforced with GFR was used for the first time as monolithic cast-in-site concrete to construct the outer walls of this house without any additional insulation [Schlaich M., et. al., 2008]. Infra-lightweight concrete is an engineered high-tech material of which its potential and several design aspects are not yet fully exploited. These several design aspects will be presented and discussed in Chapter 3.

ILWC has not enough strength to use it in construction of roof slabs. Therefore, the new LWC with very good thermal properties and with concrete strength comparable to normal concrete can be used for construction of the roof slabs in low and medium rise buildings, i.e. the outer perimeter of these buildings will be constructed using ILWC for external walls and LWC for the last floor slab as in Figure 1.1. The structural details between ILWC external walls and NC floors, the bond behaviour between ILWC and different reinforcement bars such as steel and glass-fibre bars (GFR) as well as the flexural and shear behaviour of LWC beams will be presented in the study.

**WALL:** Infra-Lightweight Concrete

\[ \rho_{\text{dry}} < 0.8 \text{ g/cm}^3 \ ; \lambda < 0.2 \text{ W/mK} \ ; f_{\text{max}} \]

**ROOF:** Lightweight Concrete

\[ f_{\text{max}} > 25 \text{ MPa} \ ; \lambda = \text{good} \ ; \rho_{\text{min}} \]

**FLOOR:** Normal Concrete

Everything is already known

![Figure 1.1: Layout of fair-faced concrete building with walls of infra-lightweight concrete and roof of lightweight concrete.](image)

As mentioned in the previous paragraph, infra-lightweight concrete with a dry density of less than 0.8 g/cm³ and concrete strength class LC 8/9 does not have enough strength to be used for floor slabs. A possible economic layout for tall buildings with low energy consumption could be: ILWC for the exterior walls, LWC with good insulation and mechanical properties for the floor slabs, and normal concrete with good heat storage capacity for vertical elements. The new structural LWC mix in this study has a dry density of 1.25 g/cm³ (about half as normal concrete) and strength comparable to normal concrete. Chapter 4 presents the details about development and manufacturing of this new LWC.

For LWC, first eight beams constructed from the new developed LWC mixture with concrete strength class of LC 30/33 and reinforced with glass-fibre reinforcement bars (GFR) and steel reinforcement (SRFT), in addition to two control beams constructed from normal concrete C 30/37 and reinforced with SRFT, were tested experimentally for flexural strength capacity,
crack behaviour, deformation capacity, ductile behaviour, and bond behaviour in tension and compression zones of the beams. The behaviour of the tested beams will be presented in Chapter 5.

From the economic point of view, using LWC in construction of the floor slabs in tall buildings will reduce the total costs of tall buildings through reduction of steel reinforcement amount, foundation type and volume in addition to reduction of vertical members’ cross-sections that saves the used horizontal area (Figure 1.2). Therefore, another experimental study was done to investigate the behaviour of interior and exterior joints between LWC beams and NC columns under seismic loads, because they are the most affected components of tall buildings during earthquake excitations.

TALL BUILDINGS WITH:

(a) Normal concrete floor slabs:
- large amount of steel
- large cross sections for vertical members
- large foundation

(b) Lightweight concrete floor slabs:
- small amount of steel
- small cross sections for vertical members
- small foundation

Figure 1.2: Layout comparison for tall buildings with normal and lightweight concrete floor slabs.

As seismic design of structures moves towards performance based design, there is need for new structural members and systems that possess enhanced deformation capacity and damage tolerance, while having simple reinforcement details. The development of a highly damage-tolerant beam-column connection would allow structural engineers to design joints for moderate shear distortions while exhibiting little damage, reducing rotation demands in beam plastic hinges, and eliminating the need for post-earthquake joint repairs. One option for achieving this goal is to use LWC beams, which reinforced with glass-fibre reinforcement bars with superior deformation capacity in beam-column connections. Chapter 6 will present the influence of different design parameters on the behaviour of these connections under dynamic loads.

The conclusions and recommendations from the study will be presented in Chapter 7.
2.1 Introduction

This chapter summarises the state-of-the-art in lightweight concrete. The different types of lightweight aggregates will be presented followed by a historical overview and the most important recent and most modern applications of lightweight concrete in tall buildings, bridges, and other special structures. The following books give a good overview for design and applications of lightweight concrete structures.

- “Lightweight Concrete - History, Applications, Economics” Expanded Shale Clay and Slate Institute, 1971
- “Lightweight Aggregate Concrete - Science, Technology, and applications” Chandra, S. and Berntsson, L., 2002
- “Leichtbeton im Konstruktiven Ingenieurbau” Faust, T., 2003
- “Structural Lightweight Aggregate Concrete” Clarke, J., 2005

Figure 2.1: The Pantheon Dome as the most notable LWC structure during the early Roman Empire [Filipaj, P., 2006].
In the following table, the most important characteristics of LWC buildings in the last 70 years are summarised.

Table 2.1: Applications of lightweight concrete in tall buildings and special structures

<table>
<thead>
<tr>
<th>Project</th>
<th>Year</th>
<th>LC [N/mm²]</th>
<th>ρ [g/cm³]</th>
<th>LC /ρ</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Tall Buildings:</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Marina City Towers, Chicago, 180 m height, 19000 m³</td>
<td>1962</td>
<td>25/28</td>
<td>1.68</td>
<td>14.88</td>
</tr>
<tr>
<td>Australia Square, Sydney, 184 m height, 31000 m³</td>
<td>1967</td>
<td>30/33</td>
<td>1.87</td>
<td>16.04</td>
</tr>
<tr>
<td>Lake Point Tower, Chicago, 196 m height, 26000 m³</td>
<td>1968</td>
<td>25/28</td>
<td>1.73</td>
<td>14.45</td>
</tr>
<tr>
<td>The Standard Bank building, Johannesburg, 139 m height</td>
<td>1968</td>
<td>35/38</td>
<td>1.95</td>
<td>17.95</td>
</tr>
<tr>
<td>The BMW administrative building, Munich, 101 m height</td>
<td>1973</td>
<td>30/33</td>
<td>1.85</td>
<td>16.22</td>
</tr>
<tr>
<td>Extension of the post office I, Augsburg, 17000 m³</td>
<td>1992</td>
<td>20-30</td>
<td>1.50</td>
<td>13-20</td>
</tr>
<tr>
<td><strong>Bridges:</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Woodrow Wilson Bridge, Washington</td>
<td>1961</td>
<td>35/38</td>
<td>1.84</td>
<td>19.02</td>
</tr>
<tr>
<td>Martinez Benicia Bridge, California</td>
<td>1962</td>
<td>LC 28</td>
<td>1.84</td>
<td>15.22</td>
</tr>
<tr>
<td>Silver Creek Overpass Bridge, Utah</td>
<td>1968</td>
<td>LC 22</td>
<td>1.60</td>
<td>13.75</td>
</tr>
<tr>
<td>The Friarton Bridge, Scotland</td>
<td>1988</td>
<td>30/33</td>
<td>1.70</td>
<td>17.65</td>
</tr>
<tr>
<td>Cooper River Bridge, South Carolina</td>
<td>1992</td>
<td>LC 28</td>
<td>1.80</td>
<td>15.55</td>
</tr>
<tr>
<td><strong>Shell and Special Structures:</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>TWA Terminal at Kennedy airport, New York, 2200 m³</td>
<td>1960</td>
<td>35/38</td>
<td>1.85</td>
<td>18.90</td>
</tr>
<tr>
<td>Assembly Hall, University of Illinois, 9600 m³</td>
<td>1962</td>
<td>25/28</td>
<td>1.68</td>
<td>14.88</td>
</tr>
<tr>
<td>Waiting Hall V, Frankfurt airport, 130 m span width</td>
<td>1970</td>
<td>30/33</td>
<td>1.65</td>
<td>18.18</td>
</tr>
<tr>
<td>Exhibition and Stampede Grandstand, Calgary</td>
<td>1974</td>
<td>40/44</td>
<td>1.85</td>
<td>21.62</td>
</tr>
<tr>
<td>Guggenheim Museum, Bilbao, 4800 m³</td>
<td>1997</td>
<td>20/22</td>
<td>1.60</td>
<td>12.50</td>
</tr>
<tr>
<td>Wellington Westpac Trust Stadium, 13000 m³</td>
<td>1999</td>
<td>35/38</td>
<td>1.80</td>
<td>19.44</td>
</tr>
<tr>
<td><strong>Recent Applications:</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Auditorium Maximum, TU München, München, Germany</td>
<td>1994</td>
<td>25/28</td>
<td>1.60</td>
<td>15.63</td>
</tr>
<tr>
<td>Youth Center Anna-Landsberger-Haus, Berlin, Germany</td>
<td>2001</td>
<td>LC 15</td>
<td>1.20</td>
<td>12.50</td>
</tr>
<tr>
<td>German Technical Museum, Berlin, Germany</td>
<td>2001</td>
<td>25/28</td>
<td>1.40</td>
<td>17.85</td>
</tr>
<tr>
<td>Gartmann Family Hause, Chur, Switzerland</td>
<td>2004</td>
<td>8/9</td>
<td>1.10</td>
<td>7.27</td>
</tr>
<tr>
<td>Amts- and Landgericht, Frankfurt/Oder, Germany</td>
<td>2006</td>
<td>LC 15</td>
<td>1.20</td>
<td>12.50</td>
</tr>
<tr>
<td>Schlaich Family House, Berlin, Germany</td>
<td>2007</td>
<td>8/9</td>
<td>0.76</td>
<td>10.53</td>
</tr>
<tr>
<td>MPU Heavy Offshore Lifter, Rotterdam, Netherland</td>
<td>2009</td>
<td>35/38</td>
<td>1.58</td>
<td>22.15</td>
</tr>
<tr>
<td><strong>Studied Infra-Lightweight Concrete (Ch. 3), TU-Berlin</strong></td>
<td>2006</td>
<td>8/9</td>
<td>0.76</td>
<td>10.53</td>
</tr>
<tr>
<td><strong>Studied Lightweight Concrete (Ch. 4), TU-Berlin</strong></td>
<td>2007</td>
<td>30/33</td>
<td>1.25</td>
<td>24.00</td>
</tr>
</tbody>
</table>

As shown in Table 2.1, the LWC that was used in these buildings has compressive strengths comparable to normal weight concrete, but it is typically 25 % to 35 % lighter [CEB-FIB, 1977 & FIB, 2000]. The new developed LWC used in the study has compressive strength of 40 MPa, but it is 50 % lighter than normal concrete.
2.1.1 Lightweight aggregate concrete

The term “Lightweight Aggregate (LWA)” describes a range of special use aggregates that have specific gravity considerably below normal sand and gravel which were at one time used in almost all concrete. These lightweight aggregates will range from the extremely light materials used for isolative and non-structural concrete to expanded clays and shales used for structural concrete. Since the lightness of these aggregates derives from the air trapped in each individual particle, the more air that is trapped per particle unit, the lighter the weight and the better the insulation, but, conversely, the lower the strength.

![Diagram of LWAC classification](image)

Figure 2.2: Classification of LWAC according to its unit weight [Asgeirsson, 1994].

The lightweight concrete is generally defined based on its density. A classification of Lightweight Aggregate Concrete (LWAC) according to its unit weight proposed by Asgeirsson, 1994, is shown in Figure 2.2. At the extreme left are Vermiculite and Perlite, which are sometimes referred to as the “super lightweights”. Concrete can be made with these aggregates weighing as little as 250 or 380 kg/m³. Next are the natural aggregates, Pumice and Scoria. These can be made into concrete weighing about 800 or 950 kg/m³, and it also may run as high as 1800 kg/m³.

Overlapping these are expanded shale, clay and slate aggregates produced by the rotary kiln method, which will produce a structural concrete ranging from 1360 to 1850 kg/m³. Expanded shale, clay or slate produced by sintering, and expanded slag, range from 1430 to 1950 kg/m³ and complete the spectrum. Beyond this, there are the air-cooled slag aggregates and the hard-rock aggregates such as sand and gravel and crushed stone, which produce conventional concretes weighing 2000 to 2800 kg/m³.

In general, the low density lightweight aggregate concretes at the lower end of the scale are used primarily for insulating purposes, as they have relatively low compressive strength, while those in the middle range are used for insulation and fill. The lightweight concretes at the upper end of the spectrum develop excellent compressive strength and are found in a number of structural applications.

Beside the classification of LWC that is shown in Figure 2.2, insulated “foam concrete” was investigated in the years 1984 - 1987 as a part of a project funded by the German Ministry of Investigation and Technology (BMFT) [Widmann H., et. al., 1991]. Foam concrete is a
lightweight concrete which contains air bubble pores in its mortar matrix. It can be produced with or without LWA depending on the required properties. The air bubbles are introduced by intermixing of natural or synthetic foam which is produced in a foam generator using a mixture of proteins in water. Based on the type of aggregate, foam concrete has dry density ranging from 600 to 1570 kg/m³ [Widmann H., et. al., 1991].

Foam glass or expanded glass is used recently as a LWA to produce lightweight concrete with dry density ranging from 550 to 1000 kg/m³. In Germany, this type of aggregate is known as Liaver. The LWC that is produced using this type of LWA has a maximum compression strength of 12 N/mm², which is used for insulation purpose with thermal conductivity ranging from 0.2 to 0.5 W/mK and for non-structural concrete according to DIN 1045.

Regarding to the production process, the lightweight aggregate can be divided in two categories as proposed by Chandra, et. al., 2002, the first is those occurring naturally and are ready to use only with mechanical treatment, i.e., crushing and sieving. The second is those produced by thermal treatment from either naturally occurring materials (kiln method) or from raw materials mixed with industrial by-products and waste materials (sintering process).

In the kiln method, raw material is crushed and introduced at the upper end of a kiln similar to the type used in the Portland cement industry. In passing through the kiln, the material reaches a temperature of 980 to 1200 °C, and begins to become plastic. Internal gases cause the material to expand, or bloat, and create a mass of small, unconnected air cells, which are retained after the material cools and solidifies. After leaving the kiln, the material is cooled and then crushed and graded.

There are several variations in the kiln process. In one case, all material retained on a ¾ inch screen after burning is crushed. In another, the raw material is pre-sized before entering the kiln so that crushing after burning is not necessary. Still another variation consists of extruding or pelletizing fine raw material as a means of pre-sizing the raw kiln feed. Combinations of these three variations are found throughout the industry.

In the sintering process, raw clay or shale is mixed with pulverized fuel and burned and expanded under controlled conditions on a moving grate. The mechanics of this method in some cases require twenty or thirty percent of the burnt material to be remixed with raw material and re-burned on the travelling grate.

The specific gravity of the lightweight aggregates is significantly less than for conventional aggregates, ranging from 1000 to 2200 Kg/m³, and structural LWC made from these aggregates is generally 20 to 30 percent lighter than conventional concrete [ESCSI]. The aggregate for lightweight concrete may consist of 100 percent lightweight aggregates, or a combination of lightweight and normal weight aggregates (usually local sand).

In many codes lightweight aggregate concrete is defined as a concrete having a dry density of less than 2000 kg/m³. However, LWAC can be produced within a range from 300 to 2000 kg/m³, corresponding to cube strengths from approximately 1 to over 60 MPa and thermal conductivities from 0.2 to 1.0 W/mK [Newman, 1993].

Although Structural Lightweight Concrete (SLWC) is defined according to expanded shale clay and slate institute as a concrete having air-dry weight not to exceed 1840 kg/m³ and a 28-day compressive strength of 17.5 N/mm² or more, the structural LWC according to German code (DIN 1045-1) is considered that concrete with cylinder compression strength up 16 N/mm². In DIN 1045-1, the concrete with an oven-dry density between 800 Kg/m³ and 2000 Kg/m³ are defined as LWC. This upper limit is a transition to Normal Concrete (NC).
In Germany, the Lightweight aggregates produced from clay or shale by the kiln process are known as Liapor. The raw material is crushed, dried, and milled into powder. It is homogenized and stored ready for pelletization. After the pelletization process for appropriate size, they are transported to a rotary kiln. In production, the pellets can be made to a predetermined size and the expansion can be controlled to produce particles of the required density. This type of LWA is used in the development of Infra-Lightweight Concrete (ILWC) and LWC mixtures used in the study.

Recently, in Switzerland and Germany, some buildings made of monolithic fair-faced insulating LWC have been constructed. Concrete mixes with densities above 1000 kg/m³ were used [Faust T., 2003, Filipaj P., 2006 & Baus U., 2007]. Worthy of mention is a residential house in Chur, Switzerland, where the architect Patrick Gartmann used expanded clay and glass as lightweight aggregates to get insulating concrete with heat conductivity of $\lambda = 0.32$ W/mK and concrete strength of LC 8/9. Even lighter concrete mixes using only expanded clay are used in shipbuilding and were developed by Professor Christian Thienel [Thienel K.C., et. al., 2007] of the “Universität der Bundeswehr” in Munich. Inspired by the Swiss house and based on the Munich findings the departments of “Conceptual and Structural design” and “Construction and Building Material Testing”, both belonging to the Institute of Structural engineering at the “Technische Universität Berlin”, started in the summer of 2006 to jointly develop ILWC with very low thermal conductivity [Schlaich M., et. al., 2007]. ILWC has not enough strength to use it in construction of roof slabs. Therefore, the new LWC with very good thermal properties and with concrete strength comparable to normal concrete [El Zareef M., et. al., 2010] is developed and investigated in the thesis.

2.1.2 Historical view

The first known use of LWC dates back over 2000 years [ACI-213R]. There are several LWC structures in the Mediterranean region, but the three most notable structures were built during the early Roman Empire and include the Port of Cosa, the Pantheon Dome (Fig. 2.1), and the Coliseum. The Port of Cosa, built in about 273 B.C., used LWC made from natural volcanic materials. These early builders learned that expanded aggregates were better suited for marine facilities than the locally available beach sand and gravel. They went 40 km to the northeast to quarry volcanic aggregates at the Volcine complex for use in the harbor at Cosa [Bremner, Holm, and Stepanova, 1994].

This harbor is on the west coast of Italy and consists of a series of four piers (4 m cubes) extending out into the sea. For two millennia they have withstood the forces of nature with only surface abrasion. They became obsolete only because of siltation of the harbor.

The Pantheon, finished in 27 B.C., incorporates concrete varying in density from the bottom to the top of the dome. Roman engineers had sufficient confidence in LWC to build a dome whose span of 43.3 m was not exceeded for almost two millenniums. The structure is in excellent condition and is still being used to this day for spiritual purposes [Bremner, Holm, and Stepanova, 1994]. The dome contains intricate recesses formed with wooden formwork to reduce the dead load, and the imprint of the grain of the wood can still be seen. The excellent cast surfaces that are visible to the observer show clearly that these early builders had successfully mastered the art of casting concrete made with lightweight aggregates.

Since World War I the applications of lightweight structural concrete is rapidly spread wide. Besides the weight savings, LWC has substantially better fire-resistant qualities than normal weight concrete, and significantly lower heat transmission. Its remarkable moisture resistance and durability is evidenced in samples which have been subjected to daily cycles of wetting
and drying in salt water for more than 30 years, showing an increase in compressive strength from 39 N/mm² to more than 70 N/mm², and with a cover of only 1.5 cm thickness, completely protecting the steel reinforcement from the corrosive action of the salt water [ESCSI, 1971].

The use of lightweight aggregate in masonry blocks permits increased labour productivity because the low weight makes for greater speed and ease of handling. Similar considerations apply in the case of precast elements and tilt-up construction using lightweight structural concrete. In these as well as in cast-in-place applications, contractors find that the same controls used with other materials and processes will produce a highly serviceable end product.

For architects and engineers, structural LWC has opened up a broad range of applications: tall building frames, long-span roof and bridge structures, thin shell construction, including the hyperbolic paraboloid roof structure, sculpture and special design effects in form and texture. Structural LWC is found in projects such as the thin shell “bird in flight” roof of the TWA Terminal at John F. Kennedy Airport; Central Administrative Building of BMW in Munich-Germany, the floors are built in lightweight concrete using expanded clay (Liapor) aggregate; the towering Southland Centre in Dallas; The West Stand of Newcastle United Football Club at St James Park in Newcastle-UK; the Sandhornoya Bridge in Norway and the Rottepolderplein Bridge near Haarlem in the Netherlands, Liapor was used as LWA for the both bridges; the ultra-modern Learning Centre-Library at the University of Utah, notable for its long spans and high load design; the University of Illinois Assembly Hall, with a concrete dome roof of near record proportions. All of these are examples of trends in construction made possible by lightweight structural concrete. Some of these structures will be presented in the following sections.

2.2 Applications of Lightweight Concrete in Tall Buildings

The World War I research on lightweight aggregate concretes put the expanded shale industry into its first commercial production, and after the war, additional experiments were conducted by private enterprise. As a result, as early as June 1919, the chief engineer of the Turner Construction Company of New York could suggest that lightweight structural concrete could offer significant construction economies - through reduction in reinforcing steel requirements - in high-rise commercial construction. Speaking at the 15th convention of the American Concrete Institute at Atlantic City in 1919, he said: “In addition to the saving in steel reinforcement there is a saving in concrete in the columns due to the reduced weight of the floor construction”.

The first commercial plant dedicated to expanded shale aggregate began operating in Kansas City, Missouri, in 1920 under the name Haydite Company. Where wartime production had been handled at brick and cement plants, the Haydite Company was an expanded shale aggregate plant, with a mission to both produce the material and introduce it into the commercial construction market. In Europe, however, it was not until 1931 that the manufacture of lightweight expanded clay aggregate commenced in Denmark. Thereafter developments quickly spread to Germany, Holland and the UK [Clarke J.L., 2005].

Even so, there were few design criteria available that could apply to use LWC in building construction, and little inclination among architects, engineers and builders to risk their reputations by experimenting with the new material. It was taken for granted that in order to be impermeable as well as durable and strong, concrete had to be of maximum density and
weight. So it was not until 1922 that the industry had a “living example” building employing lightweight structural concrete and demonstrating both its economics and its construction reliability. This was a gymnasium addition to the Westport High School in Kansas City, the first LWC building in modern history. The building employed LWC to avoid the difficult foundation work that would have been required with conventional weight concrete because of the poor load-bearing characteristics of the soil at the site. At the time, the expanded shale aggregate sold at $7.85 per cubic meter, as contrasted with $3.25 per cubic meter for sand and gravel, and yet the economies in foundation engineering made possible by the reduction in deadweight load more than compensated for the price differential.

The first major project employing structural LWC was undertaken in 1928 and 1929, in the form of an addition to the Southwestern Bell Telephone Company office (Figure 2.3) in Kansas City. The building was originally built as a 14-story structure, and the company had found that the foundations and underpinning would support an additional eight floors, taking into account the additional dead load of conventional normal concrete. However, analysis by the designers indicated that by the use of lightweight expanded shale concrete rather than conventional sand and gravel concrete, 14 LWC floors could be safely added rather than 8 conventional concrete floors, doubling the above-ground height of the building and producing a skyscraper with a total of 28 floors. The project was undertaken with concrete mixed on-site (this was before the day of the ready-mix plant). When completed, the building addition showed a total dead load reduction of more than 2700 tons through the use of lightweight structural concrete. Compressive strength of the LWC was 24.5 MPa at 28-day, an almost unprecedented high at the time. And the building has stood as a demonstration of the practicality and economics of lightweight structural concrete.

The first extensive use of structural LWC in high-rise building was the Park Plaza Hotel in St. Louis (now the Chase-Park Plaza, Fig. 2.4). Built in 1929, this 28-story structure made of structural LWC in both frame and floor systems, as well as for fireproofing [ESCSI, 1971].

With these demonstrations of the feasibility of lightweight structural concrete in high-rise buildings, acceptance of the product was established, and succeeding years saw an increasing number of architects and engineers specifying it for major construction projects. The use of lightweight aggregate concrete in structural concrete has increased rapidly since World War II, as architects, engineers, and builders have availed themselves of greatly increased research
activity and improved application technology. The first framed building constructed in Britain using lightweight aggregate concrete, a three-storey office block in Brentford, was constructed in 1958. This is some 30 years behind the United States where the Park Plaza Hotel in St. Louis and the South Western Bell Telephone Company in Kansas City were built using this material [Clarke J.L., 2005].

The first “skyscraper” using structural LWC throughout its above-ground structure was the 18-story Dallas Statler-Hilton, built in 1955. Since that time, there have been many others: the twin towers of Chicago's famed 60-story Marina City, built in 1962, rise 180 m above street level and set a new world record for height of reinforced-concrete-framed structures, using structural LWC for all floors and beams. A similar tower in Sydney, Australia (1967) - part of the ambitious Australia Square project - set (Figure 2.5) a new record as the world's tallest reinforced LWC building, standing 184 m high and featuring load bearing precast LWC formwork and 11 m span beams, slabs, columns, precast concrete and even bricks made of LWC. This record was subsequently broken by Chicago's Lake Point Tower (Figure 2.6) at 196 m, built in 1968.

One of the most interesting examples from the 1970s in the United Kingdom is Guy’s Hospital, London (Figure 2.7), for which R. Travers Morgan & Partners were the consulting engineers. Lightweight aggregate concrete was used to reduce foundation loads not only in structural elements but also in the external walls. Two tower blocks, the “User Tower” and the “Communication Tower” were built; these were, respectively, 122 and 145 m above ground floor, each with a lower ground floor storey and single basement. The first five levels of superstructure were constructed using solid lightweight aggregate slabs; above this level a special ribbed floor construction was developed. Lightweight aggregate concrete with a fluted profile was also used for the external walls of the higher tower which were cast in-situ. The mix specified was to have 28-day strength of 31.3 MPa, and both Lytag (expanded shale) coarse and fine aggregate were used, with 390 kg/m³ of cement.

Advances in lightweight construction have not been limited to high-rise apartment and office buildings. Equally spectacular achievements have been made in bridge construction, stadiums, churches, educational facilities, and commercial structures such as warehouses, manufacturing plants, piers, and even sewage treatment plants.

Figure 2.5: Park Regis, 1967, Sydney, Australia [ESCSI, 1971]. (left)
Figure 2.6: Lake Point Tower, 1968, Chicago, USA [Clarke, J.L., 2005]. (middle)
Figure 2.7: Guy’s Hospital, 1971, London, United Kingdom [Clarke J.L., 2005]. (right)
Airport and church construction in particular demonstrates the whole new vistas of design freedom that have been opened to architects and engineers with the advent of thin shell construction using expanded shale lightweight aggregate concrete. The hyperbolic paraboloid and conoid roof shapes are just two examples of the possibilities of a lightweight structural concrete that can be cast to minimum thicknesses and still provide the necessary strength. In the hyperbolic paraboloid roof of the TWA terminal (Figure 2.8), for example, due to lightweight concrete's strength, the shell thickness was shaved to 6.35 cm, and still provided a 33 percent safety factor for supporting a full water load should the central drain become restricted.

So many new records have been set, for buildings employing structural LWC framing and floor systems that at any given moment it is difficult to point with certainty to the “world’s tallest” LWC building. At one time or another, records have been set by such structures as Central Administrative Building of BMW in Munich-Germany (100 m), Bank of Georgia Building in Atlanta (119 m), the Standard Bank Building in Johannesburg (139 m), Marina Towers in Chicago (180 m), Lake Shore Plaza in Chicago (184 m), Australia Square in Sydney (184 m), and Lake Point Tower in Chicago (196 m). The advantage of structural LWC in this type of construction is in the significant reduction in dead load, which not only saves on foundation costs but also permits smaller supporting columns and is an important factor in computing wind effects, as it is in minimizing the whiplash effect of dead load in earthquake areas.

In a number of instances, use of lightweight concrete has permitted addition of extra floors beyond the original design. In the classic example, Southwestern Bell Telephone Company was able to double the height of its existing building from 14 to 28 stories, 6 more than would have been possible with normal weight concrete. An office tower in Ottawa, originally designed as a 22-story building using normal weight concrete, was extended to 25-story by changing concrete specification above the eighth floor level to lightweight. A 200 percent increase in height was feasible for the Magnolia State Savings and Loan Association Building in Jackson, Mississippi. Originally only two stories, the building now stands six stories high.

### 2.3 Applications of Lightweight Concrete in Bridges

Among the more spectacular and sensational landmarks in the growth of lightweight structural concrete applications have been its use in a number of major bridges.
In the construction of the San Francisco-Oakland Bay Bridge (Figure 2.9), for example, the use of a LWC floor in the upper deck permitted weight reduction of 122 kg/m², or a total of 14333 tons for the entire structure. This in turn permitted reduction in the area and cost of members in the superstructure, and materially reduced the direct load on foundations and the stresses on foundations and superstructure due to assumed seismic forces. In all, the cost savings affected were estimated at $3 million.

Built in 1988, the bridge over the river Rhine to carry the traffic between the south and east of the Netherlands around the city. The main span is 133.4 m since navigation does not allow for piers in the river. The approach spans over the flood plains are, beginning at the abutments, 37.0, 4×49.0 and 80.5 m on each side. The width of the bridge deck is 28.2 m, composed of four traffic lanes and a separate bicycle and pedestrian lane. The main and two adjacent spans are a box section with variable depth. The approach spans over the flood plains have a double T cross-section. A comparative study was made between normal concrete using gravel as aggregate and lightweight concrete with sintered expanded shale, as aggregate. Financial, as well as environmental reasons led to the decision to choose lightweight concrete for the whole bridge, superstructure as well as piers and abutments [Clarke J.L., 2005].

Another example for using LWAC in bridges was the bridge over the river Sinigo at Avelengo in Italy [Fabio B., et. al., 2004]. The bridge is located in the route between Merano and Avelengo and crosses the canyon of the river Sinigo at about 1250 m above sea level (Figure 2.10). Since the total abutment-to-abutment span to be crossed was 125 m, the most economic solution would have been a continuous beam on four supports (two abutments and two intermediate piers, the sub spans being 31, 63, 31 m), but this solution ran head-on into the difficulty of accessing the canyon’s sides, both because of the steepness of the slopes and because no service road could be built without having to cut down numerous good-sized trees along its route. The inevitable choice was therefore a single 125 m long span. The structural solution was a triple hinge arch [Clarke J.L., 2005].

Furthermore, during construction the two semi-arches had to act as cantilevers until the hinge at the top could be set in. It was thus necessary to provide a temporary counterbalancing and anchoring system at the abutments, able to take the high loads that a 62 m long cantilever
would transmit. On the one hand, the structure could be anchored to the rock by temporary prestressed tiebars; and on the other, the dead weight of the advancing structure could be reduced. The second approach was adopted. The segments were cast using expanded-clay concrete, with a lower density than normal concrete. The abutments functioned during the construction phase as counterweights. The abutments are truss shaped. They were cast in normal dense concrete and filled with massive concrete to increase the balancing moment. The total bridge length is 158 m, of which 125 m is clear centre span. The total deck width is 8.1 m. The cantilever structure is formed from a single box girder 5.0 m wide at the bottom. The depth of the box girder is 2.2 m at mid-span and 7.0 m near the abutments. Each cantilever consists of 15 precast elements.

2.4 Applications of Lightweight Concrete in Precast

Precasting of lightweight structural concrete is particularly advantageous in the case of bridges and similar structures where physical conditions or traffic movement make conventional procedures difficult or impractical. A lollipop-shaped, 400 m fishing pier at Venice, California, for example, employs 215 lightweight deck slabs and 103 lightweight pier caps, which were cast in a five-acre parking lot near the shore end of the pier and then moved into position.

Another recreation application was in the construction of the Los Angeles Dodger Stadium in Chavez Ravine, where most of the structural members - including floors, beams, columns and stairways - were precast with conventional reinforcement at a casting yard near the site, then hoisted into place and connected to make the stadium proper. All above-ground elements of the stadium are expanded shale aggregate lightweight concrete.

The Standard Bank building in Johannesburg (Fig. 2.11) is 139 m high and contains 30-story. The lightweight aggregate concrete was used in the floor slabs in order to reduce dead load. The floor slabs in this building were constructed using precast double-T units which were steam-cured and lifted one day after casting to achieve a rapid erection time. The units are 10 m long and up to 3.16 m wide; the slab is 75 mm thick between ribs at 1.58 m centres. The expanded clay was used for the coarse (20/10 mm) aggregate with natural sand, giving a
relatively dense mix (1950 kg/m³ at 28-day). The hanging structure allowed the creation of an open area at ground level around the central core during construction [Chandra, et. al., 2002].

Precasting can also provide substantial economies where intricate designs, modules or repetitive forms are involved. And the weight savings offered by lightweight expanded shale aggregate concrete makes it possible to deal with large and complex shapes using conventional lifting and transportation equipment. In the construction of the Oakland Airport, precast structural lightweight concrete played an important part in obtaining economies of this type. Forty-eight 2.8 m² hyperbolic paraboloid roof shapes for the TWA terminal building were precast using only two forms, while more than 20 conoid shapes for the airline's ticketing building were similarly precast using only two forms. Handling and positioning of the roof elements was done by a single mobile gantry crane. Mass production and simplified erection of the thin shell shapes of Oakland resulted in a roof cost of $16.70/m², as contrasted with $24.20/m² by the next lowest cost method - a saving of nearly 30 percent.

The new research centre of the Hyster Company near Portland, Oregon, is made up of tilt-up wall panels approximately 2.5 m² and 15 cm thick. The size of the panels made the weight reduction offered by lightweight aggregate especially important, and the ease of handling was evident in the brief period of time required for construction - less than eight months. An unusual application of this principle was seen in the new manufacturing addition to the Dominion Cellulose plant in Toronto, where standard double-T floor slabs were stood on end with the tees out to make a wall panel 16.5 m² in place in one simple addition. Approximately 200 of the precast structural LWC elements were used in the structure, with provisions made to re-erect them on another foundation in the event of future expansion.

2.5 Applications of Lightweight Concrete in Buildings Against Bombs

Among the more spectacular and sensational landmarks in the growth of lightweight structural concrete applications have been its use in test buildings used in the historic atom bomb tests at Yucca Flats, Nevada, in 1955.

In the Yucca Flats tests, known as “Operation Cue” the objective was to determine the effect of atomic blast on structures made from various materials, including brick, lumber, aluminium, expanded shale concrete and concrete masonry. Of all the buildings in the test, only four - the two LWC masonry houses and the two lightweight structural precast concrete houses - survived without major damage; the others were all virtually destroyed. The houses employing LWC were built in conventional manner, the masonry houses reinforced to withstand earthquakes in accordance with the Pacific Coast Building Code, and the slab houses in accordance with the American Concrete Institute Code.

The slab and masonry houses at 1432 m from the explosion suffered only minor structural damage, while a frame and a brick veneer house at the same location were completely demolished. Said the official report:

The above ground portion of the two-story brick and cinder block house located 1432 m from the explosion was almost completely destroyed, and the first floor system was partially collapsed into the basement. None of the brick work remained standing, and the structure as a whole was beyond repair. The one-story frame rambler located near the two-story brick dwelling 1432 m from the explosion was likewise almost completely destroyed.

Both the one-story reinforced lightweight expanded shale concrete block house and the one story precast LWC house suffered only minor structural damage. These houses were also
located 1432 m from the explosion. With the replacement of doors and window sash, both houses could be made habitable.

The one-story precast expanded shale light aggregate concrete house and the one-story reinforced masonry block house, both located 3200 m from the explosion, suffered relatively minor damage. The one-story frame rambler, also located 3200 m from the explosion, suffered relatively heavy damage. The Operation Cue tests at Yucca Flats concluded that the survival of the lightweight buildings when exposed to the greatest forces known to man would convince even the most sceptical that lightweight aggregate had proved its worth.

2.6 Applications of Lightweight Concrete in Marine Structures

By the time of World War II, expanded shale aggregate had come into its own as a construction material - and again it was put to use in ship construction. The important difference was that where the 14-World War I ships had been largely experimental, those built in World War II - 104 in all, with cargo capacities ranging from 3200 to 140250 tons - saw widespread wartime service in battle zones.

Twenty-four of these ships were large sea-going vessels and 80 were sea-going barges of tremendous size. The total cargo capacity represented was about 488000 tons, or the equivalent in capacity of 46-Liberty ships. The total cost of the project was $167 million.

Figure 2.12: World War II concrete ship passing under San Francisco-Oakland Bay Bridge, San Francisco. Both ship and bridge made extensive use of structural lightweight concrete [Clarke J.L., 2005].

In its report on these LWC ships, the U.S. Maritime Commission indicates that the ships exhibited good handling, good performance, and unexpected resistance to near misses of shells and depth bombs. One report indicated that when a bomb exploded directly astern of one of the ships, the ship “shook like an earthquake” and was showered with shell fragments but suffered no damage. Another told of six near misses from depth bombs, with no impairment of the structure or damage to the cargo.

The Commission also reported that the hulls appeared to be completely watertight in service, carrying cargoes of wheat and sugar with no damage, mould or caking from either seepage or sweating. The riding qualities of the ships were superior to steel, the Commission added, because of their bulk and rigidity; there was little vibration, and the interiors were cooler and more comfortable. It predicted that repairs in service would probably be less costly and less frequent, and that, with no rusting or attack by sea water, the life of the hulls should be greater. The Maritime Commission concluded its report by saying that “there is ample
evidence that concrete hulls are dependable, seaworthy, and structurally as sound as hulls of any other material used for seagoing vessels. Concrete hulls have been put to as severe tests as have been given any other vessels, and it has been shown conclusively that when properly designed, properly built, and well equipped, they will perform on an equal basis with comparable steel vessels”.

### 2.7 Recent Applications of Lightweight Concrete and Infra-Lightweight Concrete

Fair-faced concrete, one layer construction element, light own-weight, low thermal conductivity, monolithic construction elements, cast-in-site concrete, simple construction details, low cost, environmental construction elements, and easy to recycling are the main parameters that push engineers and architects to restart trying to develop the LWC quite a while ago. Recently, the virtual results of these trying can be presented through number of new buildings in Switzerland, Netherlands and Germany. Table 2.2 summarized some of these recent buildings.

**Table 2.2: Recent applications of lightweight concrete and infra-lightweight concrete**

<table>
<thead>
<tr>
<th>Project</th>
<th>LC [N/mm²]</th>
<th>( \rho ) [g/cm³]</th>
</tr>
</thead>
<tbody>
<tr>
<td>MPU Heavy Offshore Lifter, Rotterdam, Netherland, 2009</td>
<td>35/38</td>
<td>1.58</td>
</tr>
<tr>
<td>Schlaich Family House, Berlin, Germany, 2007</td>
<td>8/9</td>
<td>0.76</td>
</tr>
<tr>
<td>Amts- and Landgericht, Frankfurt/Oder, Germany, 2006</td>
<td>LB 15</td>
<td>1.20</td>
</tr>
<tr>
<td>Project</td>
<td>LC [N/mm²]</td>
<td>ρ [g/cm³]</td>
</tr>
<tr>
<td>------------------------------------------------------------------------</td>
<td>------------</td>
<td>----------</td>
</tr>
<tr>
<td>Gartmann Family Hause, Chur, Switzerland, 2004</td>
<td>8/9</td>
<td>1.00</td>
</tr>
<tr>
<td>German Technical Museum, Berlin, Germany, 2001</td>
<td>25/28</td>
<td>1.40</td>
</tr>
<tr>
<td>Youth Center Anna-Landsberger-Haus, Berlin, Germany, 2001</td>
<td>LB 15</td>
<td>1.20</td>
</tr>
<tr>
<td>Auditorium Maximum, TU München, München, Germany, 1994</td>
<td>25/28</td>
<td>1.60</td>
</tr>
</tbody>
</table>
Generally, for the LWC structures that are mentioned in previous literature review and even that are applied in the recent structures, it can be concluded that there were big varieties in concrete density with relative to concrete strength and concrete thermal conductivity. Figures 2.13 & 2.14 [Faust T., 2003] show the spectrum line that controls the relation between LWC density and its compressive strength as well as its thermal conductivity respectively.

In spite of this variety for LWC and its favourable mechanical and thermal properties, nowadays, the spread of LWC is considered limited. This may be due to the lack of experimental and analytical studies in this field. Therefore, in this study, two new LWC mixtures and their conceptual and structural design aspects will be presented.

- Infra-Lightweight Concrete (ILWC) is developed with dry density under 800 kg/m³, low thermal conductivity under $\lambda = 0.2 \text{ W/mK}$, and enough strength to resist bearing stress from floor slabs.

- Lightweight Concrete (LWC) is developed with minimum dry density as half as normal concrete, low thermal conductivity, and maximum compression strength comparable to strength of normal concrete and adequate to be used in construction of floor slabs and beams.

Many issues such as the development of these new materials, the structural details for the connections between ILWC walls and NC floor slabs, and the structural behaviour of LWC beams and its interaction with NC columns under dynamic loads will be presented in the following chapters.
CHAPTER 3
INFRA-LIGHTWEIGHT STRUCTURAL CONCRETE

3.1 Introduction

Monolithic structures of fair-faced concrete not only have a high architectural potential but also are very durable. Since no plaster and cladding is needed, cost is saved and recycling is made easier. Unfortunately, the heat conductivity of normal concrete (NC) is so high that in cold countries like Germany monolithic fair-faced concrete buildings have virtually disappeared. Since the oil crisis of the seventies of the last century it is either necessary to construct exterior walls as complicated and costly double-layer structures with interior insulation that is difficult to inspect, or one contents himself with fair-faced concrete on one side only and uses conventional thermal insulation on the other side of the wall.

Therefore, engineers and architects have started trying to develop concrete with low thermal conductivity quite a while ago. Already in the 80s of the last century, insulated “foam concrete” with a dry density below 1000 kg/m³ was studied as a part of a project funded by the German Ministry of Investigation and Technology (BMFT). Weight reduction was achieved by using pre-mixed protein foams [Widmann H., et. al., 1991]. Since the only prototype building with this concrete showed unacceptable cracking due to strong shrinkage deformations the project was not continued.

Recently, in Switzerland and Germany, some buildings made of monolithic fair-faced insulating LWC have been constructed. Concrete mixes with densities above 1000 kg/m³ were used [Faust T., 2003, Filipaj P., 2006 & Baus U., 2007]. Worthy of mention is a residential house in Chur, Switzerland, where the architect Patrick Gartmann used expanded clay and glass as lightweight aggregates to get insulating concrete with heat conductivity of \( \lambda = 0.32 \text{ W/mk} \) and concrete strength of LC 8/9. Even lighter concrete mixes using only expanded clay are used in shipbuilding and were developed by Professor Christian Thienel [Thienel K.C., et. al., 2007] of the “Universität der Bundeswehr” in Munich. Inspired by the Swiss house and based on the Munich findings the departments of “Conceptual and Structural design” and “Construction and Building Material Testing”, both belonging to the Institute of Structural engineering at the “Technische Universität Berlin”, started in the summer of 2006 to jointly develop ILWC with very low thermal conductivity [Schlaich M., et. al., 2007].

The concrete mix that was developed at TU-Berlin consists only of water, cement, light expanded clay as a lightweight aggregate, and an air-entraining agent. The concrete strength of this mix comes close to that of a lightweight concrete LC 8/9. Mixes which yield a closed fair-faced concrete surface, a dry density of \( \rho_{\text{dry}} < 800 \text{ kg/m}^3 \), and a thermal conductivity of \( \lambda_{\text{dry},10} < 0.2 \text{ W/mk} \), are consistently achieved in the concrete lab at TU-Berlin.

In Berlin, a recently built single family house with outside walls made of ILWC proves the practical value of this material. It was an interesting challenge to adjust typical structural and insulation details used for normal concrete to the properties of this material. To reduce the unavoidable cracks due to shrinkage, glass-fibre reinforcement bars were used. Glass-fibre reinforcement also solves the corrosion problem conventional steel reinforcement might have in a porous material such as infra-lightweight concrete. Further, compared to steel reinforcement fewer thermal bridges are produced, however this effect was not quantified.
The experience gained so far shows that ILWC allows well insulated fair-faced concrete buildings (Fig. 3.1), and that it has the potential to play a role in the future of building with concrete. In this chapter, the properties of ILWC and the experience gained with this material will be presented. Aspects of conceptual and structural design with ILWC will be discussed in the last section.

3.2 Manufacturing Process

To develop a new concrete mix is a challenge. In the initial excitement too many different additives were used which led to strong aggregate segregation during the initial tests (Figure 3.2, left photos). This was mainly due to large quantities of plasticizer in combination with the air-entraining agent. The reduction to only a few key components led to a stable mixture. The cut specimen (Figure 3.2, right) shows the equal distribution of expanded clay aggregates that was achieved by the mix given in Table 3.1.

<table>
<thead>
<tr>
<th>Material</th>
<th>Weight [kg/m³]</th>
<th>Volume [L/m³]</th>
</tr>
</thead>
<tbody>
<tr>
<td>Cement CEM III-A 32.5</td>
<td>330</td>
<td>108</td>
</tr>
<tr>
<td>Light sand* 0/2</td>
<td>200</td>
<td>158</td>
</tr>
<tr>
<td>Liapor* 1/4</td>
<td>25</td>
<td>30</td>
</tr>
<tr>
<td>Liapor* 2/9</td>
<td>170</td>
<td>315</td>
</tr>
<tr>
<td>Water</td>
<td>165</td>
<td>165</td>
</tr>
<tr>
<td>Air-entraining agent</td>
<td>2.0</td>
<td></td>
</tr>
</tbody>
</table>

*: Expanded clay aggregate

The goal of weight reduction for improving thermal insulation properties was reached by using approaches that are usually not taken as they reduce the strength of the material. However, for the given field of application the reduced compression strength was still sufficient because it is still higher than that of masonry. By making the following “mistakes” ILWC is obtained:
- Lightweight aggregates such as expanded clay or foam glass lead to a high proportion of air pores but unfortunately to relatively low strength.
- Generous amounts of air-entraining agent. The limit is reached when the concrete surface shows too many pores.
- Low cement content, which not only has a positive effect on the dry bulk density, but also on the temperature of hydration which reduces early age shrinkage. Low cement means also saving primary energy [Schlaich M., et. al., 2008].

3.2.1 Fresh and dry density

The fresh density for concrete can be obtained by determining the mass of a certain fresh concrete condensed volume. According to DIN EN 12390-7, the dry density of concrete can be obtained experimentally by determining the mass of a known volume of a dried concrete sample (Equation (3.1)). The concrete sample should be dried in oven with (105 ± 5) °C till its mass become constant.

\[ \rho = \frac{m}{V} \]  

where:
\( \rho \) = density of the body
\( m \) = mass of the body
\( V \) = volume of the body

- Infra-lightweight concrete
- Lightweight concrete
- Normal concrete
- Heavy concrete

800 kg/m³  \( \rho_{dry} = 2000 \text{ kg/m}^3 \)  2600 kg/m³

Why the term infra-lightweight concrete (ILWC)? The German code DIN 1045-1 defines lightweight, normal, and heavy concretes according to their densities. Lightweight concrete is defined as concrete with dry densities in the range 800 – 2000 kg/m³. Schlaich M., et. al., (2008) defined particularly lightweight concrete as that which has dry densities below the 800 kg/m³-limit as ILWC, adding the Latin preposition “infra” which means “below”. Rather than the low weight, ILWC has good thermal properties resulting from the high air void content of the concrete that is of interest here.

3.2.2 Workability and concrete consistency

Concrete consistency as well as workability can be measured by the concrete flow test, a simplistic measure of the plasticity of a fresh batch of concrete following DIN EN 12350-5 test standards. Concrete flow is normally measured by filling a cone with a sample from a fresh batch of concrete. The cone is placed with the wide end down onto a level, non-absorptive surface of flow table. It is then filled in two layers of equal volume, with each layer being tamped in order to consolidate the layer. When the cone is carefully lifted off, the enclosed material will flow and slump a certain amount due to gravity. After 30 seconds the
flow table is lifted from one edge with (40 ± 1) mm and let to be free fall down. After 15 times repeated free fall down of the edge of flow table, there are some aspects such as segregation of aggregates or bleeding of cement past should be noted. The maximum width of the flow sample is measured in two perpendicular directions. The average width for a tested sample is always in the range from 340 mm to 620 mm. The following figures show the flow test for ILWC.

![Figure 3.4: Slump test for infra-lightweight concrete.](image)

To reduce the formation of pores on the surface of ILWC, a concrete consistency with a water/cement (w/c) value of 0.5 and a width of flow test of about 600 mm was chosen. Naturally, such a semi-liquid concrete can easily be compacted.

### 3.3 Material Properties

As the ILWC is a new material, its mechanical properties are evaluated according to German codes. The mechanical properties for ILWC; such as compressive strength, modulus of elasticity, flexural tensile strength, and splitting tensile strength will be presented in the following sections. Table 3.2 contains the summarized mechanical properties for ILWC.

<table>
<thead>
<tr>
<th>Property Description</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Mean cube compression strength, $f_{cm,cube}$</td>
<td>7.40* N/mm²</td>
</tr>
<tr>
<td>Standard deviation (δ) for 20 cubes (150 mm x 150 mm x 150 mm)</td>
<td>0.63 N/mm²</td>
</tr>
<tr>
<td>Characteristic cylinder compression strength, $f_{ck,cyl} = f_{cm,cyl} - 1.48 \delta$</td>
<td>6.00 N/mm²</td>
</tr>
<tr>
<td>Experimental flexural strength, $f_{fc,fl}$</td>
<td>0.95 N/mm²</td>
</tr>
<tr>
<td>Experimental splitting tensile strength, $f_{fc,sp}$</td>
<td>0.55 N/mm²</td>
</tr>
<tr>
<td>Calculated tensile strength, $f_{cm}$ [DIN 1045-1]</td>
<td>0.66 N/mm²</td>
</tr>
<tr>
<td>Modulus of elasticity, $E_{cm}$</td>
<td>4000 N/mm²</td>
</tr>
<tr>
<td>Concrete consistency (flow test diameter)</td>
<td>600 mm</td>
</tr>
<tr>
<td>Fresh density</td>
<td>1.000 g/cm³</td>
</tr>
<tr>
<td>Dry density</td>
<td>0.760 g/cm³</td>
</tr>
<tr>
<td>Dry Shrinkage strain after 2 years</td>
<td>0.90 mm/m</td>
</tr>
<tr>
<td>Creep factor under sustained stress of 0.5 $f_{ck}$ at 190 days</td>
<td>4.50</td>
</tr>
<tr>
<td>Thermal conductivity, $\lambda_{dry,10}$</td>
<td>0.181 W/mK</td>
</tr>
</tbody>
</table>

* : mean cube compression strength for ILWC cubes on site.
3.3.1 Compressive strength

The compressive strength of ILWC is tested for 20 cubes (150 mm x 150 mm x 150 mm) according to DIN 1045-1.

![Figure 3.5: Development of infra-lightweight concrete compression strength.](image)

The ILWC specimens were stripped 6 days after casting and cured in water with 22 °C until the test. ILWC has an average 28-day cube compression strength of 7.40 N/mm², which increases to 8.40 N/mm² after 56-day as shown in Figure 3.5. The characteristic cylinder compression strength of 6.00 N/mm² is obtained as mentioned in Table 3.2.

3.3.2 Modulus of elasticity

The modulus of elasticity for ILWC is determined on cylinders (Ø150/300 mm³) corresponding to the European standard DIN EN 206-1 and DIN 1045-1. The modulus of elasticity reaches $E_{lc,m} = 4000$ N/mm², which is in tune with the typically low values for LWC according to DIN 1045-1 as shown in the following equations:

\[
E_{cm} = 9500 \left( f_{ck,cyl} + 8 \right)^{1/3} \tag{3.2}
\]

\[
E_{lc,m} = \eta_E \cdot E_{cm} \tag{3.3}
\]

\[
\eta_E = \left( \rho/2200 \right)^2 \tag{3.4}
\]

where:

- $E_{cm}$ = modulus of elasticity for normal concrete
- $f_{ck,cyl}$ = cylinder characteristic compression strength
- $\eta_E$ = correction factor
- $E_{lc,m}$ = modulus of elasticity for infra-lightweight concrete
3.3.3 Flexural and splitting tensile strength

The splitting tensile strength test was applied on cylinders (Ø150/300 mm³) corresponding to the European standard DIN EN 206-1 and DIN 1045-1. The splitting tensile strength for ILWC can be calculated enclosed to DIN 1045-1 as shown in the following equations:

\[ f_{lctm} = \eta_1 \cdot 0.3 \cdot f_{lck}^{(2/3)} \]  
\[ f_{lct, sp} = \frac{f_{lctm}}{0.9} \]  
\[ \eta_1 = 0.4 + 0.6 \frac{\rho}{2200} \]

where:
\( \rho \) = dry density  
\( \eta_1 \) = correction factor  
\( f_{lck} \) = cylinder characteristic compression strength of ILWC  
\( f_{lct, sp} \) = splitting tensile strength of ILWC  
\( f_{lctm} \) = mean central tensile strength of ILWC

The experimental splitting tensile strength (Fig. 3.6) of 0.55 N/mm² was obtained, while the calculated splitting tensile strength was 0.66 N/mm². The experimental flexural strength was \( f_{lct, fl} = 0.95 \) N/mm². The results are summarized in Table 3.2.

3.4 Time Dependent Deformations

Time dependent deformations or long term deformations in concrete can be divided into two main categories; load independent deformations as shrinkage and load dependent deformations as creep.

3.4.1 Shrinkage

In order to reduce the heat of hydration, CEM III-A 32.5 was used in the mixture of ILWC. CEM I together with the good insulation characteristics of the material caused temperatures of hydration of up to 90 °C inside the specimen. In addition, the use of CEM III leads to a light grey colour, which in the summer reduces warming of the surface.
Particularly striking are the comparatively high values of shrinkage and creep and the low modulus of elasticity, which need to be considered during design. Figure 3.7 demonstrates the limits of shrinkage strains from one extensive testing program for lightweight concrete with different strengths [Holm, 1980, with permission of ACI]. The Initial tests for the ILWC specimens (40 mm x 40 mm x 160 mm) show that the end shrinkage value after 2 years reaches 0.9 mm/m. However, 70 % of this value is reached in the first 3 weeks (Figure 3.8). The specimens were kept in 20 °C and 65 % relative humidity during the test.

### 3.4.2 Creep

The deformation behaviour of concrete consists of an instantaneous elastic part and a time dependent part as shown in Fig. 3.9. The time dependent part consists of creep (load induced strain) and shrinkage.

The first creep test for ILWC shows that the initial high increase in deformation slows down significantly after about 50 days. Figure 3.10 shows the time dependent deformations for sustained loads equivalent to 30 % and 50 % of the characteristic compressive strength ($f_{ck}$). The samples (40 mm x 40 mm x 160 mm) were loaded at concrete age of 28 days and are kept in temperature of 20 °C and 65 % relative humidity during the test. Creep is generally indicated with the creep factor that can be calculated as:
\[ \phi(t_o) = \frac{\varepsilon_{cr}(t)}{\varepsilon_{el}(t_o)} \]  

(3.8)

where:

\( \phi (t_o) \) = creep factor
\( \varepsilon_{cr} (t) \) = creep strain part of the time dependent deformations at time \( t \)
\( \varepsilon_{el} (t_o) \) = instantaneous elastic deformation at time \( t_o \) of application of the loading

The creep factor after 190 days for ILWC considering the creep part only of the time dependent deformations (Fig. 3.10) was 4.5 for sustained load of 50 % of the characteristic compressive strength.

3.5 Durability

The durability of concrete can be defined as its ability to resist external influences such as climatic conditions, environmental exposure, chemical attack and mechanical damage. The great importance of permeability in view of durability problems constitutes the reason why a brief summary on permeability of LWAC is included here.

3.5.1 Porosity and permeability

The porous structure of the matrix of ILWC, the porosity of the expanded clay aggregates, and the micro cracks from shrinkage deformations are the main factors that enable water to penetrate ILWC.

![Figure 3.11: Porosity and permeability, schematic representation of the difference [Stutech, 1992].](image)

Figure 3.11 shows the difference between porosity and permeability in a schematic way. The figure explains that connectivity in the pore system is a prerequisite for permeability. A material can be porous but still perform tight, namely as long as the pores are not interconnected. Knowing this, it is clear that the use of porous aggregates will not necessarily result in an increase of the permeability.
The water penetration test according to DIN EN 12390-8 was carried out to determine the water penetration depth for ILWC. The specimens of 200 mm x 200 mm x 165 mm were loaded with 5-bar water pressure on area of 75 mm diameter on one face (200 mm x 200 mm) of each specimen. After 72 hours, the specimens were removed from the machine and directly broken at the middle to see and measure the average penetration depth that reached 40 mm. Figure 3.12 shows the test setup and water penetration depth. In the fact, before this test, the real application of ILWC in the external walls of a family house in Berlin was applied. A hydrophobic material was used to reduce the ability of the external fair-faced surface of ILWC walls to absorb water.

3.5.2 Freeze-thaw resistance

The capillary suction and weathering scale for ILWC due to freeze-thaw cycles were investigated according to CF-test (Capillary suction and Freeze-thaw) in DIN EN 12390-9.
The temperature in one cycle changes from +20 to -20 °C in 12 hours (Fig. 3.13b). After 28 such cycle, the ILWC had a weathering loss of only 340 g/m². The allowable weathering value for NC is 1500 g/m², while for LWC is 1500 (ρ/2.1) as documented by Faust T., (2003) i.e. for ILWC with dry density of 760 Kg/m³, the allowable limit of weathering is 540 g/m². This means that the cement paste due to severe winter conditions is not a threat with ILWC. Figures 3.13c & 3.13d show the capillary suction values for five specimens of ILWC before the start of temperature cycles and the weathering scale for the tested specimens, respectively.

3.6 Physical Properties

There are many thermal properties such as thermal conductivity, specific heat, and coefficient of thermal expansion. The thermal conductivity is considered the most important property, when ILWC is considered as an insulating concrete.

3.6.1 Thermal conductivity - heat transfer

The moisture content of the concrete has a significant influence on the thermal conductivity. For a given dry density, the thermal conductivity will increase as the moisture content rises. This is often described by a moisture correction factor. Table 3.3 shows the moisture correction factor for the thermal conductivity in different codes.

<table>
<thead>
<tr>
<th>Code</th>
<th>Correction Factor</th>
</tr>
</thead>
<tbody>
<tr>
<td>ACI-213</td>
<td>+5.0 % for each 1.0 % by weight</td>
</tr>
<tr>
<td>France and UK codes</td>
<td>+11 % for each 1.0 % by volume</td>
</tr>
<tr>
<td>German code</td>
<td>+6.0 % for each 1.0 % by volume</td>
</tr>
</tbody>
</table>

The discrepancy between these various values will vary at different densities. Thus, even though the thermal conductivity of LWC is often presented with standard moisture content of typically 3.0 %.

![Figure 3.14: Relationship between thermal conductivity and dry density for LWAC (DIN 4108-4, 2002).](image)

The typical thermal conductivity - dry density curve for LWC according to DIN 4108-4 is shown in Figure 3.14. The figure shows that for LWC with dry density of 760 Kg/m³, a thermal conductivity of \( \lambda = 0.27 \) W/mK is obtained.
With great excitement the results of the thermal conductivity test for ILWC were expected. The experiment for two samples 500 mm x 500 mm x 50 mm was conducted in the laboratory of the Material Testing Agency (MPA) in Berlin and resulted in a value of thermal conductivity of $\lambda_{\text{dry,10}} = 0.181$ W/mK. The thermal conductivity value depends on the mean temperature of the samples. This relationship is shown in Figure 3.15.

The theoretical heat transfer coefficient ($U$-value for completely dry samples) according to Equation (3.9) for ILWC external wall with thickness $S = 0.5$ m and $\lambda_{\text{dry,10}} = 0.181$ W/mK is equal to 0.34 W/m²K.

$$U = \frac{1}{\frac{1}{\alpha_i} + \frac{S}{\lambda} + \frac{1}{\alpha_a}} \quad (3.9)$$

where:

$U = \text{heat transfer coefficient}$

$S = \text{wall thickness}$

$\lambda = \text{thermal conductivity}$

$\frac{1}{\alpha_i} = \text{inner heat transfer resistance and equals 0.13 m²K/W according to DIN 4108-4}$

$\frac{1}{\alpha_a} = \text{outer heat transfer resistance and equals 0.04 m²K/W according to DIN 4108-4}$

### 3.7 Bond Behaviour

Infra-lightweight concrete is a new engineered material of which its potential and several design aspects have not yet fully exploited. Among them is the behaviour of the bond between ILWC and different reinforcement bars.

![Beam cross-section](image-url)
Every year at the department of structural engineering at the “Technische Universität Berlin”, a structural normal concrete beam with normal reinforcement (SRFT) is tested as a study case for the students in 4th semester of structural engineering. In 2007 it was a beam constructed with ILWC with 200 mm x 400 mm cross-section, 2.0 m clear span, and reinforced with longitudinal GFR bars. Due to practical reasons the transverse reinforcements were normal steel stirrups. By using one concentrated load at the mid span, the beam failed at 90 KN. In spite of the good arrangement of shear reinforcement the cracked beam seemed to be cracked in shear as shown in Figure 3.17. It was clear that the bond between ILWC and longitudinal GFR bars was relatively small. Therefore, the bond behaviour between ILWC and different types of reinforcement such as SRFT, GFR, and GFR with a head bolt (Figure 3.18) is investigated and discussed in the following sections.

3.7.1 Introduction and previous work

A literature review shows that little experimental data is available for bond in LWC, and even less for GFR bars in ILWC and LWC. Table 3.4 shows a summary of results for some researches that investigated the bond strength and the development length between GFR bars and normal concrete with different compression strengths.

Table 3.4: Previous test results for bond strength between GFR and normal concrete

<table>
<thead>
<tr>
<th>Reference</th>
<th>Cosenza et al., 1999</th>
<th>Shield et al., 1999</th>
<th>Nanni et al., 1997</th>
<th>Rizkalla et al., 1997</th>
</tr>
</thead>
<tbody>
<tr>
<td>Bar diameter, $d_b$</td>
<td>12.7</td>
<td>15.10</td>
<td>13.0</td>
<td>12.0</td>
</tr>
<tr>
<td>Bar strength</td>
<td>770</td>
<td>427</td>
<td>568</td>
<td>640</td>
</tr>
<tr>
<td>Concrete strength</td>
<td>39.0</td>
<td>29.7</td>
<td>34.5</td>
<td>44.0</td>
</tr>
<tr>
<td>Bond strength</td>
<td>14.5</td>
<td>4.50</td>
<td>17.0</td>
<td>21.3</td>
</tr>
<tr>
<td>Development length</td>
<td>10 $d_b$</td>
<td>24 $d_b$</td>
<td>5 $d_b$</td>
<td>15 $d_b$</td>
</tr>
<tr>
<td>Specimen size</td>
<td>127x150 x150</td>
<td>381x305x457</td>
<td>150x150x150</td>
<td>150x150x150</td>
</tr>
</tbody>
</table>

Because of the relatively small young’s modulus of infra-lightweight concrete and its small tensile strength, polypropylene fibres were used to investigate the effect of splitting tensile strength on the bond behaviour between ILWC and different types and shapes of reinforcement bars. 27 Specimens were tested to understand the bond behaviour in fibred and non-fibred ILWC using different confinement ratios.

The main objective in the following sections is to investigate the bond behaviour between ILWC and different types of reinforcement such as GF and SRFT bars. Another objective is to improve the bond-slip relationship in ILWC by enhancing the tensile strength of ILWC using polypropylene (PP) fibres or using confinement stirrups. For these objectives 27 ILWC specimens were tested as pull-out test. The study investigated that the ribs on the surface of the bars play the main role on the bond-slip relation. Improving of radial tensile strength of ILWC by using confinement stirrups or by adding PP fibres with length not less than 20 mm, enhances the bond behaviour, especially with bars that have more ribs per unit length. However, using PP fibres reduces the slip at the maximum bond stress, which is required to control the crack width in serviceability limit state [El Zareef M., et. al., 2008].

3.7.2 Steel reinforcement and glass fibres reinforcement

High alkalinity of cement matrix, low permeability and concrete cover play the main role to protect steel reinforcement from corrosion. Using infra-lightweight concrete as a structural material has many advantages such as low thermal conductivity and low self-weight. On the
opposite side, it has a tendency to absorb water and its carbonation depth is relatively big. Therefore, using GFR bars as non-corrosion reinforcement in ILWC walls is a good solution to solve these problems.

Figure 3.18: Types of tested SRFT & GFR.

Figure 3.19: Stress-strain curve for SRFT and GFR.

Strength and rigidity of GFR bars can be defined according to the kind, number and adjustment of the glass fibres. GFR behaves linear-elastic up to fracture. Yielding of the material does not happen. The material has relatively low compression and tensile strength perpendicular to the fibres [Schoeck Combar, 2009]. The shape and the stress-strain curves for the used reinforcement in this chapter are shown in Figures 3.18 & 3.19. The head bolt for the GFR bars has a 48 mm base-diameter, which is reduced to the bar diameter through 100 mm length. Table 3.5 shows the properties for the used reinforcement in the study.

Table 3.5: Properties of reinforcement

<table>
<thead>
<tr>
<th>Material properties of straight bars</th>
<th>Steel reinforcement SRFT 500, DIN 488</th>
<th>Glass-fibre reinforcement GFR Combar</th>
</tr>
</thead>
<tbody>
<tr>
<td>Tensile strength, $f_t$</td>
<td>550</td>
<td>1000</td>
</tr>
<tr>
<td>Characteristic yield strength, $f_y$</td>
<td>500</td>
<td>1000</td>
</tr>
<tr>
<td>Designed yield strength, $f_{yd}$</td>
<td>435</td>
<td>435*</td>
</tr>
<tr>
<td>Strain at yield stress, $\varepsilon_y$</td>
<td>2.18</td>
<td>7.25**</td>
</tr>
<tr>
<td>Tension modulus of elasticity, $E$</td>
<td>200000</td>
<td>600000</td>
</tr>
<tr>
<td>Concrete cover</td>
<td>acc. DIN 1045-1</td>
<td>$d_b + 10$</td>
</tr>
<tr>
<td>Density, $\rho$</td>
<td>7.85</td>
<td>2.20</td>
</tr>
<tr>
<td>Thermal conductivity, $\lambda$</td>
<td>60</td>
<td>0.50</td>
</tr>
</tbody>
</table>

*: Design stress for the GFR bars (no yield).
**: Design strain for the GFR bars at design stress of 435 N/mm².

3.7.3 Pull-out test specimens

In order to test the behaviour of ILWC in bond with different types of reinforcement bars, a concrete specimen of 150 mm x 150 mm x 150 mm was used with contact length between the reinforcement bar and the concrete of five times the bar diameter ($d_b$).
A special mould was produced to ensure the rebar position during casting (Figure 3.20). The free length of the bar was 300 mm from the active end and 50 mm from the dead end. In order to ensure a good contact in the case of GFR with a head bolt the head was embedded at the mid height of concrete specimen. Styrofoam was used to reduce the contact length. The concrete specimen was fixed in the machine as shown in Figure 3.21, and the bar was pulled out with a loading rate of 0.005 mm/s. The slip was measured from the dead end of the bar.

### 3.7.4 Experimental analysis

#### 3.7.4.1 Effect of different reinforcement bars on bond behaviour

At the same concrete strength, the depth of ribs as well as the distance between them plays the main role to determine the relation between bond stress and slip.

In Fig. 3.22, the maximum bond stresses were 0.87 MPa at 0.703 mm in the case of GFR, 1.04 MPa at 0.169 mm in the case of SRFT, and 1.99 MPa at 13.86 mm in the case of GFR with head bolt. In the first two cases, the bond stress depends on the distribution of compression stress on the surrounding matrix, which is better in the case of SRFT because of the large number of ribs per unit length as shown in Figures 3.23a & 3.23b.
In other words, the number of ribs per unit length in the case of SRFT is 25% more than that in the case of GFR, which increases the maximum bond stress compared to GFR by 20%. After the bond stress reaches its maximum level, the bond stress depends on the friction between the outer surface area of ribs and the surrounding matrix which is smaller in the case of SRFT than that in the case of GFR by 53%, this leads to the suddenly drop in the bond stress in the case of SRFT after the maximum bond stress reaches. Using head bolts at the end of GFR enhances the transition of the force from the bar to surrounding concrete. The bond stress in this case depends mainly on the compression and tensile strength of the concrete (Figure 3.23c). Using the head bolt with GF bars increases the slip at maximum bond stress by about 20 times and increases the maximum bond stress more than double.

3.7.4.2 Effect of polypropylene fibres on bond behaviour

In order to improve the splitting tensile strength of ILWC, PP fibres (1.0 kg/m³) were used with different lengths; 6, 12, and 20 mm, which increased the tensile strength with 10%, 23%, and 30% respectively. The long PP fibres increase the amount of energy being absorbed in the de-bonding of the fibre from the concrete matrix prior to the complete tension failure of concrete. On the other hand, adding PP fibres reduced the compact ability of the cement matrix and in consequence reduced the compressive strength with 56%, 43%, and 41% respectively. Therefore, using of long PP fibres is more suitable than small ones, especially in low strength materials.

Adding the 6 mm PP fibres to ILWC reduced the maximum bond stress in the cases of GFR and SRFT by 37% and 2.4% respectively, while adding the 20 mm PP fibres increased the maximum bond stress in the cases of GFR and SRFT by 4.6% and 25.3% respectively. In the case of 20 mm PP fibres, the reduction of compressive strength balanced with the increasing of the tensile strength (Figures 3.24 & 3.25). Another important note is that using of PP fibres
reduced the slip at maximum bond stress especially in the case of GFR, which is better for crack width control in serviceability limit state.

![Figure 3.26: Bond stress versus slip for GFR with head bolt with different lengths of pp fibres.](image)

As it shown in Figure 3.26, adding PP fibres to the mixture of ILWC has a negative effect on the bond behaviour in the case of GFR with head bolt. In this case, the bond stress depends on the compressive strength of ILWC (Figure 3.23c), which is reduced due to the loss of workability after adding PP fibres.

### 3.7.4.3 Effect of confinement on bond behaviour

Another effective method was used to improve the radial tensile strength in the case of GFR with head bolt (Figure 3.23c). The stirrups are used as a confinement tools with different numbers (1, 2, and 3 stirrups with 12 cm side length). Using the stirrups in the cases of SRFT and GFR bars without head bolt has no significant effect on the bond behaviour.

![Figure 3.27: Bond stress versus slip for GFR with head bolt using different numbers of stirrups.](image)

The effect of using different confinement ratios on the behaviour of bond in the case of GFR with head bolt is shown in Figure 3.27. The slope of the bond stress-slip curve after the maximum bond stress as well as the number of cracks and crack width are reduced by increasing the confinement ratio. In the case of using 3 stirrups, the first crack starts at slip of 55 mm, while it starts after 12 mm slip in the case of using no stirrups.
The previous experimental analysis for the bond behaviour in ILWC concludes that at the same concrete strength, the configuration and the number of the ribs on the surface of the bars play the main role on the relationship between bond stress and slip. Optimizing the shape of ribs and using additional parts such as the head bolt (Fig. 3.23c) are essential to enhance the bond behaviour through the good transition of the force from the bars to concrete. Using of PP fibres with length not less than 20 mm in ILWC improves the bond behaviour especially with bars that have more numbers of ribs per unit length as in SRFT. Adding PP fibres reduces the slip between the bars and ILWC at the maximum bond stress, which is required to control the crack width in serviceability limit state. Using the confinement stirrups reduces the number and the width of cracks especially in the case of GFR with head bolt, where the ring tensile strength is more effective than in the cases of SRFT and GFR without head bolt. The slope of the bond stress-slip curve after the maximum bond stress is reduced by increasing the confinement ratio (Fig. 3.27).

3.8 Handling and Construction

After the previous intensive experimental tests for the ILWC mixture, its real application is carried out through the casting of external fair-faced walls for a family house in Berlin in 2007. The reproduction of laboratory test results on site is not an easy task. However, the mixture proved to be surprisingly stable. The concrete was ordered with slightly smaller slump test width than needed and on site the desired value was obtained by adding small amounts of water to the mix.

Achieving good surface quality was particularly difficult. Different formwork types and different formwork release compounds were tested. For the building described here, simple and new concrete planning tables without any release agent were used for each cast. A sample wall was cast in order to investigate issues such as the influence of climatic conditions on site and the time the mix remained in the transport vehicles.

Since pumping of light-weight concrete with expanded clay concrete is considered to be difficult, for this project the concrete was cast using a concrete bucket. To minimize the height of fall, an appropriate tube was attached to the bucket (Figure 3.28, right photo). The 50 cm thick exterior walls were cast story-wise in layers of approximately 50 cm. A conventional concrete vibrator placed at distances equal to five times its diameter was used. No clear optimum for vibration time and distances could be identified. Mix stability and surface quality were surprisingly independent of the variation of these parameters. Only very long vibration times led to desegregation. To eliminate potentially segregated material, some extra concrete was poured and the upper few centimeters of the concrete wall were skimmed after the end of vibrating. Stripping of the forms took place after at least 7 days. For concrete curing, the walls were covered with plastic foil.
To reduce the formation of pores on the surface, a concrete consistency with a $w/c$ ratio of 0.5 and a width of flow-test of about 60 cm was chosen (Figure 3.28, left photo). Naturally, such a semi-liquid concrete can easily be compacted.

### 3.9 Structural Details

The large shrinkage and creep values of infra-lightweight concrete require a structural design that leads to as little restraining forces as possible [Schlaich M., et. al., 2008]. The non-reinforced samples in the laboratory showed that after a short time few but large shrinkage cracks occurred. In the exterior walls of the house, glass-fibre reinforcement bars (inside and outside $d = 8$ mm spaced at $a = 15$ cm horizontally and vertically) were used, which led to good crack distribution. So far, all crack widths remain well below 0.1 mm after two years from the construction.

The low modulus of elasticity, in conjunction with strong creep and shrinkage must be carefully considered when relative deformations are analyzed. “Soft” external walls of infra-lightweight concrete may shorten considerably more than a “hard” building core from normal concrete. Due to the low strength of the material, the walls must be treated like masonry walls. A normal concrete ceiling slab cannot be rigidly fixed to a wall of infra-lightweight concrete. The different structural details of the house were developed accordingly.

The deck slab-to-wall connection in Figure 3.30 shows that additional insulation is needed because the ceiling slab reduces the thickness of the insulating wall. The edge of the slab was insulated with foam glass panels as well as at the mullion transom facade. Such panels were inserted in the wall to reduce the negative effect of thermal bridges (Figure 3.31).
For reasons of expediency and contrary to what is shown in Figure 3.32 – for the flat roof a slab of normal concrete (C 20/25) with insulation on top was built. Only the fascia at the cantilevering front face of the roof is made of infra-lightweight concrete. It is connected via steel stirrups to the roof slab. External louvers as sun and light protectors are fixed under the cantilever. The small bathroom windows in the north facade are placed flush into the voids of the ILWC walls. Together with interior wood panels, they provide sufficient insulation.

### 3.10 Prospects

Infra-lightweight concrete does not have enough strength to be used for load bearing deck slabs. A possible layout of a house with low energy consumption could be as shown in Figure 3.35; Infra-lightweight concrete for the exterior walls, lightweight concrete with good insulation properties for the roof-slab (will be discussed in the following chapters), and normal concrete with good heat storage capacity for the interior ceilings and walls.

The studies in this chapter show the limits of infra-lightweight concrete, but also its great potential for building with insulated fair-faced concrete. On the other hand the question of what minimum value of thermal conductivity can be reached with lightweight concrete, to be used for load-bearing roof-slabs, is investigated and presented in the following chapter.
4.1 Introduction

Even if the infra-lightweight concrete that is presented in the previous chapter is improved to get its maximum strength without detrimental effects on its favourable thermal properties, it does not have enough strength to be used for load bearing deck slabs. Therefore, more than 100 trials were done in the concrete lab to get a new lightweight concrete mixture with accepted workability, good thermal conductivity, and desirable strength enough to use it in construction of deck slabs and beams. The new lightweight concrete is developed with dry density of 1.25 g/m³, thermal conductivity of 0.33 W/mK, and concrete strength class of LC 30/33.

The high cost of reinforcing materials such as steel reinforcement, carbon reinforcement, and glass-fibre reinforcement bars makes it desirable to reduce the required amount of reinforcement especially in the case of tall structures such as high-rise buildings and long span bridges, where the high dead load of the structure leads to large amounts of reinforcement. The LWC is one option that may participate to reduce this problem.

LWC has been successfully used in construction for a long time. However, so far its weight could not be reduced to 50% of the weight of normal concrete without losing its strength. Normal lightweight concrete are only 25% to 35% lighter than normal concrete (Table 2.1).

The following sections will present the mechanical and thermal properties for this new LWC mixture that has a dry density of about 50% as normal density concrete and concrete strength comparable to NC. The series of static and dynamic experimental tests on beams and beam-column joints constructed from this new LWC mixture will be presented and discussed in the following chapters to investigate its behaviour in structural elements.

4.2 Manufacturing Process

In order to get this new fair-faced LWC with minimum dry density and maximum concrete strength, more than 100 trials were done taking into account the following aspects:

- The simplest possible way of modelling hardened concrete is to consider the material as a two-phase composite consisting of a matrix phase and a particle phase. The matrix phase then can be defined as cement paste, and the particle phase consists of the aggregates. The mechanical properties of the material are determined by the mechanical properties of each phase and the interaction between the phases [Smeplass, 1992]. Therefore, three sizes of well-graded lightweight expanded clay aggregates were chosen to get maximum global compressive strength for aggregates and minimum global aggregate density.

- In order to increase the strength of the cement paste, CEM I 52.5 was used, but on the opposite side it has a high temperature of hydration. A replacement of some amount of cement with silica fume would reduce the hydration generated temperatures.
- Improving the matrixaggregate interfacial zone, decreasing the permeability, and improving the bond between concrete and reinforcement bars, which are achieved by adding silica fume to the LWC mixture.

- Dry LWA absorbs some of mixing water and reduces the water/cement ratio at the early stage after batching. As a consequence, micro bleeding at the aggregate surfaces is also prevented. This absorbed water in the LWA will feed the cement paste with water in later stages providing “internal curing”.

In the following table the final contents of LWC mixture depending on the previous aspects are presented.

<table>
<thead>
<tr>
<th>Material</th>
<th>Weight [kg]</th>
<th>Volume [dm³]</th>
</tr>
</thead>
<tbody>
<tr>
<td>Cement CEM I 52.5</td>
<td>400</td>
<td>130</td>
</tr>
<tr>
<td>Silica fume</td>
<td>32</td>
<td>20</td>
</tr>
<tr>
<td>K sand 0/2</td>
<td>333</td>
<td>195</td>
</tr>
<tr>
<td>Liapor 2/9E</td>
<td>59</td>
<td>109</td>
</tr>
<tr>
<td>Liapor 6/5</td>
<td>425</td>
<td>360</td>
</tr>
<tr>
<td>Water</td>
<td>255</td>
<td>255</td>
</tr>
<tr>
<td>Super-plasticizer</td>
<td>4.0</td>
<td>3.7</td>
</tr>
</tbody>
</table>

*: Expanded clay aggregate

4.2.1 Fresh and dry density

According to DIN 1045-2 and DIN EN 206-1, the density class for LWC is depending on its dry density. The calculated value of dry density and the corresponding characteristic value of density for LWC to be used in the design according to DIN 1045-1 are shown in Table 4.2.

<table>
<thead>
<tr>
<th>Density class, Calculated dry density, and characteristic value of density for LWC</th>
</tr>
</thead>
<tbody>
<tr>
<td>Density class</td>
</tr>
<tr>
<td>---------------</td>
</tr>
<tr>
<td></td>
</tr>
<tr>
<td>D 1.0</td>
</tr>
<tr>
<td>D 1.2</td>
</tr>
<tr>
<td>D 1.4</td>
</tr>
<tr>
<td>D 1.6</td>
</tr>
<tr>
<td>D 1.8</td>
</tr>
<tr>
<td>D 2.0</td>
</tr>
</tbody>
</table>

The dry density of LWC can be theoretical approximately calculated according to German Association of Cement Production as in the following equation.

$$\rho \approx \frac{1.2 C + a_{dry}}{1000}$$

(4.1)

where:

\(\rho\) = dry density of lightweight concrete

\(C\) = cement content

\(a_{dry}\) = dry weight of aggregates and additional materials such as silica fume

Equation (4.1) is based on that an amount of water equal to 20 %\(\text{M}\). of cement content will be involved in the hydration process.
Theoretically, the fresh density of 1407 kg/m³ can be calculated from Table 4.1. Experimentally, the fresh density of the studied LWC reaches 1450 to 1470 kg/m³. This difference between the theoretical and experimental values may be due to the effect of some factors such as the air voids content and aggregate volume changes due to water absorption. The dry density of 1250 to 1270 kg/m³ for the studied LWC mixture was observed in the lab according to DIN EN 12390-7.

### 4.2.2 Workability and concrete consistency

The workability or consistency class for concrete is classified according to DIN 1045-2 as shown in Table 4.3. The accepted concrete consistency class for cast-in-site concrete should be not less than F3 as recommended by DIN 1045-2, however the consistency class for the studied LWC was in the range of 470 to 500 mm, i.e. between F3 and F4.

<table>
<thead>
<tr>
<th>Consistency class</th>
<th>Spread value (diameter) [mm]</th>
<th>Consistency description</th>
</tr>
</thead>
<tbody>
<tr>
<td>F 1</td>
<td>&lt; 340</td>
<td>stiff</td>
</tr>
<tr>
<td>F 2</td>
<td>350 – 410</td>
<td>plastic</td>
</tr>
<tr>
<td>F 3</td>
<td>420 – 480</td>
<td>soft</td>
</tr>
<tr>
<td>F 4</td>
<td>490 – 550</td>
<td>very soft</td>
</tr>
<tr>
<td>F 5</td>
<td>560 – 620</td>
<td>flow</td>
</tr>
<tr>
<td>F 6</td>
<td>&gt; 630</td>
<td>very flow</td>
</tr>
</tbody>
</table>

### 4.3 Material Properties

The mechanical properties for the new LWC used in the study are evaluated according to German codes. The mechanical properties such as compressive strength, modulus of elasticity, flexural strength and splitting tensile strength will be presented in the following sections. Table 4.4 contains the summary for these mechanical properties.

<table>
<thead>
<tr>
<th>Property</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Mean cube compression strength, ( f_{cm, cube} )</td>
<td>40.00 N/mm²</td>
</tr>
<tr>
<td>Characteristic cylinder compression strength, ( f_{ck,cyl} )</td>
<td>31.00 N/mm²</td>
</tr>
<tr>
<td>Experimental flexural strength, ( f_{fl} )</td>
<td>2.45 N/mm²</td>
</tr>
<tr>
<td>Experimental splitting tensile strength, ( f_{fl, sp} )</td>
<td>2.27 N/mm²</td>
</tr>
<tr>
<td>Calculated tensile strength, ( f_{cm} ) [DIN 1045-1]</td>
<td>2.15 N/mm²</td>
</tr>
<tr>
<td>Modulus of elasticity, ( E_{cm} )</td>
<td>12000 N/mm²</td>
</tr>
<tr>
<td>Concrete consistency (flow test diameter)</td>
<td>485 mm</td>
</tr>
<tr>
<td>Fresh density</td>
<td>1.45 g/cm³</td>
</tr>
<tr>
<td>Dry density</td>
<td>1.25 g/cm³</td>
</tr>
<tr>
<td>Dry Shrinkage strain after one year</td>
<td>0.70 mm/m</td>
</tr>
<tr>
<td>Thermal conductivity, ( \lambda_{dry,10} )</td>
<td>0.33 W/mK</td>
</tr>
</tbody>
</table>

\( \delta \) : standard deviation of compression strength for 33 cylinders.
4.3.1 Compressive strength

33 cylinders (Ø150/300 mm³) of LWC at different mix-time were prepared and tested for compression strength according to DIN EN 12390-3. The results were statistically analysed according to DIN EN 206-1/DIN 1045-2 to get the characteristic compression strength that no more than 5.0 % of the tested samples have compression strength less than it (Equation (4.2)). The characteristic cylinder compression strength reached 31 MPa at 28 days.

\[
\begin{align*}
 \delta 
\end{align*}
\]

where:

- \( f_{ck} \) = characteristic concrete compression strength
- \( f_{cm} \) = mean concrete compression strength
- \( \delta \) = standard deviation for compression strength values

\[
(4.2)
\]

4.3.2 Modulus of elasticity

The elastic modulus of an object is defined as the slope of its stress-strain curve in the elastic deformation region. 6 cylinders (Ø150/300 mm³) of LWC were tested according to DIN 1048-5 to get their elastic modulus.

![Figure 4.1: Modulus of elasticity test set-up for LWC.](image)

The elastic strain at a low limit of elastic stress 0.5 MPa for each specimen was observed using sensitive deformation sensors that attached to the tested cylinder at its middle height (Figure 4.1). The tested cylinder is loaded till the upper limit of elastic stress (8.0 MPa < 0.33 \( f_{ck} \)) is reached. The strain at the upper limit of elastic stress is also observed. The tested cylinder is unloaded to a low limit of elastic stress equals to 0.5 N/mm² and then loaded again to the upper limit of elastic stress 8.0 MPa. The last step is repeated for 2 times. The elastic modulus for LWC is calculated as the ratio between the difference of upper and lower limits of elastic stress to the difference of corresponding upper and lower strains. The elastic modulus for this new LWC of 12000 MPa was obtained in the lab. Theoretically, an elastic modulus of 10300 MPa is calculated for this LWC according to DIN 1045-1 as shown in Equation (3.3), Chapter 3, Section 3.3.2.

4.3.3 Stress-strain curves

Stress-strain relationships for LWAC are generally characterised by a more linear ascending curve, more limited plastic strain and a steeper descending branch than NC. This will appear
In concrete with moderate strength too, if moderate density LWA is combined with high strength cementitious matrix.

In order to investigate the stress-strain relationship for the studied LWC, 3 cylinders were prepared and tested. The strain of the tested specimens was measured using 3 strain-gages for each specimen (Figure 4.2). The strain gages were attached to each cylinder at the mid height and at 120° angle between them in horizontal direction. Figure 4.3 shows the stress-strain curve for the tested 3 cylinders of LWC.

4.3.4 Tensile strength

The tensile strength of concrete is important when considering cracking. Principal differences between LWAC and NC are:

- Fracture path. This travels through rather than around the aggregate particles.
- Total water content. This is higher for LWAC so greater moisture gradients during curing (due to higher hydration temperature as well) can cause a significant reduction in tensile strength.
- Flexural strength is more affected than cylinder splitting strength [Newman, 1993].

The following codes reduce the tensile strength of LWC compared with NC of the same compression strength if the tensile strength is not determined by testing.

German code [DIN 1045-1] reduction factor:

\[ \eta_t = (0.4 + 0.6 \cdot \rho/2200) \]  \hspace{1cm} (4.3)

Norwegian code [NS 3473 E/1992] reduction factor:

\[ \eta_t = (0.3 + 0.7 \cdot \rho/2400) \]  \hspace{1cm} (4.4)

where:
\[ \rho = \text{dry density of lightweight concrete} \]

The experimental results of the new studied LWC (air curing) showed that both of codes are conservative with 12 % and 21 % respectively.
LWAC represents a flexural and splitting tensile strength slightly inferior to that of NC of the same compressive strength [Zhang, et. al., 1995]. The effect of moisture on mechanical properties has been studied by Hammer, (1993). The most important influences of moisture were found for tensile strength. For the newly studied LWC a reduction of 40 % for flexural tensile strength was observed for air curing specimens compared with water curing specimens. This attributes to that the air curing specimen lost some of its strength in shrinkage cracks. Therefore, it is recommended to keep the specimen in a foil till the tensile test and reduce the resulted tensile strength by about 40 % - 50 %.

4.4 Time Dependent Deformations

In the old version of Eurocode-2 (ENV 1992-1-4, 1996) it is denoted that for lightweight concrete the creep coefficient $\phi$ can be assumed equal to the value of normal density concrete multiplied by a factor $(\rho / 2300)^2$ for $\rho > 1800$ kg/m$^3$. For $\rho < 1500$ kg/m$^3$ a factor 1.3 $(\rho / 2300)^2$ can be used. For intermediate values of $\rho$ linear interpolation may be applied. Furthermore the creep strain has to be multiplied by a factor 1.3 for lightweight concrete classes lower than LC 20/25.

Commentary of EC-2, (2008) confirmed that a reconsideration of the formulation for creep of LWAC, as given in ENV 1992-1-4 (1996) and provisionally adopted in the version of EC-2 (2000) seems to be necessary. The best formulation seems to be that creep of LWAC is the same as creep of NC and can be calculated with the same formula’s (Figure 4.5).
In the CEB/FIP manual on LWAC the specific creep of LWAC is stated to exceed that of NC of similar mix composition by about 10 to 30 %. The creep factor ($\phi$) for LWC is related to that of NC according to:

$$\phi_{LWC} = 1.2 \frac{E_{cm}}{E_{lcm}} \phi_{NC} \tag{4.5}$$

where:

$\phi_{LWC} = $ creep factor for lightweight concrete  
$\phi_{NC} = $ creep factor for normal concrete  
$E_{lcm} = $ elastic modulus of lightweight concrete  
$E_{cm} = $ elastic modulus of normal concrete

If water saturated, or even partially saturated lightweight aggregate is used in low w/c ratio mixes, existing water in the aggregate can compensate for water shortage in the hydrating paste. The supply of water from the aggregate to the drying microstructure will prevent a significant drop of the relative humidity in the paste and will thus reduce autogenous shrinkage. This phenomenon is known as “internal curing”.

![Figure 4.6: Drying shrinkage of the studied lightweight concrete at 20 °C and 65 % RH.](image)

For LWAC made with expanded shale and expanded clay and with strength of 30 to 50 MPa, stored at 20 °C and 65 % RH, a final shrinkage of 0.5 to 0.6 ‰ has been found at Cement Industry Research Institute in Düsseldorf [CEB/FIP, 1977]. However the drying shrinkage for the studied LWC (40 mm x 40 mm x 160 mm) samples after 1 year at 20 °C and 65 % RH reached 0.7 ‰ as shown in Figure 4.6.

### 4.5 Durability

In practice rebar corrosion and freeze-thaw volume changes are the most common durability problem in structural concrete. The great importance of durability constitutes the reason why a brief summary on water penetration test and freeze-thaw resistance for the studied LWC are included here.

#### 4.5.1 Water penetration test

As the studied LWC is considered a new LWC as it has bigger strength to density ratio than other types of LWC, it was important to measure the efficiency of its concrete cover through the water penetration test according to DIN EN 12390-8.
Figure 4.7: Water penetration test for the used LWC in the study.

After 72 hours at 5 bars constant water pressure, the studied LWC specimens show maximum penetration depths not exceed 15 mm. However, in normal concrete it reaches 20 – 30 mm. The results of the water penetration test for the studied LWC were in tune with what is well known that adding of more fine particles such as silica fume to the LWC mixture improves the strength of concrete and makes the cement paste more condense.

4.5.2 Freeze-thaw resistance

As long as the water/cement ratio is kept below 0.35, no need for air-entraining agents to achieve a good frost resistance was reported by Heimdal E., (1995). For the LWC used in the study, the water/cement ratio of 0.32 was applied without using of air-entraining agents. The air voids within the expanded aggregate acts as additional air voids so that LWAC can in some instances have slightly better freeze-thaw durability than for the high density material [Portland Cement Association].

The CF-test according to DIN EN 12390-9 for the studied LWC was done. After 28 cycles of changing temperature from +20 to -20 °C, the maximum weathering loss of only 100 g/m² was observed as shown in Figure 4.8b. What is noteworthy here, the maximum amount of capillary suction of the tested specimen after 7 days did not exceed 1.0 %-M of the specimen. Figure 4.8a shows the development of capillary suction for five LWC specimens 7 days before the start of freeze-thaw test.
4.6 Physical Properties

According to German code DIN EN 12664, the heat transfer resistance experiment for the studied LWC was conducted in the laboratory of the Material Testing Agency (MPA) in Berlin and resulted in a value of thermal conductivity of $\lambda_{dry,10} = 0.33 \text{ W/mK}$.

![Figure 4.9: Thermal conductivity of LWC used in the study.](image)

The relationship between the thermal conductivity and the mean temperature of the samples is shown in Figure 4.9. According to DIN 4108-4, the design thermal conductivity value of 0.66 W/mK is considered for LWC with porous aggregates and with dry concrete density of 1250 kg/m³, which is double as that experimentally obtained for the new LWC used in the study.

4.7 Bond Behaviour

According to DIN 1045-1, the bond stress in the case of good bond and bar diameter $d_b < 32 \text{ mm}$ can be calculated for LWC as in the following equations:

$$f_{bd} = 2.25 \cdot \frac{f_{lctk;0.05}}{\gamma_c} \quad (4.6)$$

$$f_{lctk;0.05} = \eta_1 \cdot f_{ctk;0.05} \quad (4.7)$$

where:

- $f_{bd}$ = bond stress
- $f_{lctk;0.05}$ = characteristic strength of LWC tensile strength with 5 % Quantile
- $\eta_1$ = reduction factor according to Equation 4.3
As a comparison, the calculated design value of bond stress for LWC (LC 30/33) with steel reinforcement rods - according to previous equations - represents only 20% of the observed experimental value (Fig. 4.10) for the studied LWC.

In this chapter the mechanical and thermal properties for the new LWC were determined. The remaining important questions about the actual behaviour of the new LWC in beams and beam-column joints under static and dynamic loads using different types of reinforcement such as GFR and SRFT will be discussed in the following chapters.
CHAPTER 5

CONCEPTUAL AND STRUCTURAL DESIGN OF LIGHTWEIGHT CONCRETE BEAMS REINFORCED WITH GLASS FIBRE RODS

5.1 Introduction and Previous Work

Literature review shows that little experimental data is available for behaviour of beams made from LWC and reinforced with GFR, and even less for a structural lightweight concrete beams with dry density of 1.25 g/cm³ and concrete class LC 30/33. Thiagarajan, (2003) presented the result of an experimental and analytical comparison of a study on the flexural behaviour of concrete beams reinforced with sandblasted carbon fibre-based composite rods. He concluded that the excessive deformation for achieving the predicted moment capacity could be a limiting factor in the design of these beams. Nanni, et. al., (1995) published a state-of-the-art review of the test methods of fibre reinforced polymer (FRP) systems, which is a very comprehensive compilation. Nanni, (1993) discussed the issues involved in the flexural behaviour and design of RC members using FRP reinforcement and observed that deflection control is as important as flexural design. The advantages and disadvantages of using the working stress method and the ultimate strength method for the design of these beams are discussed, which concluded that the working stress method could be better suited in this case.

Many researchers have worked on GFRP based reinforcement rods. Sadaatmanesh, et. al., (1991) conducted experiments on concrete beams using GFRP rods and made precise predictions of maximum loads using GFRP properties. Almusallam, (1997) presented an analytical model for the prediction of the flexural strength, compared it with experimental tests on concrete beams reinforced with GFRP rods, and demonstrated a good correlation between analytical and experimental values for moment-curvature and load-deflection relationship. Satoh, et. al., (1991) conducted tests using CFRP, GFRP and steel bars. For CFRP rods, they reported a failure ratio of 0.876 for the experimental to predicted failure loads. Other related research has focussed on the effect of varying the strength of concrete, reinforcement ratio, etc. using FRP rods in concrete beams. Theriault, et. al., (1998) studied the effect of reinforcement ratio and concrete strength on the flexural behaviour of concrete beams with GFRP reinforcement. They concluded that the effect of reinforcement ratio and concrete strength on crack spacing appeared to be negligible and that a higher reinforcement ratio decreased the width and height of cracks. Faza, et. al., (1991) studied the bending response of beams with different concrete strengths using FRP rods. They have shown that the ultimate moment capacity of beams made with high strength concrete (50 MPa) increased by 90 % when FRP bars of ultimate strength of 900 MPa are used, when compared to mild steel bars. An ultimate moment capacity increase of 100 % has been noted when the concrete strength was increased from 34.5 MPa to 51.7 MPa.

The purpose of experimental and analytical study in this chapter is to investigate the behaviour of beams constructed from lightweight concrete with dry density of 1.25 g/cm³ and concrete strength of LC 30/33 (Chapter 4). The LWC beams were reinforced with GFR and SRFT (Section 3.7.2). Ten beams, including two control beams made from normal concrete C 30/37 and reinforced with steel reinforcement, were tested for flexural strength capacity, crack behaviour, deformation capacity, ductile behaviour, and bond behaviour in tension and compression zones of the beams. The details of these beams will be presented in the following section.
5.2 Materials and Dimensions

The Finite Element Programs (Cubus and Diana) were used before the experimental work to design the cross sections of the tested beams according to DIN 1045-1 and to investigate the theoretical prediction for the behaviour of the beams such as the moment curvature relation and distribution of stress and strain along the cross section of the beams in addition to determine the predicted location, where the maximum compression diagonal component in the support area lays. The details of the tested beams are shown in Figure 5.1 and Table 5.1.

Table 5.1: Details of the tested beams

<table>
<thead>
<tr>
<th>Beam Number</th>
<th>Concrete Class</th>
<th>Stirrups Type</th>
<th>Reinforcement Type</th>
<th>Top SRFT</th>
<th>Bottom SRFT</th>
<th>Additional Reinforcement</th>
</tr>
</thead>
<tbody>
<tr>
<td>B1 (LG1.0)</td>
<td>LC 30/33</td>
<td>SRFT</td>
<td>GFR</td>
<td>2 φ 8</td>
<td>4 φ 12</td>
<td></td>
</tr>
<tr>
<td>B2 (LG1.0)</td>
<td>LC 30/33</td>
<td>SRFT</td>
<td>GFR</td>
<td>2 φ 8</td>
<td>9 φ 8</td>
<td></td>
</tr>
<tr>
<td>B3 (LG0.5)</td>
<td>LC 30/33</td>
<td>SRFT</td>
<td>GFR</td>
<td>2 φ 8</td>
<td>2 φ 12</td>
<td></td>
</tr>
<tr>
<td>B4 (LG1.0)</td>
<td>LC 30/33</td>
<td>SRFT</td>
<td>GFR</td>
<td>2 φ 8</td>
<td>4 φ 12</td>
<td>15cm 1φ8 / 25cm</td>
</tr>
<tr>
<td>B5 (LG1.0)</td>
<td>LC 30/33</td>
<td>SRFT</td>
<td>GFR</td>
<td>2 φ 8</td>
<td>4 φ 12</td>
<td></td>
</tr>
<tr>
<td>B6 (LG1.0)</td>
<td>LC 30/33</td>
<td>SRFT</td>
<td>GFR</td>
<td>2 φ 8</td>
<td>4 φ 12</td>
<td></td>
</tr>
<tr>
<td>B7 (LS1.0)</td>
<td>LC 30/33</td>
<td>SRFT</td>
<td>GFR</td>
<td>2 φ 8</td>
<td>4 φ 12</td>
<td></td>
</tr>
<tr>
<td>B8 (LS0.5)</td>
<td>LC 30/33</td>
<td>SRFT</td>
<td>GFR</td>
<td>2 φ 8</td>
<td>2 φ 12</td>
<td></td>
</tr>
<tr>
<td>B9 (NS1.0)</td>
<td>C 30/37</td>
<td>SRFT</td>
<td>GFR</td>
<td>2 φ 8</td>
<td>4 φ 12</td>
<td></td>
</tr>
<tr>
<td>B10 (NS0.5)</td>
<td>C 30/37</td>
<td>SRFT</td>
<td>GFR</td>
<td>2 φ 8</td>
<td>2 φ 12</td>
<td></td>
</tr>
</tbody>
</table>

(LG1.0): L= Lightweight concrete beam, G= Glass fibre bars, 1.0= reinforcement ratio of 1.0%
(NS0.5): N= Normal concrete beam, S= Steel reinforced bars, 0.5= reinforcement ratio of 0.5%

5.3 Preparing, Loading and Monitoring

The anchorage length was calculated depending on the results obtained from pull-out tests (Section 4.7) and the anchorage length equation in DIN 1045-1. As shown in Figure 5.1, the used reinforcement bars have the same length as the beam length with a straight anchorage length of 150 mm after the support point. Four displacement gages were fixed at ends of the beam during test; in order to verify that no slip occurred between GFR or SRFT and lightweight concrete.

Figure 5.1: Schematic detail of beam layout.

Figure 5.2: Positions of displacement and strain gauges on concrete surface and reinforcement.
The beams were loaded with a displacement-controlled actuator at a constant rate of 0.02 mm/s. Loading continued after the maximum load was reached so that the post peak behavior of the beams could be observed. All tests were monitored in different ways. The deflection of the beam was measured using displacement gauges at five points under each beam. The slip between the longitudinal compression as well as tensile reinforcement and concrete was monitored. To measure the bond slip, displacement sensors were placed at the two ends of the reinforcing bars (Figure 5.2). The readings of the displacement sensors give the actual slip/relative displacement with reference to concrete. The observed slip in all tested beams was negligible for anchorage straight length of only 150 mm after the supporting point. There was no failure due to bond slip in any of the specimens. The compression and tensile strains in the longitudinal and transverse reinforcement were measured with strain gauges. In order to determine the depth of compression zone at mid span of the beam, 6 strain gauges were attached to concrete surface in compression zone every 20 mm. The diagonal compressive strains of concrete in shear area between the load point and support point were measured for the beams that have different lateral reinforcement ratio, the position of these diagonal strain gauges were previously determined using Diana-FE program and the shape of strut-and-tie models for these beams. Figure 5.2 shows the position of strain gauges on the concrete surface and the reinforcement bars.

The positions of diagonal compression trajectories according to strut-and-tie model and FEM are shown in Figure 5.3. The crack widths, heights, and numbers were observed during test for all beams.

5.4 Conventional and Modified Ductility and Deformability Indices

Ductility is a desirable structural property because it allows stress redistribution and provides warning of impending failure. In general, high ductility ratios indicate that a structural member is capable of undergoing large deflections prior to failure. Steel-reinforced concrete beams should be designed, so that failure is initiated by yielding of the steel reinforcement, followed, after considerable deformation at no substantial loss of load carrying capacity, by concrete crushing and ultimate failure. This mode of failure is ductile and is guaranteed by designing the tensile reinforcement ratio to be substantially below [ACI-318 requires at least 25 % below] the balanced ratio, which is the ratio at which steel yielding and concrete crushing occur simultaneously. Conventional definitions of ductility involve the onset of yield of steel reinforcement and, therefore, are inappropriate for evaluation of the ductility of
concrete beams reinforced with fibre reinforced polymer (FRP) because FRP has no yield point. Unless ductility requirements are satisfied, FRP cannot be used with confidence in structural members. Thus, there is a need for both quantitative and qualitative evaluations of ductility when using FRP in reinforced concrete structural members [Naaman and Jeong, 1995]. This includes computation of a suitable ductility index and comparison of behavior at ultimate between beams containing FRP reinforcement and those containing steel reinforcement.

Ductility factors have been commonly expressed in terms of the various parameters related to deformation, i.e., curvatures, rotations, and displacements [Bachmann, 1992; Park, 1992]. These conventional ductility factors can be calculated as:

**Curvature ductility** \( \mu_\phi \):

\[
\mu_\phi = \frac{\phi_u}{\phi_y}
\]

where:
\( \phi_u \) = curvature of the element at ultimate state
\( \phi_y \) = curvature of the element at yield limit

**Rotation ductility** \( \mu_\theta \):

\[
\mu_\theta = \frac{\theta_u}{\theta_y}
\]

where:
\( \theta_u \) = ultimate plastic hinge rotation
\( \theta_y \) = yield plastic hinge rotation

**Displacement ductility** \( \mu_\Delta \):

\[
\mu_\Delta = \frac{\Delta_u}{\Delta_y}
\]

where:
\( \Delta_u \) = displacement of a structural element or of a whole structure at the ultimate state
\( \Delta_y \) = displacement at yield limit

Ashour (2000) mentioned that members with a displacement ductility in the range of 3 to 5 has adequate ductility and can be considered for structural members subjected to large displacements, such as sudden forces caused by earthquake.

All these conventional methods for ductility are not suitable for beams with FRP as FRP does not have a yield point [Patrick, 2003]. Researchers have proposed some new equations to quantify the ductility of concrete beams reinforced by FRP and by steel so that a comparison may be made between them. These modified ductility or deformability factors can be expressed as following.
Naaman and Jeong (1995) proposed a ductility index $\mu_{en}$ as:

$$
\mu_{en} = 0.5 \left( \frac{E_{tot}}{E_{ela}} + 1 \right)
$$

(5.4)

where:

$E_{tot} =$ total energy under the load-deflection curve up to failure load (Fig. 5.4)

$E_{ela} =$ elastic energy computed as the area of the triangle formed at failure load by unloading the beam (Fig. 5.4)

The development of this equation was based on the assumption that the concrete beams has fully elasto-plastic behaviour and is equally applicable for beams with FRP. ACI-440 defined the ductility index as the ratio of energy absorption (area under the moment-curvature curve) at ultimate strength of the section to the energy absorption at service level [ACI-440, 2001].

Abdelrahman, et. al., (1995) presented a deformability factor $\mu$ as:

$$
\mu = \frac{\Delta_u}{\Delta_l}
$$

(5.5)

where:

$\Delta_u =$ ultimate deflection

$\Delta_l =$ equivalent deflection at uncracked section
Mufti, et. al., (1996) and Jaeger, et. al., (1997) proposed an overall performance factor $\mu_M$ as:

$$\mu_M = \left(\frac{M_u}{M_0.001}\phi_{0.001}\right)$$  \hspace{1cm} (5.6)

where:
- $M_u$ = ultimate moment
- $\phi_u$ = curvature at ultimate state
- $\phi_{0.001}$ = curvature at concrete strain of 0.001 at the outer most compression fibre (Fig. 5.5)
- $M_{0.001}$ = moment at a concrete strain of 0.001 at the outer most compression fibre (Fig. 5.5)

This model was developed for concrete beams with a rectangular cross-section and was based on a particular type of failure mode, namely, crushing of the concrete. The researchers claimed that under service load conditions, the concrete strain at the top compression fibre is about 0.001 for reinforced concrete beams.

Canadian Highway Bridge Design (CHBD) Code has included provisions for fibre-reinforced structures where FRP is used as reinforcement and the Code includes a section “Design for Deformability”. The code mentioned that for concrete beams reinforced with FRP bars or grids, the overall performance factor ($\mu_M$) must be at least 4.0 for rectangular sections and 6.0 for T–sections [Bakht, et. al., 2000].

According to CEB-FIB Model Code, (1990) sufficient ductility can be assumed to be present, if the relative depth of the neutral axis $x/d$ in the critical cross-sections in the ultimate limit state (ULS) is in accordance with the following:

- For concrete grades C 12 to C 35: $x/d \leq 0.45$ \hspace{1cm} (5.7)
- For concrete grades C 40 to C 80: $x/d \leq 0.35$ \hspace{1cm} (5.8)

However in DIN 1045-1, sufficient ductility can be presented when:

- For concrete grades C 16/20 to C 50/60: $x/d \leq 0.45$ \hspace{1cm} (5.9)
- For concrete grades > C 55/67 and lightweight concrete: $x/d \leq 0.35$ \hspace{1cm} (5.10)

where:
- $x/d$ = relative depth of the neutral axis in the critical cross-section at ultimate limit state

### 5.5 Moment-Curvature Comparison

Theoretically, the experimental curvature can be calculated from curvature of the deflection line of the beam during test. This may be accepted according to the linear elastic behaviour of the beam, but it may not accurate according to nonlinear behaviour. Therefore, the experimental curvature ($\phi_{ex}$) was accurately calculated from the following equation.

$$\phi_{ex} = \varepsilon_c / x$$ \hspace{1cm} (5.11)

where:
- $\varepsilon_c$ = concrete strain at the outer most compression fibre
- $x$ = depth of compression zone corresponding to $\varepsilon_c$
Figure 5.6: Moment-Curvature relationship for LWC and NC beams reinforced with GFR and SRFT rods.

Figure 5.6 shows the analytical (FEP - Cubus) and experimental moment-curvature curves for LWC and NC beam groups (B1, B7 and B9) and (B3, B8 and B10) with reinforcement ratios of 1.0 % and 0.5 %, respectively. In general, there is no significant difference between the analytical and experimental moment-curvature curves except in the case of B1 which is a LWC beam reinforced with GFR bars with reinforcement ratio of 1.0 %; the difference in curvature between experimental and analytical curves reaches 28.2 % at failure.

What is noteworthy here, the behaviour of LWC beams that are reinforced with SRFT bars is close to the behaviour of control beams especially at low reinforcement ratio, however using of GFR bars increases the curvature capacity of the LWC beams at the same moment level.

From Figure 5.6, it can be noted that the steel reinforced beams reach a certain moment capacity and sustain it for a long increase in curvature before failure. On the other hand, the GF reinforced beams reach its peak moment capacity just before failing. However, it should be stated that the GF reinforced beams are capable of exhibiting deformation characteristics comparable to that of steel reinforced beams before failure, the only difference being that they are unable to sustain peak capacities for long before failing. This can be attributed to the elastic ideal plastic behaviour of steel and a purely elastic behaviour of GFR rods. The high deformation of GF reinforced beams can be attributed to that the GF rods are being capable of fairly large strains due to their small modulus of elasticity before reaching their ultimate strength of 1000 MPa.
5.6  End Rotation Behaviour

The moment-end rotation curves for steel and GF reinforced LWC beams as well as for control beams (B9 & B10) are presented in Figure 5.7. As the deflection values at a distance of 500 mm from the two end supporting points were observed during the test of each beam, the end rotation could be calculated for each loading step till failure of the beams.

Figure 5.7 illustrates that the moment-end rotation curves for LWC beams that reinforced with SRFT are very close to NC beams. For steel reinforced LWC and NC beams, the shape of the moment-end rotation curve follows the general behaviour of a moment-curvature curve, in which it increases linearly until yielding of steel occurred. Once yielding occurred, there was a rapid increase in the end rotations with very little increase in the moment especially with low reinforcement ratio as in beams B8 & B10. This behaviour translates that the rotation ductility for LWC and NC beams that reinforced with SRFT are very close together (Table 5.2). It was observed that the end rotation of these beams just prior to failure varied from 2° 15' to 3° 12' depending on the reinforcement ratio.

For GF reinforced LWC beams, the moment-end rotation curves are linearly increase till failure limit. The maximum end rotation for GF reinforced beams varied from 2° 51' to 3° 31', which was relatively big than that for steel reinforced beams. In general the moment-end rotation relation is significantly affected with the elastic modulus of the reinforcement bars.

5.7  Load-Deflection Relations

Figure 5.8 shows typical load-deflection relationships for the tested beam groups (B1, B7 and B9) and (B3, B8 and B10).

Figure 5.8: Load-deflection relationship for LWC and NC beams reinforced with GFR and SRFT rods.
Although, the elastic modulus for NC is more than double as that for the studied LWC, the steel reinforced LWC beams (B7 & B8) showed similar behaviour like the NC control beams (B9 & B10), respectively. This behaviour is attributed to that the total elastic modulus for the whole cross section, which consists of concrete (LWC or NC) and steel reinforcement is approximately equal for both LWC and NC cross-sections. As expected, the steel reinforced LWC and NC beams became nonlinear after yielding of SRFT, with a large deflection increase but with little load gain. However, the GF reinforced beams behaved differently, the load continued to increase with an increase of deflection, and the load deflection relationship was almost linear till failure.

Before yielding of SRFT, the mid span deflection in GF reinforced beams was 3 times as that in steel reinforced beams. The steel reinforced beams with reinforcement ratio of 0.5 % (B8 & B10) showed more ductile behaviour than that with reinforcement ratio of 1.0 % (B7 & B9). In consequence, the maximum deflection in GF reinforced beam (B1) was 1.5 times as that in steel reinforced beams (B7 & B9), while the maximum deflection in GF reinforced beam (B3) was approximately equal to that in steel reinforced beams (B8 & B10). In other words, it can be stated that at high reinforcement ratio the difference between the maximum deflection values for GF reinforced beams and steel reinforced beams will be more than that at low reinforcement ratio.

In steel reinforced LWC and NC beams, the deflection prior to failure was 1/100 and 1/45 of the span for reinforcement ratios of 1.0 % and 0.5 %, respectively. However the deflection prior to failure in GF reinforced beams was 1/60 and 1/50 of the span for reinforcement ratios of 1.0 % and 0.5 %, respectively, which is considered to be sufficient deflection to provide warning before failure. Abdelrahman (1995) also arrived at a similar conclusion for beams with CFRP.

### 5.8 Ductility and Deformability Indices

Depending on the conventional and modified ductility and deformability indices that mentioned in Section 5.4 as well as the experimental results in Sections 5.5 to 5.7, the comparison between the ductility and deformability indices for the flexural cracked tested beams are summarized in Table 5.2.

**Table 5.2: Ductility and deformability indices for the flexural tested beams**

<table>
<thead>
<tr>
<th>Beam number</th>
<th>B1</th>
<th>B3</th>
<th>B7</th>
<th>B8</th>
<th>B9</th>
<th>B10</th>
</tr>
</thead>
<tbody>
<tr>
<td>Concrete type</td>
<td>LWC</td>
<td>LWC</td>
<td>LWC</td>
<td>LWC</td>
<td>NC</td>
<td>NC</td>
</tr>
<tr>
<td>Reinforcement type</td>
<td>GFR</td>
<td>GFR</td>
<td>SRFT</td>
<td>SRFT</td>
<td>SRFT</td>
<td>SRFT</td>
</tr>
<tr>
<td>Reinforcement ratio</td>
<td>1.0 %</td>
<td>0.5 %</td>
<td>1.0 %</td>
<td>0.5 %</td>
<td>1.0 %</td>
<td>0.5 %</td>
</tr>
<tr>
<td>Conventional</td>
<td>Displacement ductility, $\mu_{\Delta}$</td>
<td>na</td>
<td>na</td>
<td>1.58</td>
<td>3.28</td>
<td>2.19</td>
</tr>
<tr>
<td></td>
<td>Rotation ductility, $\mu_{\theta}$</td>
<td>na</td>
<td>na</td>
<td>1.44</td>
<td>2.80</td>
<td>1.91</td>
</tr>
<tr>
<td></td>
<td>Curvature ductility, $\mu_{\phi}$</td>
<td>na</td>
<td>na</td>
<td>1.65</td>
<td>3.61</td>
<td>2.45</td>
</tr>
<tr>
<td>Modified</td>
<td>Ductility index, $\mu_{\text{d}}$</td>
<td>1.25</td>
<td>1.48</td>
<td>2.61</td>
<td>4.51</td>
<td>7.85</td>
</tr>
<tr>
<td></td>
<td>Deformability factor, $\mu$</td>
<td>30.0</td>
<td>30.4</td>
<td>35.0</td>
<td>52.9</td>
<td>49.1</td>
</tr>
<tr>
<td></td>
<td>Performance factor, $\mu_{\text{f}}$</td>
<td>5.103</td>
<td>8.99</td>
<td>7.66</td>
<td>9.47</td>
<td>5.65</td>
</tr>
<tr>
<td>DIN 1045-1</td>
<td>Calculated, $x/d$</td>
<td>0.22</td>
<td>0.12</td>
<td>0.20</td>
<td>0.11</td>
<td>0.18</td>
</tr>
<tr>
<td></td>
<td>Experimental, $x/d$</td>
<td>0.21</td>
<td>0.17</td>
<td>0.27</td>
<td>0.15</td>
<td>0.23</td>
</tr>
</tbody>
</table>

na: not applicable
For the conventional ductility (Table 5.2), the ductility of steel reinforced LWC beams was less than that of steel reinforced NC beams at high reinforcement ratio, while it became close together at low reinforcement ratio especially in displacement and rotation ductility.

As a comparison between the different modified ductility and deformability indices mentioned in Table 5.2, the ductility index ($\mu_{en}$) and the deformability factor ($\mu$) show logical and acceptable trend for the ductility of all tested beams. The performance factor ($\mu_M$) plays as indicator for the ductile behaviour when it compared to the limit value that were mentioned in Canadian Highway Bridge Design (CHBD) Code, which must be at least 4.0 for rectangular sections. In spite of their ductile behaviour based on the value of their performance factor ($\mu_M$), the ductility index ($\mu_{en}$) and the deformability factor ($\mu$) for the GF reinforced LWC beams represent ductility values smaller than that for steel reinforced LWC and NC beams. In the cases of steel reinforced LWC and NC beams, the ductility index ($\mu_{en}$) and the deformability factor ($\mu$) become close together especially at low reinforcement ratio.

The calculated and experimental values of ($x/d$) according to DIN 1045-1 show that all the tested beams behave as ductile beams, although they contain brittle materials such as GFR and concrete. This may be due to the high deformation capacity of these beams before their failure, which present enough warranty before failure.

### 5.9 Crack Behaviour

Table 5.3 gives a summary of the observed crack pattern in the beams. Figure 5.9 shows the crack patterns just before failure state in beams reinforced with GFR and SRFT bars at the same reinforcement ratio.

**Table 5.3: Observed crack pattern in beams**

<table>
<thead>
<tr>
<th>Beam Number</th>
<th>Sequence of failure</th>
<th>Distance between cracks, mm</th>
<th>Number of cracks</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Central zone</td>
<td>Outer zone</td>
</tr>
<tr>
<td>B1</td>
<td>Concrete crushing – no yielding</td>
<td>35</td>
<td>28</td>
</tr>
<tr>
<td>B2</td>
<td>Concrete crushing – no yielding</td>
<td>30</td>
<td>36</td>
</tr>
<tr>
<td>B3</td>
<td>Concrete crushing – no yielding</td>
<td>30</td>
<td>40</td>
</tr>
<tr>
<td>B4</td>
<td>Concrete crushing – no yielding</td>
<td>35</td>
<td>33</td>
</tr>
<tr>
<td>B5</td>
<td>Shear failure</td>
<td>80</td>
<td>14</td>
</tr>
<tr>
<td>B6</td>
<td>Concrete crushing – no yielding</td>
<td>40</td>
<td>33</td>
</tr>
<tr>
<td>B7</td>
<td>Steel yielding – Concrete crushing</td>
<td>60</td>
<td>21</td>
</tr>
<tr>
<td>B8</td>
<td>Steel yielding – Concrete crushing</td>
<td>65</td>
<td>17</td>
</tr>
<tr>
<td>B9</td>
<td>Steel yielding – Concrete crushing</td>
<td>90</td>
<td>12</td>
</tr>
<tr>
<td>B10</td>
<td>Steel yielding – Concrete crushing</td>
<td>110</td>
<td>10</td>
</tr>
</tbody>
</table>

a. Crack pattern for B3 with GFR rods.  
b. Crack pattern for B10 with SRFT rods.

*Figure 5.9: Crack pattern for GFR and SRFT reinforced beams.*
All beams, except B5, showed bending failure which was nicely indicated by numerous vertical cracks in the central zone of the beam. The number of cracks in the central zone for the GF reinforced beams tends to increase with smaller GF-bar diameters as well as with smaller longitudinal and transverse reinforcement ratios. The comparison between the two rod types (GFR & SRFT) indicates that the greater the elastic modulus of reinforcement bars, the smaller the number of cracks and the smaller the crack width (Fig. 5.9). Another remark is that in steel reinforced NC and LWC beams, the cracks progressed upward with gradual increase in loads. But for GF reinforced beams, the cracks quickly developed to a height of three-quarters the beam depth. This enables the GFR rods to have high strains to develop the required stresses to sustain force equilibrium and also explains the effect of low modulus of elasticity for GF rods.

Figure 5.10: Crack width – Moment relation for LWC and NC beams reinforced with GFR and SRFT rods.

Figure 5.10 shows a comparison of the measured experimental crack widths at various load levels. For beams B1 & B2, it can be stated that using small GF-bar diameters with the same reinforcement ratio has no significant effects on the crack width, while it increases the number of cracks in the flexural region. The GFR rods increase the crack width with about 50% more than that in the case of SRFT rods. However, the residual crack width after unloading in the case of GFR rods is considered negligible compared to that in the case of SRFT rods. This explains the linear-elastic behaviour of GF rods. What is noteworthy here, the effect of reinforcement types (GFR & SRFT) on the crack width is significant, while the effect of concrete types (NC & LWC) is not significant.

The design crack width can be calculated according to DIN 1045-1 as shown in the following equations.

\[
w_k = s_{r, \text{max}} \cdot (\varepsilon_{sm} - \varepsilon_{cm})
\]

(5.12)

where:
\( w_k \) = crack width
\( s_{r, \text{max}} \) = maximum distance between cracks
\( \varepsilon_{sm} \) = tensile strain of reinforcement bar
\( \varepsilon_{cm} \) = concrete strain between cracks

The term \((\varepsilon_{sm} - \varepsilon_{cm})\) can be calculated as:
where:
\( \sigma_s \) = actual stress in the reinforcement bar
\( f_{ct,\text{eff}} \) = effective concrete tensile strength
\( \rho_{\text{eff}} \) = effective reinforcement ratio
\( E_s \) = elastic modulus of the reinforcement bar
\( \alpha_e \) = ratio between \( E_s \) and the concrete elastic modulus

According to Equations (5.12) and (5.13), the calculated design crack widths for the beams B3, B8, and B10 are quite similar but slightly higher than the experimental values.

### 5.10 Concrete and Reinforcement Strains Comparison

Figure 5.11 depicts the maximum concrete compression strain along with the corresponding tensile strains in reinforcement rods for beam groups (B1, B7 and B9) and (B3, B8 and B10).

A linear relation was observed between the concrete maximum compression strain and GFR rods tensile strain for LWC beams. The reinforcement ratio had no significant effect on the slope of this linear relation. In steel reinforced LWC and NC beams, a difference in strain behaviour was detected in the lower strain regions especially with high reinforcement ratio. This difference may refer to that the bond stress between the studied LWC and SRFT rods is more than that in the case of NC (Chapter 4, Section 4.7).

The maximum concrete compression strains in steel reinforced NC beams ranged from 0.0028 to 0.0032, which are not in tune with the fact that it normally reaches 0.0035. In steel reinforced LWC beams, the maximum concrete compression strains ranged from 0.0027 to 0.003.
The relation between the compression strain of reinforcement rods in compression zone and the strain of concrete layer at the same level is investigated for the new LWC beams. This relation is presented in Figure 5.12. An approximate linear relation was observed for the all tested beams.

In the case of steel reinforced NC beams, the difference between the steel compression strain and concrete strain was negligible. The LWC layer at the level of compression reinforcement strains more than compression reinforcement. This difference translates to slip between concrete and compression reinforcement in the maximum compression zone at mid span of the beam prior to failure. The displacement sensors that attached to the two ends of the compression reinforcement bars had not detected any relative deformations between the bars and surrounding concrete, i.e. the relative deformation between concrete and compression reinforcement at mid span becomes smaller to be negligible at the end of the beam.

As depicted in this chapter, the steel reinforced LWC beams showed the same behaviour regarding bending capacity, deformation behaviour, and crack behaviour like NC beams. This proves that this new material with very high strength to weight ratio (Table 2.1) behaves and can be treated just like normal lightweight concrete. In spite of the high deformation capacity of glass-fibre reinforced LWC beams, they presented negligible residual deformations after unloading. This behaviour of GF reinforced LWC beams could be one option to reduce damage and post repair works for beam-column joints after earthquake excitations. For this reason and to achieve the seismic design concept of strong-columns weak-beams, the dynamic behaviour of the joint between LWC beams and NC columns was studied. The results of this study for interior and exterior beam-column joints will be presented in the following chapter.
CHAPTER 6

SEISMIC BEHAVIOUR OF LIGHTWEIGHT CONCRETE BEAM-NORMAL CONCRETE COLUMN JOINTS

6.1 Introduction and Previous Work

Regarding technical-construction aspects, the beam-column joints are the most critical area in tall buildings especially in seismic regions. One option to achieve the desired seismic design concept of “strong-columns weak-beams” is to construct the beam-column joints from a LWC beam that is reinforced with steel rods or glass fibre rods and normal concrete (NC) columns with steel reinforcement rods. According to Schlaich J., et. al., (2001) the design of beam column joints should be considered as geometric discontinuity regions. The design of these discontinuity regions according to conventional cross section design aspects is not accurate. The conventional methods define the requirements in terms of reinforcement, without carrying out a check of the diagonal compression strength of the concrete in the joint intersection area (Fig. 6.13). Even the strut-and-tie models only work well if beams and columns are made of the same material.

As seismic design of structures moves towards performance based design, there is a need for new structural members and systems that possess enhanced deformation capacity and damage tolerance, while requiring simple reinforcement details. The development of a highly damage-tolerant beam-column connection would allow structural engineers to design joints for moderate shear distortions while exhibiting little damage, reducing rotation demands in plastic hinges of the beam, and eliminating the need for post-earthquake joint repairs. One option for achieving this goal is to use a LWC beam, which is reinforced with glass-fibre reinforcement bars with superior deformation capacity in beam-column connections. As is mentioned in the previous chapter, in spite of the high deformation capacity of glass-fibre reinforced LWC beams during tests, they presented negligible residual deformations after unloading. This behaviour of GF reinforced LWC beams could be one option to reduce damage and post repair works for beam-column joints after earthquake excitations. For this reason, the dynamic behaviour of the joints between LWC beams and NC columns was studied and will be presented in this chapter.

Bertero, et. al., (1980) describe an experimental investigation into the behaviour of interior beam-column joints of a ductile moment-resisting frame constructed of lightweight aggregate concrete. Emphasis is placed on the effects of bond deterioration in the joint region. Results of experiments carried out on two lightweight reinforced concrete specimens are compared with similar experiments on specimens constructed of normal weight concrete. Their study concluded that, the behaviour of the lightweight specimens under cyclic loading was drastically different from that observed under monotonic loading due to earlier slippage of the beam reinforcement through the joint. Yielding of this reinforcement accelerates bond deterioration and, therefore, slippage. In addition, the energy dissipated by the lightweight specimen was considerably smaller than that of similar normal weight concrete specimen.

Beam-column connections in reinforced concrete frame structures under earthquake-induced lateral displacements are generally subjected to large shear stresses that may lead to significant joint damage and loss of stiffness in the structure [Gustavo, et. al., 2005]. Since the 1960s, several researchers have devoted significant effort studying the behaviour of joints
under shear reversals, as well as on the development of design recommendations for ensuring adequate connection behaviour in frame structures expected to undergo large inelastic deformations. Current design recommendations for RC beam-column joints in earthquake resistant construction given by Joint ACI-ASCE Committee 352 (2002), focus on three main aspects: 1) confinement requirements; 2) evaluation of shear strength; and 3) anchorage of beam and column bars passing through the connection. Using these recommendations in addition to ensure the strong column-weak beam mechanism enable the joint to behave satisfactorily during earthquakes. The minimum amount and maximum spacing of joint transverse reinforcement are based on the requirements for critical regions of RC columns, which when combined with the longitudinal reinforcement from beams and columns, often lead to severe reinforcement congestion and construction difficulties. Further, the need to satisfy the anchorage length requirements for beam and column longitudinal bars may require either the use of large column and/or beam sections or a large number of small diameter bars, which might in turn increase reinforcement congestion in the connection. It is worth mentioning that satisfying the minimum ACI Code provisions does not prevent the formation of wide diagonal cracks in connections during large displacement reversals [Joint ACI-ASCE Committee 352, 2002] and thus, these provisions are primarily intended to provide protection against loss of lives and structural collapse.

As investigated by Chalioris, et. al., (2008) the behaviour of beam-column joints has long been recognized as a significant factor that frequently becomes critical for the overall behaviour of reinforced concrete framed structures subjected to seismic excitations. Inadequate shear reinforcement in joint regions of existing RC structures proved to be the cause of brittle failure and catastrophic collapse during earthquake events [Pagni, et. al., 2006 and Pantelides, et. al., 2002]. Experimental research is the main tool for the investigation of the parameters that influence and improve the joint performance [Karayannis, et. al., 2006]. It is also noted that detailed design recommendations for RC beam-column joints were first published in the last two or three decades in the U.S.A. and Europe. Attempts for the improvement of the seismic properties of these members are usually focused on the use of non-conventional reinforcement, such as steel fibres [Shannag, et. al., 2005], composite materials (FRPs) [Karayannis, et. al., 2002 and Ghobarah, et. al., 2001], spiral reinforcement [Karayannis, et. al., 2005] and crossed inclined bars [Paulay and Park, 1984]. Further experimental studies indicated that exterior beam–column joints with common stirrups and inclined bars as supplementary shear reinforcement in joint area exhibited significantly improved behaviour with respect to joints with only conventional reinforcement, such as stirrups and vertical bars [Tsionos, et. al., 1992]. The advantages of the crossed inclined bars as a feasible solution for joint reinforcement have also been emerged in the theoretical study of Bakir, (2003) although in this study the increase in the joint shear strength due to X-bars proved to be dependent on the joint aspect ratio due to geometrical constrains.

6.2 Classification of Beam-Column Joints

![Image of beam-column joints]

Figure 6.1: Types of beam-column joints.
In a moment resisting frame, three types of joints can be identified; interior joint, exterior joint, and corner joint (Figure 6.1). The severity of forces and demands on the performance of these joints calls for greater understanding of their seismic behaviour. These forces develop complex mechanisms involving bond and shear within the joint. The joints should have adequate strength and stiffness to resist the internal forces induced by the framing members and to enable the adjoining members to develop and sustain their ultimate capacity.

6.3 Forces Acting on the Beam-Column Joints

The pattern of forces acting on a joint depends on the configuration of the joint and the type of loads acting on it. The forces on an interior joint subjected to gravity loading can be depicted as shown in Fig. 6.2a.

![Diagram showing forces on interior joints](image)

a. Gravity loads  
b. Seismic loads

*Figure 6.2: Forces acting on interior joints [Macgregor, J.G., 1988].*

In the case of lateral (or seismic) loading, the equilibrating forces from beams and columns, as shown in Fig. 6.2b develop diagonal tensile and compressive stresses within the joint. Cracks develop perpendicular to the diagonal tension A-B plane and at the faces of the joint. The compression struts are shown by dashed lines and tension ties are shown by solid lines. The transverse reinforcements are provided in such a way that they cross the plane of failure to resist the diagonal tensile forces.

![Diagram showing forces on exterior joints](image)

a. Loads  
b. Poor details  
c. Satisfactory details

*Figure 6.3: Forces acting on exterior joints [Uma, et. al., 2004].*
The forces acting on an exterior joint can be idealized as shown in Fig. 6.3a. The shear force in the joint gives rise to diagonal cracks thus requiring reinforcement of the joint. The detailing patterns of longitudinal reinforcements significantly affect joint efficiency. Some of the detailing patterns for exterior joints are shown in Fig. 6.3b and Fig. 6.3c.

### 6.4 Shear Requirements

The external forces acting on the face of the joint develop high shear stresses within the joint. The shear stresses give rise to diagonal stresses causing diagonal cracks. Extensive cracking occur within the joint under load reversals, affecting its strength and stiffness and hence the joint becomes flexible enough to undergo substantial shear deformation. Before discussing the shear behaviour, it is imperative to arrive at the shear force demand on joints. The determination of shear force in the vertical and horizontal direction is usually essential. Since well established code procedures aim at the beam hinging mechanism, it is generally sufficient to discuss the shear force demand in the horizontal direction only. The Strut-and-tie model method is used in this study to calculate both of horizontal and vertical components of the diagonal compression force in the joint.

### 6.5 Studied Parameters for Interior and Exterior Joints

In reinforced concrete members, the inelastic rotations spread over definite regions are called plastic hinges. The plastic hinges are “expected” locations where the structural damage can be allowed to occur due to inelastic actions involving large deformations. Hence, in seismic design, the damages in the form of plastic hinges are accepted to be formed in beams rather than in columns. Mechanism with beam yielding is characteristic of strong-column-weak beam behaviour in which the imposed inelastic rotational demands can be achieved reasonably well through proper detailing practice in beams. Therefore, in this mode of behaviour, it is possible for the structure to attain the desired inelastic response and ductility.

In the beam-column joint, the force in a bar passes continuously through the joint changes from compression to tension during seismic excitation. This causes a push-pull effect which imposes severe demand on bond strength and necessitates adequate development length within the joint. For this reason two different development lengths (15 and 25 times bar diameter) for the beam bars passing through the joint in interior and exterior joints were tested. In addition, the effect of the end bar conditions in exterior joint is taken into account in the study.

One of the basic design demands is that the column above and below the joint intersection area should have sufficient flexural strength when the adjoining beams develop flexural over strength at their plastic hinges. This column to beam flexural strength ratio is an important parameter to ensure that possible hinging occurs in beams rather than in columns. For this purpose two different ratios of beam reinforcement are used in the study as will as two different types of reinforcement - steel and glass-fibre reinforcement bars – are used in the study (Table 6.1).

In order to reduce the effect of geometric nonlinearity ($P\Delta$-effect) in the design of tall buildings in seismic regions, most of designers try to increase the stiffness of vertical members to reduce $\Delta$-term, while there is another important option to reduce $P\Delta$-effect by reducing the own-weight of the floor slabs using LWC, which reduces axial loads on the column ($P$-term). Therefore, the effect of column axial load on the overall behaviour of LWC beam – NC column joints is investigated in this study.
Another important demand for the design of beam-column joints in seismic regions is its shear strength capacity and the joint ability to dissipate energy with small shear deformations. Therefore, the effect of confinement ratio in the joint intersection area is aimed in this study.

### 6.6 Preparing, Loading and Monitoring

A Finite Element Program (FEP) Diana was used before the experimental work to design the cross sections of the tested beam-column joints according to DIN 1045-1, taking into account the current design recommendations for RC beam-column joints in earthquake resistant construction given by Joint ACI-ASCE Committee 352 (2002). In addition, Diana was used to investigate the theoretical prediction for the behaviour of the beam-column joints such as the lateral load-story drift diagram, position of first yield, failure mode of the tested joints in addition to determine the predicted place, where the plastic hinge in the specimen occurs. The details of the tested beam-column joints are shown in Figure 6.4 and Table 6.1.

![Figure 6.4: Schematic detail and test setup for interior and exterior beam-column joints.](image)

a. Interior beam-column joint.  
b. Exterior beam-column joint.  
c. Interior beam-column joint – test setup.  
d. Exterior beam-column joint – test setup.
Table 6.1: Details of the ten interior and ten exterior beam-column joints

<table>
<thead>
<tr>
<th>Section</th>
<th>Category</th>
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<th>S2</th>
<th>S3</th>
<th>S4</th>
<th>S5</th>
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<td>Middle RFT I/E</td>
<td>φ 8/10cm</td>
<td>φ 8/10cm</td>
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<td>Interior J.</td>
<td>200x400</td>
<td>200x400</td>
<td>200x400</td>
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All dimensions in mm.

NC = normal concrete, LWC = lightweight concrete, RFT = Reinforcement, Typ. = Type,
I/E = internal joint / external joint.

S1: The beam longitudinal bars passing through the joint has a development length of 15 times bar diameter.
The column in all specimens is loaded with 25 % P<sub>c</sub>, except in specimens S8, S9, and S10 the column is loaded with 50 % P<sub>c</sub>, where P<sub>c</sub>: Column axial design load according to DIN 1045-1.
The strain gauges were used to measure the strains in the top and bottom reinforcement bars of the beam, side and middle bars in the column, and transverse reinforcement in the joint intersection area. The positions of strain gauges are shown in Fig. 6.5 for interior and exterior joints. Eight displacement gauges were located at the four faces of the interior joint intersection zone to measure the rotation angle for both column and beam during the test as shown in Figure 6.4c, while four displacement gauges were used for the exterior joints as shown in Fig. 6.4d.

The test starts by loading the column with the axial compression load (Table 6.1) using four 30 ton capacity hydraulic cylinders that are fixed to the base of the roller support under the column. The value of axial load on the column is measured by two 100 ton load cells that are fixed between the top of the column and the hinge support.

After applying the axial load to the column the lower end of the column is fixed to the arm of 100 ton dynamic hydraulic cylinder, which applies the horizontal hysteretic loading on the specimen with frequency of 1 cycle per 50 seconds as shown in Fig. 6.6. The two ends of the beam are fixed to link elements that supplied with load cells to measure the reaction force at the end of the beams during the test. The test set up details is shown in Fig. 6.4.
6.7 Experiments

The following figure shows the details of terms used in the following sections:

As shown in Figure 6.7a, the story drift ratio is calculated as the ratio between the lateral story displacement and the height of the column. The horizontal part of the envelope of hysteresis response represents the plastic drift capacity or the ductility behaviour of the specimen, while the descend part shows the strength degradation of the element (Fig. 6.7b). In all following diagrams the horizontal axis represents the story drift and the vertical axis represent the beam, column, and joint internal forces. As shown in Fig. 6.7c, at the points 3, 6, 9, and 12 of time-story drift diagram, the moment of the column at the face of intersection area balances with the additional moment due to the eccentricity of the column vertical axis. This behaviour makes these points away from the origin point in the story drift-column moment diagram for every cycle of loading.

6.7.1 Interior beam-column joints

6.7.1.1 Influence of column dimensions (development length)

The development length for the beam longitudinal reinforcement bars passing through the joint has to satisfy the requirements for compression and for tension forces in the same bar. Two specimens S1 and S2 were tested with development lengths (Fig. 6.2) of 15 and 25 times the bar diameter respectively.
A plastic tube was used to reduce the contact length for the bar through the joint. The story drift – column and beam moment diagrams for S1 and S2 are shown in Fig. 6.8. The specimen S1 shows high degradation in beam and column strengths after 1.8 % story drift. Insufficient development length and the spread of splitting cracks into the joint core may result in slippage of bars in the joint (Fig. 6.9). In the case of interior joints, the column depth is the available development length for the straight longitudinal bars passing through the joint. Research has shown that when the development length is greater than 25 bar diameters little or no bond degradation was observed with respect to various shear stress levels in the joint. In other words, to avoid bond deterioration, the column depth should be around 25 times the diameter of the bar.
6.7.1.2 Strut-and-tie model and influence of beam reinforcement

In order to study the influence of beam reinforcement type and ratio on the overall behaviour of LWC beam and NC column joint, two types of reinforcement bars with two different ratios were used. The specimens S2 and S3 were reinforced with steel reinforcement bars with 1.75 % and 3.50 % reinforcement ratios respectively. While the specimens S4 and S5 were reinforced with glass-fibre reinforcement bars with 1.75 % and 3.50 % reinforcement ratios respectively.

![a. Story drift – beam moment diagram for S2 (left) and S3 (right)](image)

![b. Story drift – beam moment diagram for S4 (left) and S5 (right)](image)

Figure 6.10: Story drift – beam moment diagram for specimens S2, S3, S4 and S5.

In general, the low reinforcement ratio enhances the overall behaviour of the joint by reducing the flexural strength of the beam (Fig. 6.10) comparable to that of the column and then minimizes the cracks in the column and joint intersection area to a very low limit especially in the case of steel reinforcement bars as shown in Fig. 6.11.

![a. Crack pattern for S2 (left) and S3 (right)](image)
b. Crack pattern for S4 (left) and S5 (right)

Figure 6.11: Crack pattern for specimens S2, S3, S4 and S5.

![Figure 6.11: Crack pattern for specimens S2, S3, S4 and S5.]

-100 0 100 200 300
Story Drift [%]

-5 -4 -3 -2 -1 0 1 2 3 4 5
Joint Shear [KN]

1.75 % - SRFT

-100 0 100 200 300 400 500 600 700 800 900
Story Drift [%]

-5 -4 -3 -2 -1 0 1 2 3 4 5
Joint Shear [KN]

3.50 % - SRFT

Figure 6.12: Story drift – joint shear diagram for specimens S2 (left) and S3 (right).

The story drift – joint shear diagrams for the specimens S2 and S3 are shown in Figure 6.12. The joint shear force is reduced in the joint intersection area due to reduction of the longitudinal reinforcement ratio in the beams which coincide with that proved by strut-and-tie model.

![Figure 6.13: Strut-and-tie model for interior joint.]

Figure 6.13: Strut-and-tie model for interior joint.

Figure 6.13 shows the strut-and-tie model for the studied interior joint, $V_c$ is the horizontal force that is applied to the specimens by horizontal hydraulic jack and represent the story shear force. $V_b$ represents the reaction of the two link elements at the end of the beam arms. $T_c$ and $T_b$ represent the tensile forces in the longitudinal reinforcement bars in column and beam, respectively. $C_c$ and $C_s$ represent the compression forces that are carried with concrete and
reinforcement bars, respectively. $C_c$ and $C_b$ represent the total compression forces in column and beam, respectively. It should be noted that the direction of $V_c$ and $V_b$ will be reversibly changed during the test.

Figure 6.14: Principle stress for the interior joint using finite element program Diana.

The strut-and-tie model shows diagonal compression trajectories in the joint intersection area. The diagonal compression force $C_d$ and the diagonal stresses are verified by FEM - Diana (Fig. 6.14).

Figure 6.15: Strut-and-tie models for an interior joint with different details [Schlaich J., et. al., 2001].

The different structural details for an interior joint and its corresponding strut-and-tie model according to Schlaich J., et. al., (2001) are shown in Figure 6.15. It is suitable to use reinforcement details that is shown in Fig. 6.15b which simulate the details for frame corner, however using of diagonal reinforcement bars in the joint as shown in Fig. 6.15a is not practical. The details in Fig. 6.15c that is used in our study is more suitable for hysteretic loading of the joint but the concentration of compression stress at the corner $A$ should be taken into account.

According to the rules for design of structural elements by strut-and-tie model, the following equations are used to calculate the joint shear force ($T_h$). This shear force is the horizontal component of the diagonal compression force in the joint intersection area (Fig. 6.13). The middle longitudinal bars in the column are very essential to carry the vertical component ($T_v$) of the diagonal compression force in the joint intersection area.
\[
T_h = T_b + C^- - V_c \quad \text{(6.1)}
\]
\[
T_v = T_c + C^- - V_b \quad \text{(When \(N\) is neglected)} \quad \text{(6.2)}
\]

where:

\(T_h\) = joint shear force carried by transverse stirrups in the joint intersection area

\(T_v\) = vertical component of the diagonal compression force in the joint intersection area which is carried with the middle vertical bars in the column

\(V_c\) = the horizontal force that is applied to the specimens by horizontal hydraulic jack and represents the story shear force

\(V_b\) = reaction of the two link elements at the end of the beam arms

\(T_c\) and \(T_b\) = tensile forces in the longitudinal reinforcement bars in column and beam respectively

\(C_c^-\) and \(C_b^-\) = the total compression forces in column and beam respectively

As recommended in strut-and-tie model method; it is important to design the stress concentration at corner \(A\) (Fig. 6.13 and Fig. 6.15c). The corner \(A\) is considered as three compression components joint (CCC-Joint). The effective design strength \(f_{cd,eff}\) for CCC-joint can be calculated as

\[
f_{cd,eff} = \alpha \cdot f_{cd1} \quad \text{(6.3)}
\]

where:

\(f_{cd,eff}\) = The effective design compression strength

\(\alpha\) = reduction factor and equals 1.0 for CCC-joint

\[
f_{cd1} = \alpha \cdot \frac{f_{ck}}{\gamma_c} \quad \text{(6.4)}
\]

where:

\(f_{cd1}\) = design compression strength

\(\alpha\) = reduction factor for long-term effect on concrete strength and equals 0.85 for normal concrete and 0.80 for lightweight concrete according to DIN 1045-1

\(\gamma_c\) = concrete strength safety factor and equals 1.5

According to the obtained results from the test of interior joints, the effective design strength \(f_{cd,eff}\) was not reached. Therefore, it is recommended to reduce \(\alpha\) in Equation (6.3) to be 0.85 for two reasons; the first is, the three compression components \(C_c, C_b^-\) and \(C_c^-\) in Fig. 6.13 act on two different materials which are normal and lightweight concrete with equal compression strength but with different E-modulus, the second reason is the type of loads which is hysteretic load as in this study, this means that the concrete at this corner \(A\) is cracked while it reaches its tensile strength at early cycles of loading.
6.7.1.3 Influence of column axial compression load

Figure 6.16 shows the story drift – column moment diagram for specimens S3, S5, S8 and S9.

![Diagram](image1)

a. Story drift – column moment diagram for S3 (left) and S8 (right)

![Diagram](image2)

b. Story drift – column moment diagram for S5 (left) and S9 (right)

Figure 6.16: Story drift – column moment diagram for specimens S3, S5, S8 and S9.

Figure 6.16 illustrates that at the same story drifts, the moment on the column section at the face of intersection zone in the case of 25 % \( P_o \) is smaller than that in the case of 50 % \( P_o \). This refers to the additional moment due to eccentricity in the vertical axis of the column during the test especially in the case of 50 % \( P_o \). This eccentricity is proportional to the story drift and in consequence the additional moment increase with increasing of the story drift which led to increase the slope of loading curve in the case of 50 % \( P_o \) than that in the case of 25 % \( P_o \). In other words, the flexural strength of the column increases by increasing the column axial load especially at the early cycles of loading. However, the column strength degradation will be also higher as the column load increases especially at the later cycles of loading.

![Diagram](image3)

a. Story drift – beam moment diagram for S3 (left) and S8 (right)
As shown in Fig. 6.17, the ductile behaviour of the LWC beams in the case of 25 % $P_o$ (S3 & S5) is better than that in the case of 50 % $P_o$ (S8 & S9). The peak moment values in the case of S3 and S5 happened at story drift values more than that in the case of S8 and S9 respectively. The flexural strength of the beam was higher in the case of 25 % $P_o$ than that in the case of 50 % $P_o$. The additional moment on the column at the face of intersection zone - due to the column axis eccentricity during the test - balances with an additional moment on the beam section at the face of joint connection that accelerates the peak moment of the beam to happen at small values of story drift.

Column and beam rotation angles - story drift diagrams for specimens S3, S5, S8 and S9 are shown in Fig. 6.18. The column rotation angles in the case of 50 % $P_o$ (S8 & S9) were more than that in the case of 25 % $P_o$ (S3 & S5), while the beam rotation angles were vice versa at the same story drift. This explains that the reduction of vertical loads on the column by using LWC will reduce the rotation angle in columns and increase it in beams i.e. the most amount of kinetic energy during earthquake excitations will be dissipated by the beams rather than by the columns which achieves the principle of strong columns - weak beams.

6.7.1.4 Influence of using lightweight concrete beams

Figure 6.19 shows the story drift against the moment on column and beam for specimens S8 with LWC beams and S10 with NC beam.
The beam moment in specimen S10 reaches its maximum peak at story drift of 2.7 % and the column strength degradation for the same specimen starts at story drift of 2.0 %, while the column strength degradation in specimen S8 doesn’t happen till 3.0 % story drift and the beam strength degradation start after 3.0 % story drift. This means that the LWC beams absorbs the most of energy and dissipates it in the form of inelastic deformations while the SRFT of the column still in elastic regions. In the case of NC beams and after a small value of story drift the column and beam shear both to dissipate this energy.

Figure 6.20: Story drift – column rotation angle diagram for specimens S8, S9 and S10.

The column rotation angles for specimens S8 and S10 at different values of story drift are given in Fig. 6.20. The figure shows that the column rotation angle reaches 6° at story drift of 3.5 %, while it reaches 4° at the same story drift in the case of specimen S8, this difference is absorbed by the deformations in the LWC beam in the case of specimen S8. For specimen S9, the column rotation angles shows also low values till 4.0 % story drift comparable to specimen S10, which may refers to the high elastic deformation in the beam of specimen S9 due to use of GFR bars.
6.7.1.5 Influence of horizontal links in the connection area

Kim, et. al., (2008) presented the influence of non-geometric parameters on joint shear stress vs. joint shear strain for normal reinforced concrete joints.

Fig. 6.21 concluded that an increase of concrete compressive strength or beam reinforcement results in an increase of both shear stress and shear strain simultaneously; an increase of joint transverse reinforcement causes an increase of joint shear stiffness.

To study the effect of transverse reinforcement in the studied joints, the required confinement stirrups to resist the shear force in the joint are calculated according to strut-and-tie model method as presented in Equation (6.1). The required stirrups are used to confine the joint area for specimens S3 and S5 with confinement ratio of 4.54 %, while only one stirrup with confinement ratio of 0.65 % is used to measure the stirrups strain for specimens S6 and S7.

a. Story drift – column moment diagram for S3 (left) and S6 (right)

b. Story drift – beam moment diagram for S3 (left) and S6 (right)
The high confinement ratio in the joint area enhances the shear stiffness in the joint and in consequence improves the strength degradation for both column and beam as shown in Fig. 6.22. For example in Fig. 6.22b the peak of moment for the beam of specimen S3 reaches 100 KN.m at story drift of -3.2 % and the degradation of moment reaches 70 KN.m at story drift of -4 %. However the peak moment for the beam of specimen S6 reaches 80 KN.m at -3.0 % story drift and the degradation of moment reaches 40 KN.m at -4.0 % story drift.

Fig. 6.23 shows the relation between story drift and stirrups strain for specimens S3, S5, S6 and S7. As expected, at low confinement ratio, the stirrups in joint intersection area are yielding at 2.0 % and 3.0 % story drifts for specimens S6 and S7 respectively, while the beam did not reach its maximum flexural capacity yet. On the other hand at high confinement ratio in specimens S3 and S5, the beam reached its maximum flexural capacity, while the stirrups in the joint still in elastic strain domain (Fig. 6.24).
6.7.2 Exterior beam-column joints

6.7.2.1 Influence of column dimensions and anchorage conditions

In exterior joints the beam longitudinal reinforcement that frames into the column terminates within the joint core. If the anchorage length of the bars is not sufficient, the bond deterioration initiates at the column face due to yield penetration and splitting cracks, progresses towards the joint core. Repeated loading will aggravate the situation and a complete loss of bond up to the beginning of the bent portion of the bar may take place. The longitudinal reinforcement bar, if terminating straight, will get pulled out due to progressive loss of bond. The pull out failure of the longitudinal bars of the beam results in complete loss of flexural strength. This kind of failure is unacceptable at any stage. Hooks for steel bars or end heads for glass-fibre bars (Fig. 6.25) are helpful in providing adequate anchorage when furnished with sufficient horizontal development length and a tail extension.

![Figure 6.25: End condition for the longitudinal bars of the beam in exterior joints.](image)

**Figure 6.25: End condition for the longitudinal bars of the beam in exterior joints.**

- **a. Story drift – column moment diagram for S1 (left) and S2 (right)**
- **b. Story drift – beam moment diagram for S1 (left) and S2 (right)**

*Figure 6.26: Story drift – column and beam moment diagrams for specimens S1 and S2.*
The specimens S1 and S2 were tested with horizontal development lengths of 10 and 20 times the bar diameter, respectively. The story drift – column and beam moment diagrams are shown in Fig. 6.26. The specimen S1 shows high degradation in beam strength after 1.5 % story drift, where the column flexural strength starts to increase. While for the specimen S2, the flexural strength of the beam stays constant after it reaches its peak up to 3.8 % story drift.

![Figure 6.27: Crack pattern for specimens S1 (left) and S2 (right).](image1)

When the reinforcement bar is subjected to compression, the tail end of hooks or the end heads of GF bars is not generally helpful to cater to the requirements of development length in compression. However, the horizontal ties in the form of transverse reinforcement in the joint provide effective restraints against the hooks when the beam bar is subjected to compression force (Fig. 6.27). In other words, to avoid bond deterioration, the column depth should be around 25 times bar diameter.

### 6.7.2.2 Strut-and-tie model and influence of beam reinforcement

Increasing the column flexural strength to beam flexural strength ratio minimizes the cracks in the column as well as in the joint intersection area to a very low limit especially in the case of steel reinforcement bars as shown in Figure 6.28 for specimens S2 and S3.

![Figure 6.28: Crack pattern for specimens S2 (left) and S3 (right).](image2)
The low reinforcement ratio enhances the overall behaviour of the joint by reducing the flexural strength of the beam as shown in Fig. 6.29. For example in specimens S2 with low reinforcement ratio and S3 with high reinforcement ratio, the peak moment values reach 50 and 80 KN.m respectively. Because of the high reinforcement ratio for the beam in S3, the cracks start in the joint intersection area before the beam reaches its yield.

The story drift – joint shear diagrams for the specimens S2 and S3 are shown in Figure 6.30. The joint shear force is reduced in the joint intersection area due to the reduction of the reinforcement ratio in the beam which coincides with that proved by strut-and-tie model as shown in Fig. 6.31.
Fig. 6.31b and Fig. 6.31c show the effect of the beam bars end conditions. Where, $V_c$ is the horizontal force that applied to the specimens by horizontal hydraulic jack and represent the story shear force. $V_b$ represents the reaction of the two link elements at the end of the beam arms. $T_b$ represents the tensile force in the longitudinal reinforcement bars in beam. $C_c$ and $C_s$ represent the compression forces that carried with concrete and reinforcement bars respectively. $C_c$ and $C_b$ represent the total compression forces in column and beam respectively. It should be noted that the direction of $V_c$ and $V_b$ will be changed to the opposite direction during the test. The diagonal compression stresses in the joint intersection area are verified by the FEM (Diana) in order to calculate the effective compression stresses at the corner of the joint.
The different structural details for exterior joint and its corresponding strut-and-tie model are shown in Figure 6.33. It is suitable to use reinforcement detail that is shown in Fig. 6.33c for monotonic loading, while the reinforcement detail in Fig. 6.33d that is used in our study is more suitable for hysteretic loading of exterior joints.

The joint shear force ($T_h$) is calculated as shown in Equation (6.5). Using the middle longitudinal bars in the column is essential to carry the vertical component ($T_v$) of the diagonal compression force in the joint intersection zone (Fig. 6.31).

$$T_h = T_b - V_c$$  \hspace{1cm} (6.5)

$$T_v = T_c + C_c - V_b$$ \hspace{1cm} (When N is neglected) \hspace{1cm} (6.6)

where:

- $T_h$ = joint shear force carried by transverse stirrups in the joint intersection area
- $T_v$ = vertical component of the diagonal compression force in the joint intersection area which is carried with the middle vertical bars in the column
- $V_c$ = the horizontal force that is applied to the specimens by horizontal hydraulic jack and represents the story shear force
- $V_b$ = reaction of the two link elements at the end of the beam arms
- $T_c$ and $T_b$ = tensile forces in the longitudinal reinforcement bars in column and beam respectively
- $C_c$ = the total compression force in column

The effective design strength $f_{cd,eff}$ for CCC-joint at the corner of the joint can be calculated as shown in Equations (6.3) and (6.4).

### 6.7.2.3 Influence of column axial compression load

The column axial compression force was 25% of the column design axial load ($P_o$) for the specimens S3 and S5, while it was 50% $P_o$ for the specimens S8 and S9.
Figure 6.34: Story drift – column moment diagram for specimens S3, S5, S8 and S9.

Figure 6.34 shows the story drift – column moment diagram for specimens S3, S5, S8 and S9. The figures present that at the same story drifts, the moment on the column section at the face of intersection zone in the case of 25 % $P_o$ is smaller than that in the case of 50 % $P_o$. This refers to the additional moment due to eccentricity in the vertical axis of the column during the test especially in the case of 50 % $P_o$. This eccentricity is proportional to the story drift and in consequence the additional moment increase with increasing of the story drift which led to increase the slope of loading curve in the case of 50 % $P_o$ than that in the case of 25 % $P_o$. In other words, the flexural strength of the column increased with increased column axial load especially at the early cycles of loading. However, strength degradation will be also higher as the column load increased especially at the later cycles of loading, where there is a big eccentricity and then big additional moment.
As shown in Fig. 6.35, the behaviour of the LWC beams in the case of 25% $P_o$ (S3 & S5) is better than that in the case of 50% $P_o$ (S8 & S9), which the peak moment values in the case of S3 and S5 happened at story drift values more than that in the case of S8 and S9 respectively. The flexural beam strength was higher in the case of 25% $P_o$ than that in the case of 50% $P_o$. The additional moment on the column at the face of intersection zone - due to the column eccentricity during the test - balances with an additional moment on the beam section at the face of the joint intersection area which accelerates the peak moment of the beam to happen at small values of story drift.

Column and beam rotation angles - story drift diagrams for specimens S3, S5, S8 and S9 are shown in Fig. 6.36. The column rotation angles in the case of 50% $P_o$ (S8 & S9) were more than that in the case of 25% $P_o$ (S3 & S5), while the beam rotation angles were vice versa at the same story drift. This explains that the reduction of vertical loads on the column by using of LWC in construction of floor slabs in multi-story buildings will reduce the rotation angle in columns and will accelerate it to increase in beams, which means that the most amount of kinetic energy during earthquake excitations will be dissipated by the beams rather than by the columns which achieves the principle of strong columns weak beams.
6.7.2.4 Influence of using lightweight concrete beams

Figure 6.37 shows the story drift against the moment on column and beam for specimens S8 with LWC beams and S10 with NC beam. In general both of the specimens presented an approximately equal behaviour.

Figure 6.37: Story drift – column and beam moment diagram for specimens S8 and S10.

Very small cracks are observed in the joint intersection area for specimen S10, while no cracks are observed in the specimen S8 with LWC beam as shown in Fig. 6.38. The yielding was observed in the LWC beam a little before that in the NC beam which explains why the cracks begin to appear in specimen S10 in joint intersection area before that in specimen S8.

Figure 6.38: Crack pattern for specimens S8 and S10.
The column rotation angles for specimens S8, S9, and S10 at different values of story drift are given in Fig. 6.39. The column rotation angle for specimen S8 is smaller than that for specimen S10, this difference is absorbed by the deformations in the LWC beam in the case of specimen S8. For the specimen S9 with LWC beam reinforced with GFR, the column rotation angles shows more small value especially at high story drift ratios, which may refers to the high elastic deformation in the beam of specimen S9 due to use of GFR bars.

6.7.2.5 Influence of horizontal links in the connection area

As for the interior joints, the transverse reinforcements or the horizontal confinement stirrups in the joint intersection area of exterior joints play an important role in improving the joint shear strength capacity and enhance the ability of the joint to accept moderate deformations with small joint shear strains (distortion).
c. Story drift – beam moment diagram for S5 (left) and S7 (right)

Figure 6.40: Story drift – column and beam moment diagram for specimens S3, S5, S6 and S7.

The horizontal confinement stirrups in the joint intersection area of exterior joints are helpful in providing adequate bearing resistance against the bent tail portion of the beam longitudinal bars, which are catered in the joint area and are subjected to compression force during the cyclic loading of the joint. Without good confinement for the joint intersection area in exterior joints the beam and then the column loss their flexural strength capacity earlier at small value of story drift as shown in Fig. 6.40 for specimens S6 and S7 with low confinement ratio.

Figure 6.41: Crack pattern for specimens S3 (left) and S6 (right).

The good confinement for the joint intersection area as in specimens S3 and S5 prevents the splitting of concrete at the outside part of exterior joints as shown in Fig. 6.41. The splitting of concrete comes from the compression stress beyond the end of beam longitudinal compression bars as shown in strut-and-tie model in Fig. 6.31.
CHAPTER 7

CONCLUSIONS AND RECOMMENDATIONS

7.1 Conclusions

The thesis not only treats the development of new lightweight concrete mixtures with very good mechanical and physical properties, but also the potential practical applications for these new materials in the field of construction as well as its ability to carry static and dynamic loads. As depicted in this study, it can be concluded that:

- Monolithic structures of infra-lightweight fair-faced concrete not only have a high architectural potential but also are very durable. Since no plaster and cladding is needed cost is saved and recycling is made easier.

- The high content of the air voids in the infra-lightweight concrete has a beneficial effect. Surface damage or spalling in severe winter conditions is not a threat with this material.

- Optimising the shape of ribs and using additional parts such as the end head to the surface of reinforced bars especially in low strength material as in infra-lightweight concrete are essential to enhance the behaviour of bond through the good transition of the force from the bars to concrete.

- Using of PP fibres with length not less than 20 mm in infra-lightweight concrete improves the bond behaviour especially with bars that have more numbers of ribs per unit length as in steel reinforcement bars, where the improving of tensile strength has an effective role. However, adding PP fibres reduced the slip at the maximum bond stress, which is required to control crack width in serviceability limit state.

- In the pull-out test of infra-lightweight concrete, use of confinement stirrups reduced the number and the width of cracks especially in the case of glass-fibre rods with head bolt, where the ring tensile strength is more effective than in the cases of steel and glass-fibre rods without head bolt.

- The slope of the bond stress-slip curve after the maximum bond stress is reduced by increasing the confinement ratio, which is recommended for the ductile behaviour of bond failure between infra-lightweight concrete and glass-fibre rods with head bolt.

- In the pull out test of the studied lightweight concrete (LC 30/33), the samples presented very good bond behaviour with steel and glass-fibre rods, however the calculated design value of bond stress between the lightweight concrete and steel reinforcement rods - according to DIN 1045-1 - presents only 20 % of the observed experimental value.

- The steel reinforced lightweight concrete beams showed the same behaviour like normal concrete beams.

- In spite of the high deformation capacity of glass-fibre reinforced lightweight concrete beams, it presented negligible residual deformations after unloading.

- From the economic point of view, the use of lightweight concrete in construction of the floor slabs in tall buildings will reduce the total cost of such buildings through the
reduction of the amount of steel reinforcement, the reduction of foundation volume, and the reduction of vertical members’ cross-sections.

- Regarding technical-construction aspects, one option to achieve the desired seismic design concept of “strong columns – weak beams” is to construct the beam-column joints from a lightweight concrete beam that is reinforced with steel rods or glass fibre rods and normal concrete columns with steel reinforcement rods.

- The beam-column joints with glass-fibre reinforced lightweight concrete beams presented negligible residual deformations after unloading and no cracks are observed at the column or joint intersection area.

- The lightweight concrete beams could be one option to reduce damage and post repair works for beam-column joints after earthquake excitations.

- Using of lightweight concrete in construction of floor slabs in tall buildings will reduce the geometric nonlinearity effect presented by $P\Delta$-effect.

- The transverse reinforcements or the horizontal confinement stirrups in the joint intersection area of interior and exterior beam-column joints play an important role in improving the joint shear strength capacity and enhance the ability of the joint to accept moderate deformations with small joint shear strains (distortion).

- The horizontal confinement stirrups in the joint intersection area of exterior joints provide an adequate bearing resistance against the bent tail portion of the beam longitudinal bars catered in the joint area and prevent the splitting of concrete at the outside part of exterior joints.

- Without good confinement for the joint intersection area in exterior joints the beam and then the column lost their flexural strength capacity earlier at small value of story drift.

### 7.2 Recommendations

The large shrinkage and creep values of infra-lightweight concrete require more studies for a structural design that lead to as little restraining forces as possible. More research is required to find out what maximum strength can be reached with infra-lightweight concrete without losing its favourable insulation properties. On the other hand and because of the high fabrication costs of expanded clay aggregates, the question of what minimum value of thermal conductivity can be reached for infra-lightweight concrete by using other types of lightweight aggregates needs more research.

Among the workability aspects of LWAC, pump ability is still considered to be problematic. The type of lightweight aggregate is said to play a role. However, all lightweight aggregate concrete mixes are said to give problems. More research is recommended.

It is recommended to reduce the factor $\alpha$ to 0.85 which is used to calculate the effective compression strength ($f_{cd,eff}$) for the design of 3-compression force joint at the stress concentration corners of the beam-column joints according to strut-and-tie model method.
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